Design-Construction of The Paramount – A 39-Story Precast Prestressed Concrete Apartment Building

Robert E. Englekirk, Ph.D., P.E., S.E.
Consulting Structural Engineer
Englekirk Systems Development, Inc.
Los Angeles, California
Also, Adjunct Professor
University of California at San Diego

At 39 stories and 420 ft (128 m) high, The Paramount (located in San Francisco, California) is the tallest concrete structure in addition to being the tallest precast, prestressed concrete framed building in Seismic Zone 4 (a double record). It is the first major high rise building to be braced by an architecturally finished exposed precast concrete ductile frame. The reinforcement used to create this seismic ductile frame includes post-tensioning and high strength reinforcing steel. All this represents a major milestone in the development of precast/prestressed concrete. The building is basically an apartment complex, although the lower floors accommodate retail space, vehicle parking and recreational amenities. This article presents the design considerations, construction highlights, research and development, and code approval process that led to the realization of this structure.

Long-time PCI Professional Member **Robert E. Englekirk** has been a strong advocate of precast concrete construction for many years. A PCI Fellow, he is the author of numerous technical articles, three of which have won the Martin P. Korn, Robert J. Lyman and Charles C. Zollman PCI JOURNAL Awards. He has just completed a book titled "Seismic Design of Concrete and Precast Concrete Structures to a Performance Criterion," which will be published by John Wiley & Sons in early 2003.

Soaring majestically amidst the other high-rise buildings in San Francisco is The Paramount — a 39-story residential apartment tower that reaches 420 ft (128 m) skyward (see Fig. 1). Costing nearly $93 million, the newly constructed building is prestigiously located at Third and Mission across from the famous Moscone Center, further enriching the city's world renowned skyline.

What distinguishes this building from the other highrises surrounding it is that the structure incorporates a novel precast hybrid moment resisting frame that is particularly effective in the severest seismic regions of the United States and indeed the world. As such, The Paramount is not only the tallest concrete structure built in Seismic Zone 4, but it is also by far the tallest precast, prestressed concrete framed structure built in a region of high seismicity.

From a precast concrete perspective, it is the first major high rise building to be braced by an architecturally finished exposed concrete ductile frame.

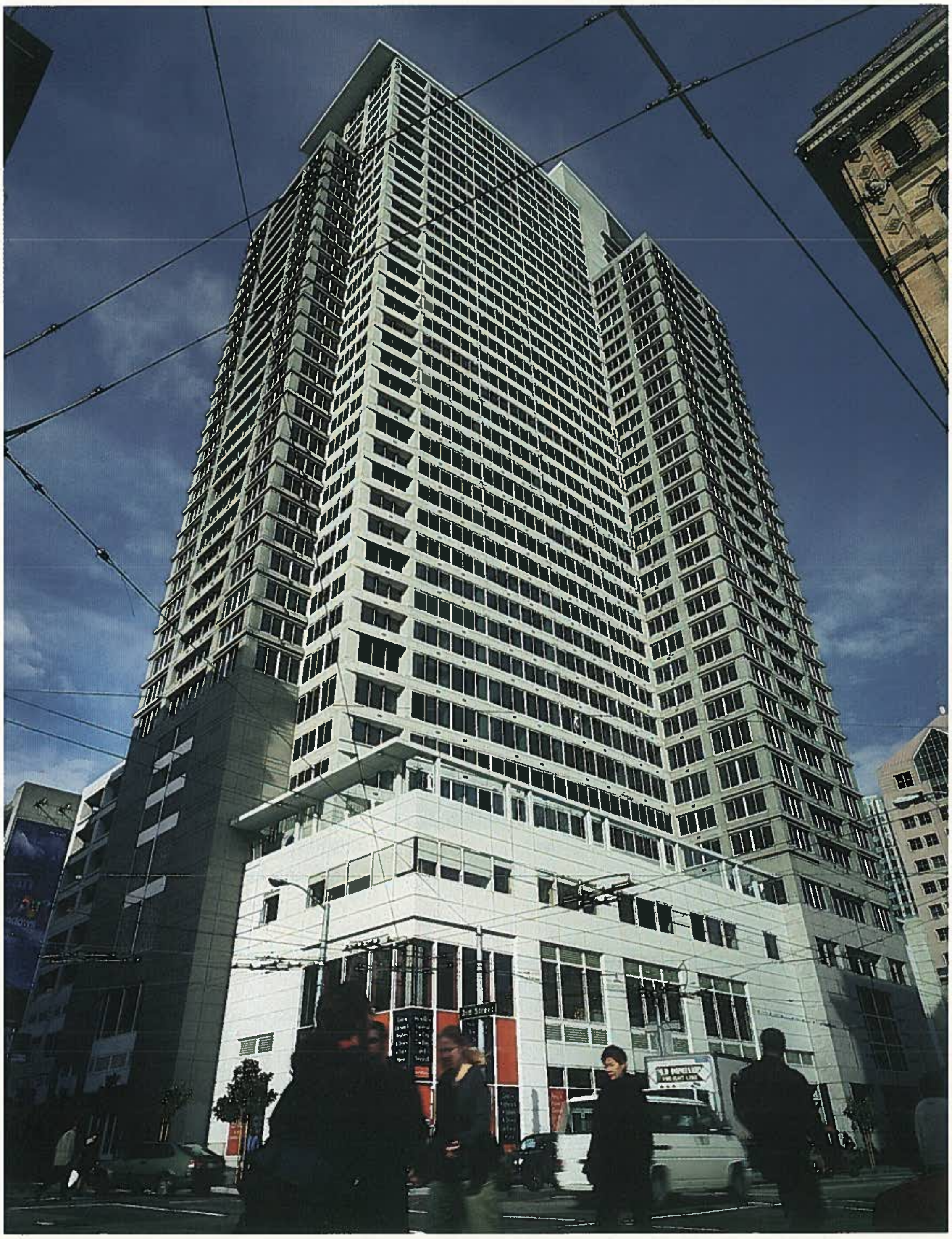


Fig. 1. The Paramount at Third and Mission, San Francisco, California. Photo courtesy: Kwan Henmi, Architecture and Planning.

The reinforcement used to create this ductile frame includes both post-tensioning strand and high strength mild steel reinforcing with a yield strength of 120 ksi (8280 MPa). This combination of materials also represents a significant technological breakthrough.

The accomplishment of these milestones is a credit to the courage and perserverance of the design-construction team:

- Owner: Third and Mission Associates, Inc. – For courageously accepting a concrete high rise building with a brand new structural bracing system in a very severe seismic area.
- Architects: Kwan Henmi, Architecture and Planning, and Elkus/Manfredi Architects, Ltd. – For very imaginatively integrating the structural framing system into the architectural design, and thereby creating a very functional and beautiful building.
- Structural Engineers: Robert Englekirk Consulting Structural Engineers, Inc. – For pioneering the structural engineering concept and developing the details of the bracing system for this building.
- Contractor: Pankow Residential Builders II, Ltd. – For enthusiasti-

Fig. 2. Artist's rendering of The Paramount. Photo courtesy: Kwan Henmi, Architecture and Planning. Artist: W. Yeliseyev.

cally embracing the seismic bracing system and sponsoring the necessary research to test the system at the University of Washington.

How the design-construction team realized their dream is the subject of this paper.

SYSTEM SELECTION

The obvious first question to ask is, why, in such a pedantic industry, would any one of the team members elect to follow such a difficult road? New systems are only developed when it is clear that they will be less expensive and better than the standard they propose to replace. The author is not sure whether the "better" is essential, but in this project, both conditions were in fact met.

Economies in construction are produced by an efficient use of materials and/or reducing the time required to deliver an occupiable building. The Paramount team fulfilled both of these parameters. Fig. 2 shows an artist's rendering of the project, while Figs. 3 and 4 show a plan and elevation, respectively, of the building.

Two expensive building components, namely, the architectural cladding and the seismic bracing sys-

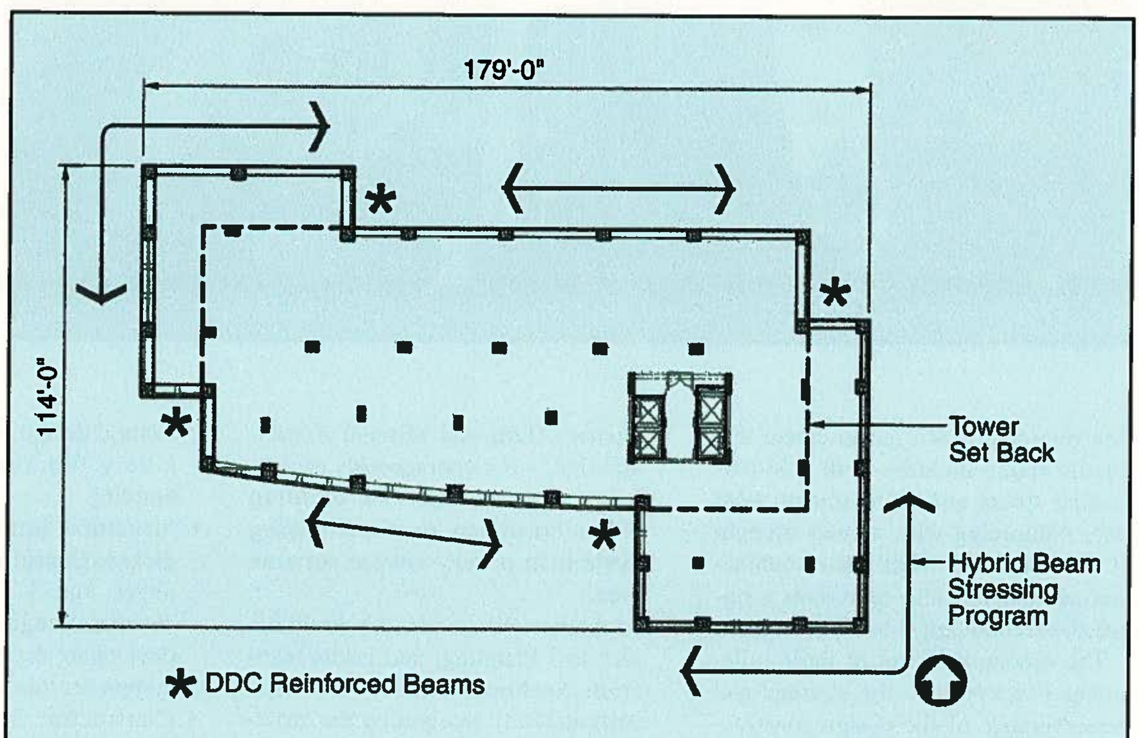


Fig. 3. Typical plan of building.

tem, were efficiently combined on this project. This not only resulted in lower construction costs, but also produced a more water resistant exterior, an especially important benefit in San Francisco. When the exterior water barrier is applied to the frame of a building located in a seismically active area, it must be designed to accept the movement expected during an earthquake.

This means that slip joints must be introduced that can accommodate differential displacements of as much as $2^1/_2$ in. (64 mm). This is usually accomplished with the introduction of large caulked joints which, in addition to being expensive, require significant maintenance.

The construction of the Paramount exterior avoids this problem, because the precast components are rigidly attached to each other and designed so that the pieces move together. Further, the beams are post-tensioned to 760

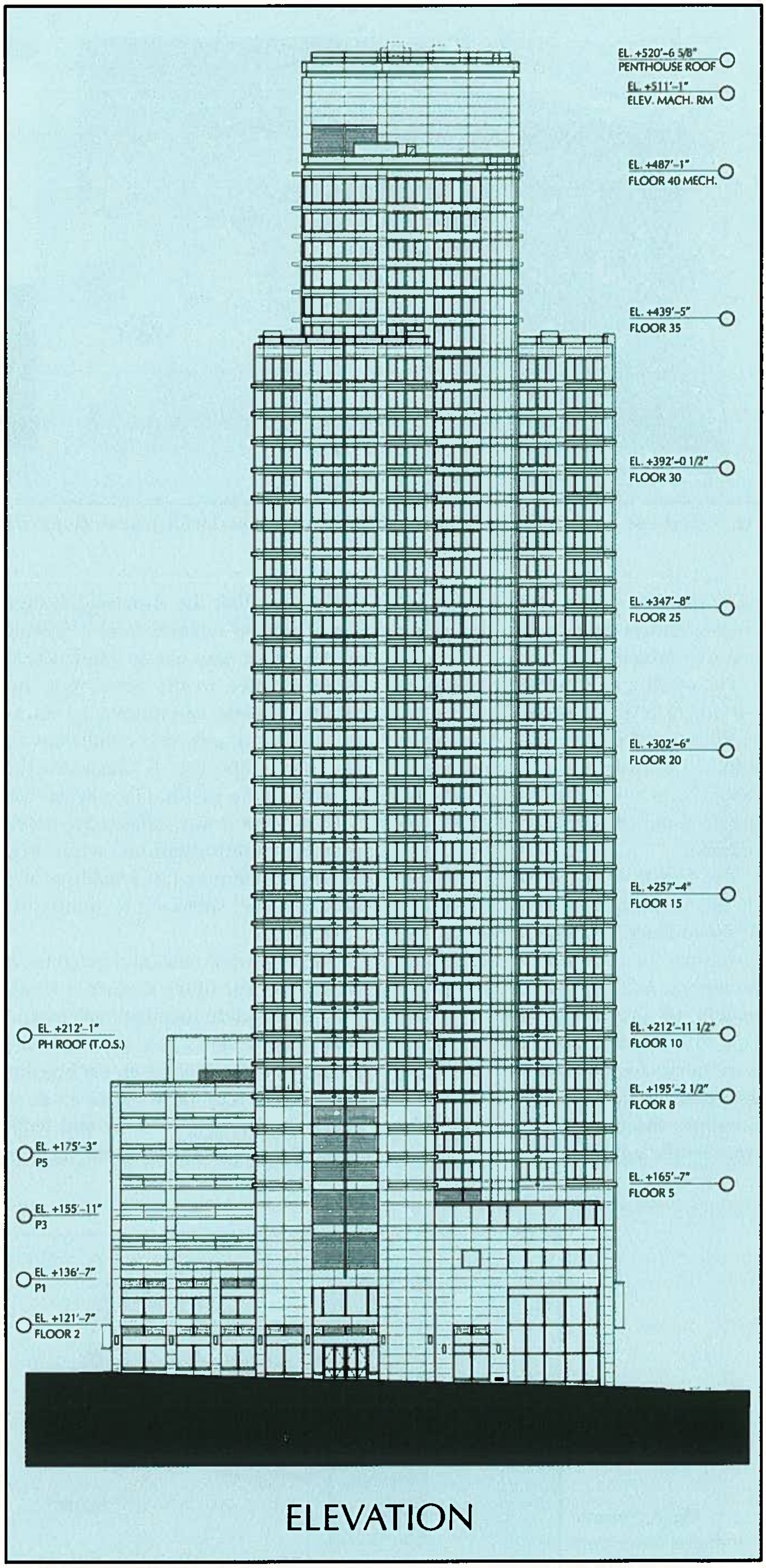


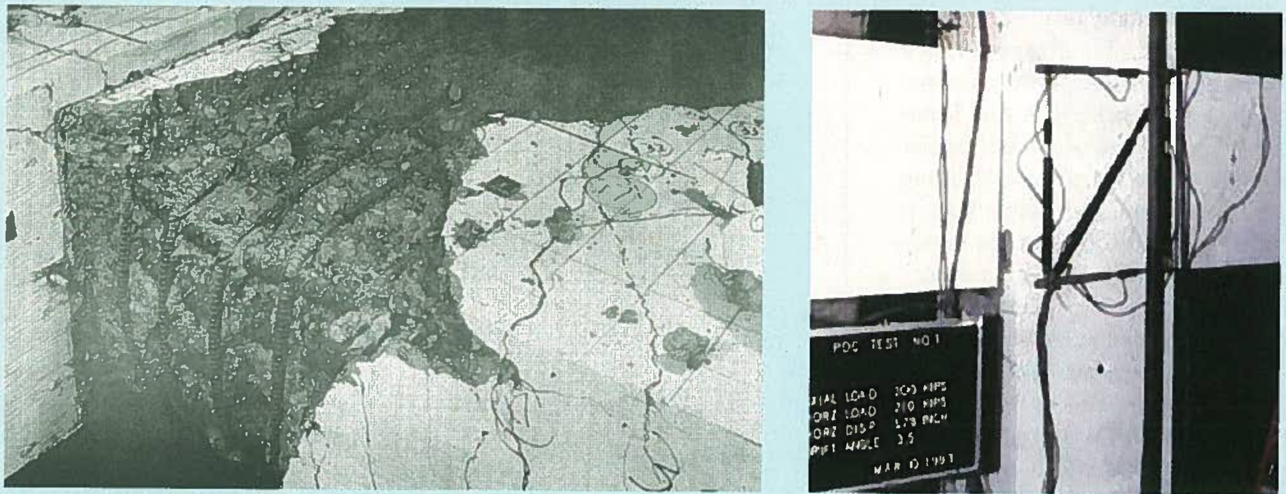
Fig. 4. Elevation of building showing heights of various floors. Drawing courtesy: Kwan Henmi, Architecture and Planning.

BUILDING FACILITIES

The building contains 486 apartment units comprising a total of 660,000 sq ft (61380 m^2) area of rentable space. The lower eight floors and one basement level accommodate a variety of functions within a floor area of 31,000 sq ft (2880 m^2). Retail space occupies most of the first and second floors. Residential amenities include a leasing office and business center on the third floor with a fitness center and outdoor swimming pool on the fourth floor.

Floors 3 to 7 of the north side of the building accommodate parking for 350 vehicles, including an all-valet parking station served by elevators instead of ramps. Residential units are located on the south side of the building at the fourth through seventh floors.

The eighth floor serves as a podium for the typical 13,700 sq ft (1274 m^2) residential floors above on Floors 9 through 33. The building steps back at the 34th floor to a 9900 sq ft (920 m^2) floor for Floors 34 through 39. Level 40 is an outdoor recreation deck area.



(a) Cast-in-place beam

(b) Precast beam-column frame

Fig. 5. Post-test condition of cast-in-place beam and precast ductile beam-column frame.

psi (5.2 MPa) and the columns are, of course, always subjected to high compressive stresses.

The result is a watertight enclosure. An added level of protection is provided by caulking the beam-to-column joints. Because the joints are small [about 3/4 in. (19 mm)], and are always prestressed, maintenance should be minimal.

The façade of the building consists of 732 sandblasted precast beams and 478 two-story precast moment frame columns. In addition to the frame members, 641 architectural precast panels, 68 precast gravity columns, and 312 precast, prestressed beams were fabricated off-site. By using such an extensive quantity of precast components, the construction schedule was significantly expedited.

The fact that the structural system provided was superior from a seismic perspective was easily (and often) demonstrated to the developer, investors, lenders, and insurers by showing them the post-test conditions of tested models. Fig. 5 illustrates the condition of a cast-in-place beam (see Fig. 5a) after it was subjected to earthquake-like deformations, while Fig. 5b shows the post-test condition of a precast frame subjected to similar deformations.

The improved structural behavior is a characteristic of the manner in which precast concrete members are assembled. Fig. 6 shows how the beam and column interact during an earthquake. In essence, they must rotate so as to allow the building to sway and withstand the shock of the earthquake. In the precast system, this is accomplished by opening a gap (θ) between the beam and column.

In the cast-in-place system, this gap distributes itself over a finite region and causes the outer shell of the concrete to spall (see Fig. 5a). Demonstrating the improved behavior of the precast system and the basis for it made the system an easy sell to the design-construction team, its financial backers, and the building officials.

The final building system was the result of many design iterations that considered both functional, aesthetic and construction needs. The floor plan for the typical floors (up to Floor 34), as well as the setbacks required of Floors 35 through 39, is shown in Fig. 3. The entire perimeter of every tower floor including the setbacks consists of precast concrete spandrel beams and two-story precast concrete columns. The floor was cast-in-place post-tensioned concrete stressed through the top of the slab along the south edge where the slab span is short.

Two types of structural lateral force resisting systems are utilized in the building. Below the eighth floor podium, the varying slab elevations and the required occupancy separations between parking and living spaces created a natural location for shear walls. Consequently, a shear wall system and a precast and cast-in-place moment frame bracing system

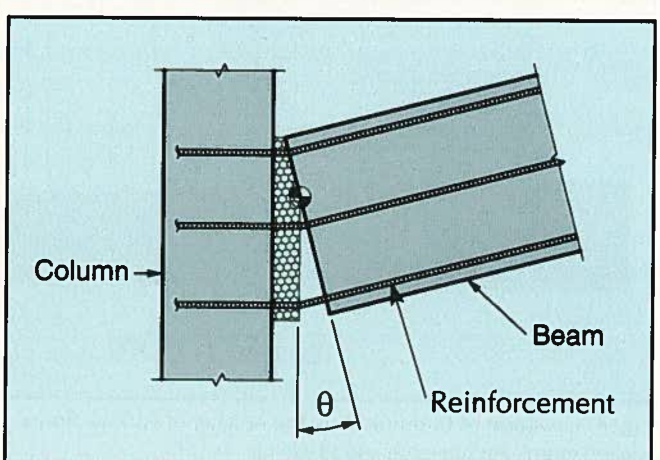


Fig. 6. Seismic induced movement (exaggerated) of precast frame components.

was implemented from the mat foundation to the eighth floor. The shear walls terminate at the eighth floor.

Above the eighth floor, a perimeter precast moment frame was developed using both the Precast Hybrid Moment Resistance Frame (PHMRF) System and the Dywidag Ductile Connector (DDC) System®. The PHMRF system is the predominant frame system utilized for the multi-bay frames. The DDC System was used at single-bay frames that occur at re-entrant corners of the building where the effective post-tensioning force required by the PHMRF could not be developed.

Fig. 7 is a schematic of the post-tensioning anchorage details for an exterior column, while Fig. 8 shows the reinforcing cage of a precast column. The newly developed DDC reinforced frame beam is shown in Fig. 9. A typical corner column detail is shown in Fig. 10. Fig. 11 shows the post-tensioning jacking operation.

Typical frame columns are 36 in. (914 mm) square up to the 20th level and reduce to as small as 24 x 36 in. (610 x 914 mm) above. Frame beams are typically 24 x 36 in. (610 x 914 mm) and were set flush to the outside face of the columns.

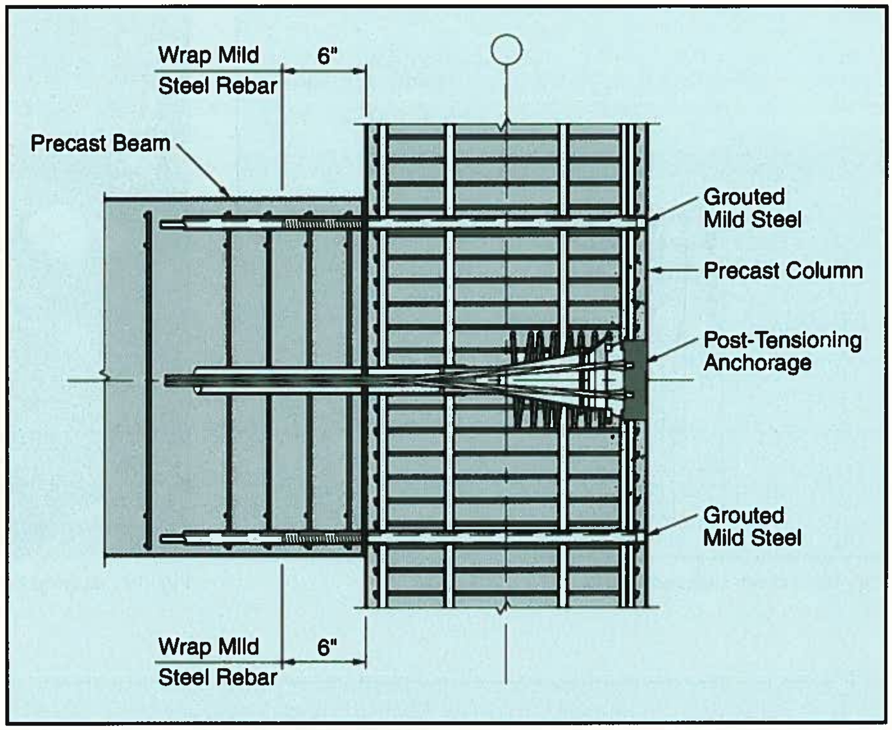


Fig. 7. Reinforcing details of exterior column.

THE DESIGN PROCESS

Neither the building nor any portion of its seismic system fit within the framework of the existing building code. The functionally logical bracing program for the building included a shear wall base, which extended from the foundation to the eighth level, for this was the top of the parking structure that occupied the north side of the building. The precast concrete ductile frame braced the building above Level 8.

Since buildings over 160 ft (48.8 m) [now 240 ft (73.1 m)] require the existence of a ductile frame or a dual system (shear wall plus ductile frame), an exception was necessary to allow a stacked bracing system-frame on top of the shear wall. The Hybrid System, which provided the bulk of the lateral support, had yet to be approved, while the DDC® System represented a modification of the precast system approved by the International Conference of Building Officials (ICBO).[1] Accordingly, the project lacked a well-defined seismic criterion.

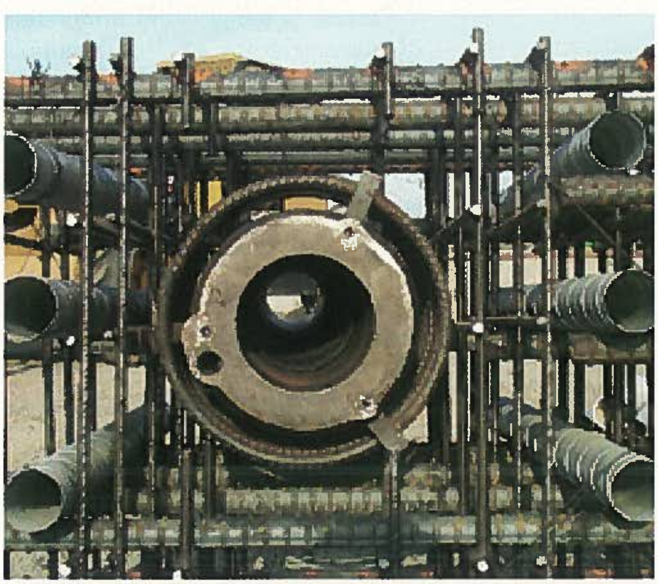


Fig. 8. Reinforcing cage of precast column.


Fig. 9. DDC reinforced frame beam.

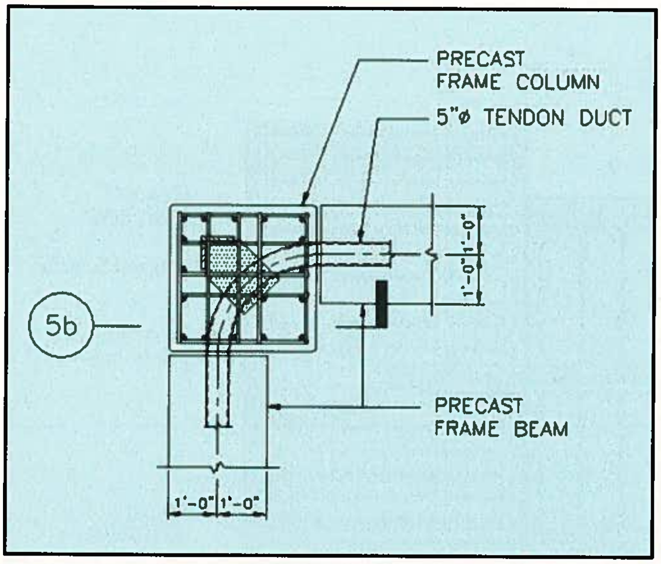


Fig. 10. Corner column detail.

Fig. 11. Jacking operation.

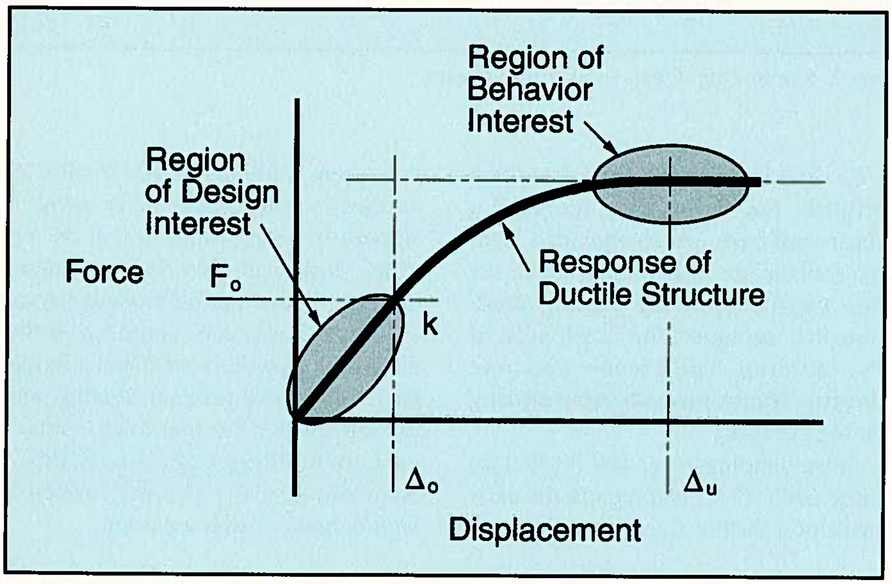


Fig. 12. System behavior idealization.

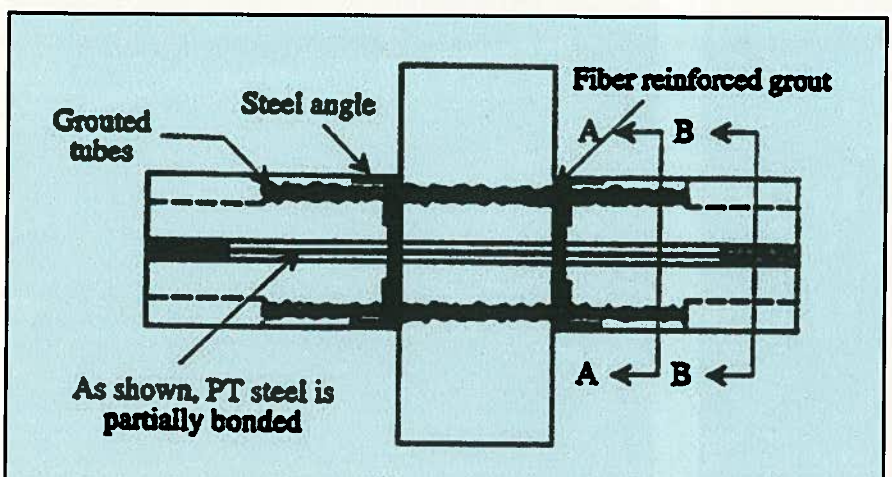


Fig. 13. Hybrid beam system as developed by NIST.[5]

In some ways, this lack of a specific criterion was an advantage. This is because building codes should not be used as the exclusive criterion for the design of a complex structure. Current building codes have as their focus the elastic behavior range. In essence, the structure is designed to a strength criterion (F_o) (see Fig. 12).

The design engineer understands that the structure will respond when subjected to a major earthquake in a displacement range many times that associated with the elastic design load (F_o, Δ_o) (see Fig. 12). The probable response range is understood to be in the vicinity of Δ_u (see Fig. 12) and this, of course, should be the region of primary concern to the design engineer.

Current building codes bridge this displacement gap ($\Delta_o - \Delta_u$) by prescriptions intended to cover all eventualities. Obviously, it is not possible to generically cover all possible systems by prescription. Consequently, conflicts tend to arise, and this often results in the acceptance of less than optimal behavior.

The design approach that has as its focus the region of behavior interest (see Fig. 12) is now referred to as Performance Based Design, and only attempts to codify this approach are new, for many designers including the author have used it for 30 years. The essence of Performance Based Design is to predict member strain states at the anticipated level of building drift.

Strain objectives or limit states are established experimentally, and the linkage between analysis and component behavior is established through the testing of large-scale models of members and subassemblies. On this project, exclusive reliance on a Performance Based Design approach was adopted by the design team, although they, of course, were required to demonstrate compliance with the intent of the code to obtain a building permit.

PRECAST SEISMIC BRACING SYSTEMS

The combining of precast components by post-tensioning is not new. Professor Robert Park (University of Canterbury, Christchurch, New Zealand) and his associates tested several post-tensioned subassemblies in the 1960s. Clearly, the concept preceded the ability of builders to exploit it, so the idea was not pursued.

In 1978, the author proposed the development of a post-tensioned assembly and subsequently presented the concept at several workshops.[2-4] H. S. Lew at the National Institute of Standards and Technology (NIST)[5] obtained a grant to develop what became known as the Hybrid System. Ultimately, the system described in Fig. 13 was produced after a number of iterations at the NIST test facility.

This particular model, however, needed additional work to satisfy the needs of the Paramount project because the angles that armed the corners of the beams (see Fig. 10) were not acceptable. Also, the concrete strain limit states required to effect a performance based limit state were not established as a consequence of arming the corners. Further, the strength of the beam-to-column joint was not established because the strength of the beam-to-column joint tested significantly exceeded the demand imposed on it.

Subsequently, the Hybrid System was tested as one of the four precast frame systems used in the five-story PRESSS building conducted at the University of California, San Diego, in September 1999. The PRESSS research program was funded primarily by NSF with strong industry support from PCI and PCMAC.

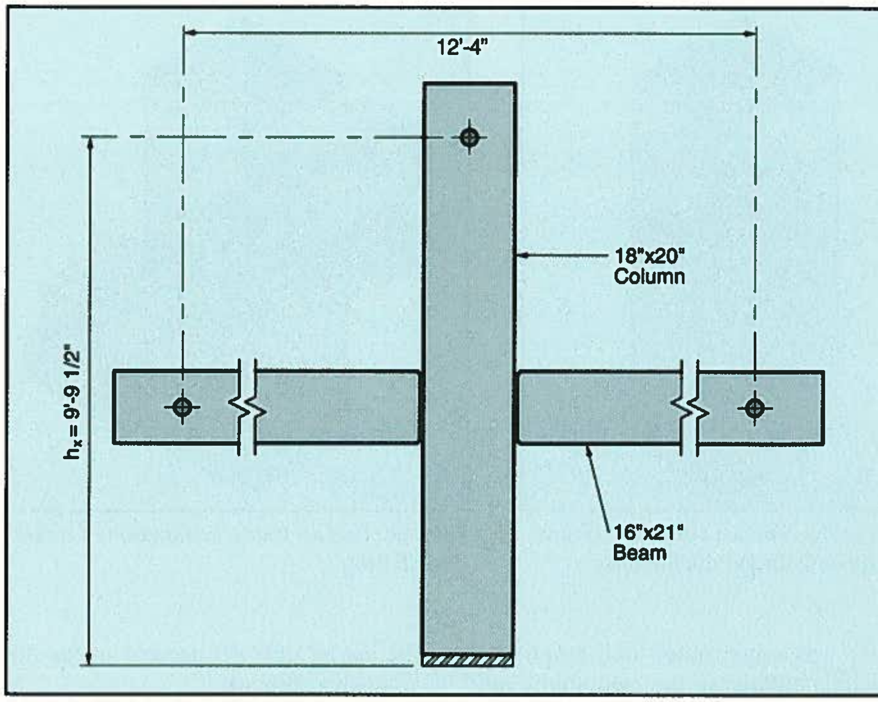


Fig. 14. Interior Hybrid beam test assembly.

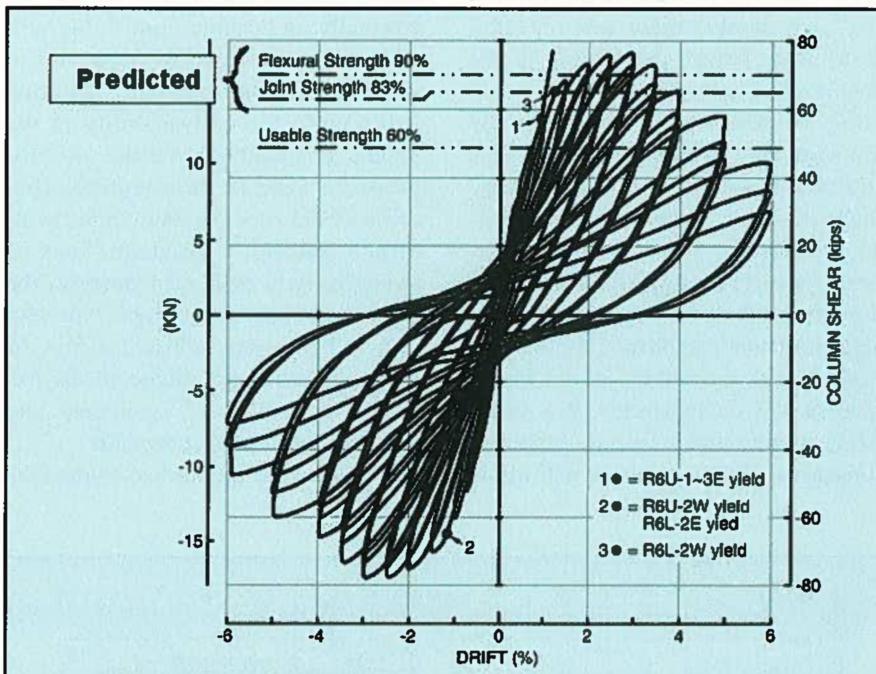


Fig. 15. Behavior of Hybrid beam test assembly (see Fig. 14).

Additional testing was required to develop a performance based design criterion for the Hybrid System, and this was undertaken at the University of Washington. Interior, exterior, and corner subassemblies were tested. The attempt was to follow the guidelines contained in ACI's proposed acceptance criterion.[6] The interior test subassembly is described in Fig. 14.

This subassembly was modeled so as to represent a two-thirds scale model of the frame proposed for the Paramount building. The ACI acceptance criterion required that the subassembly be designed prior to testing so as to predict its strength and deflection, as well as the point at which the subassembly would start losing its strength. The subassembly (see Fig.

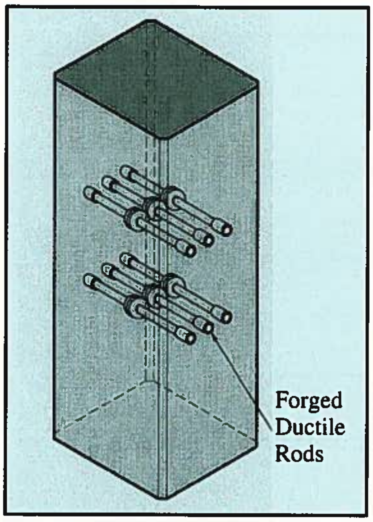


Fig. 16a. Precast concrete column showing forged ductile rods.

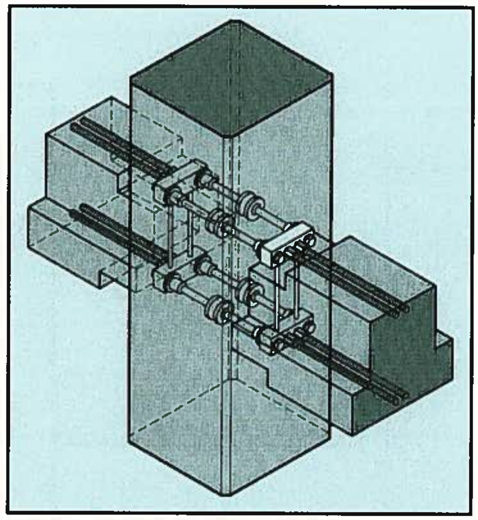


Fig. 16b. Precast frame application of forged ductile rods.

14) was constructed and assembled strictly following the procedures proposed for the project. The experimental model was then subjected to displacements of increasing magnitude, and each displacement was repeated three times before proceeding to the next level of deformation.

Fig. 15 describes the behavior of the test specimen. The strength that was attained exceeded that predicted by analysis. The predicted nominal flexural strength was 90 percent of that experimentally attained, while the strength predicted for the joint was 83 percent of that established by the test. Accordingly, the validity of the design process was demonstrated. It is interesting to note that a recently proposed consensus design criterion will allow the use of only 60 percent of the developable strength.

From a displacement perspective, the performance of the subassembly also exceeded the desired objectives. Normally, in Seismic Zone 4, the subassembly is designed to drifts of 2 to 2.5 percent. The acceptance criterion looked into the deformability of the system and required that the test program be able to demonstrate that drifts of 3.5 to 4 percent could be attained without significant loss of strength. At a drift of 4 percent, the third cycle loss of strength was less than 30 percent, while no loss of strength was experienced in the expected drift range (2.5 percent), and this was considered acceptable.

For years, the author had studied the behavior of cast-in-place beams subjected to post-yield seismic deformations in an attempt to predict the behavior of limit states (see Fig. 5a). About 15 years ago, it occurred to him that the best approach was to avoid the principal causative action responsible for the deterioration of the concrete described in Fig. 5a.

The high concrete strains imposed on the unconfined shell of the beam described in Fig. 5a are exacerbated by the tendency of the once overstrained reinforcing bars to buckle outward when subjected to compression loads. It seemed virtually impossible to prevent this outward displacement of the bar clearly visible in Fig. 5a with confining ties.

The solution seemed so obvious once it occurred to the author — why not simply move the yielding element out of the frame beam and into the column where the yielding bar could be recompressed without damaging the surrounding concrete? This relocation was made possible through the development of a forged ductile rod which could be placed in a column (see Fig. 16a). A high strength bar [120 ksi (8280 MPa)] would then be screwed into the end of the ductile rod (see Fig. 9). Ultimately, this was the system that was used in the Paramount project.

The ductile rod concept seemed appropriate for use in all precast buildings, so the author worked with Dywidag Systems International to develop the precast subassembly described in Fig. 16b, which was the originally proposed short and single span solution (locations marked with an asterisk in Fig. 3). The builder was concerned, however, that bolt alignment might become a problem, so he opted to use the system described in Fig. 9.

Alignment problems should not be anticipated in the precast system (see Fig. 16b) because $^1/_2$ in. (12.7 mm) tolerances are provided for in the connection. During this same period (2000-2001) the DDC System (see Fig. 16b) was used to brace the Hollywood Highland project, and 6700 bolts were placed absent any misalignments despite the fact that members were assembled in self-stabilizing towers more than 100 ft (30.5 m) high.

Tests had been performed on a cast-

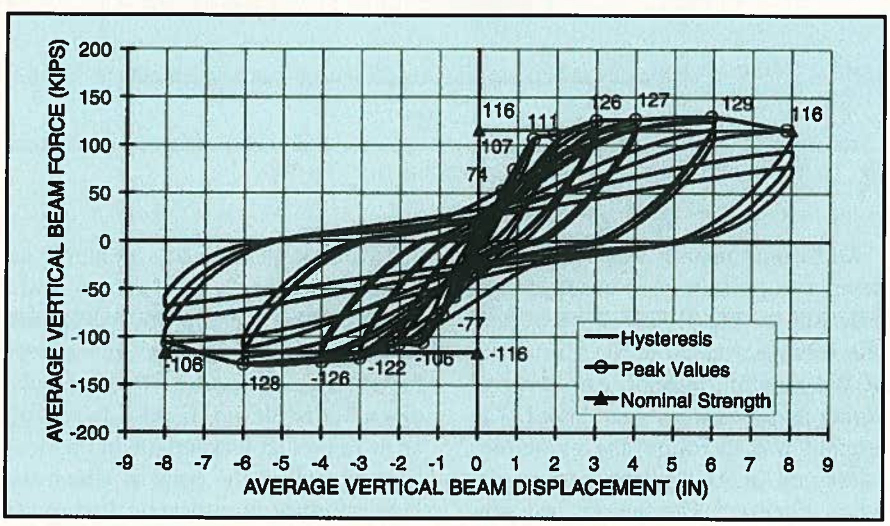


Fig. 17. Precast frame application of forged ductile rods.

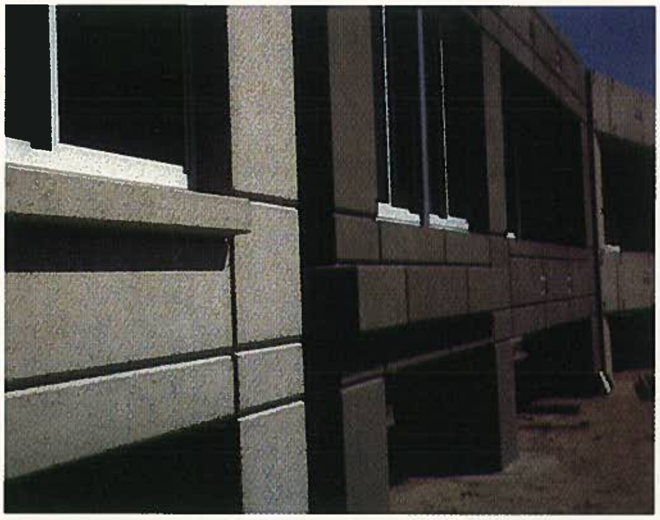


Fig. 18. Mockup of architectural precast panel assembly at precaster's yard.

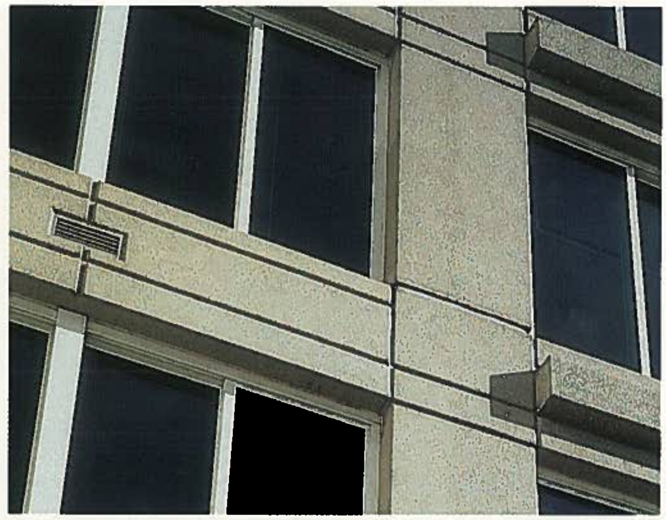


Fig. 19. Architectural precast panel assembly installed on building façade.

in-place DDC subassembly. These test results are described in Reference 7. In this case, the nominal strength of the subassembly was maintained through a drift of 6.7 percent [δ_b = 8 in. (203 mm)]. The resulting damage was only cosmetic (see Fig. 5b).

The DDC system required no additional testing in support of its use on the Paramount project, and this is because a special product approval had been granted Dywidag Systems International by the ICBO in the early 1990s.[1] Note that the ICBO approval process is based on submitted experimental and analytical evidence which supports the design process, as well as production control procedures. The result is a quality assured product and design procedure. This approval process is essential to the responsible advancement of precast concrete, for the days of closely guarded secret homemade connections are gone forever.

BUILDING DESIGN

The adopted Performance Based Design required that the probable range of building displacements be predicted. This was done through the use of several analytical procedures which have evolved over the last 30 years. Response spectrum based procedures are the key conceptual design tool. Once the design has been developed, elastic three-dimensional time histories are performed.

This process involves modeling the anticipated ground motion both in terms of intensity and characteristics. The result is a number of sets of earthquake ground motions, which are then fed into the base of the analytical model of the building. This process suggests the extent of building movement.

In the case of the Paramount apartments, the movement is expected to be about 40 in. (1016 mm) at the roof if the design ground motion were to occur. However, do not rush to the roof in wild anticipation of an exciting ride, for this design earthquake is expected to reoccur at intervals of about 500 years!

The analytic testing of the building does not stop with elastic behavior predictions because the anticipated level of ground motion will cause the frame beams to reach the inelastic behavior range (see Fig. 15). To predict the extent of post-yield demand on the frame beams, inelastic time history

Fig. 20. Progress view of precast frame erection. Photo courtesy: Kwan Henmi, Architecture and Planning.

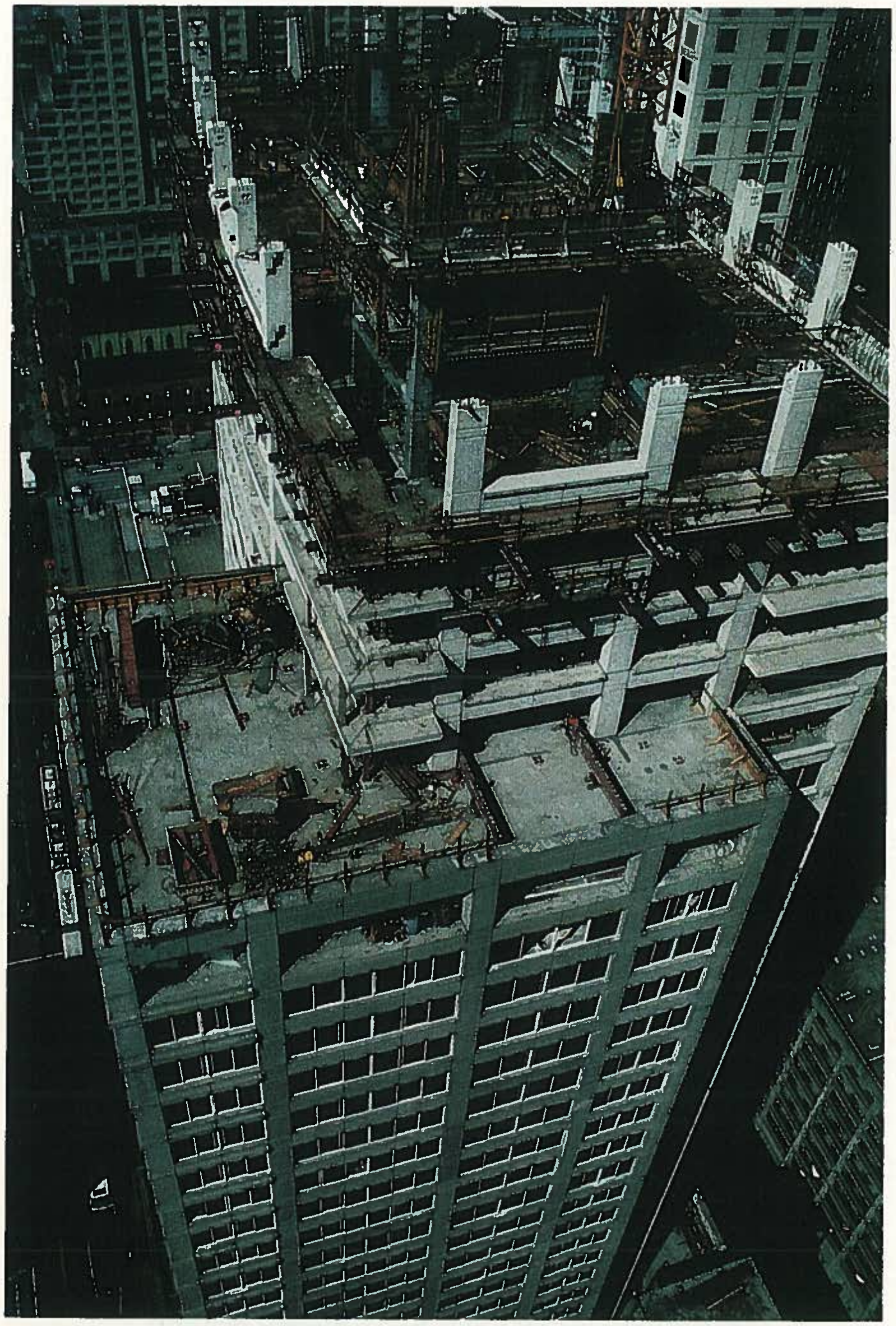


Fig. 21. Erection of precast perimeter frames nearing top of building. Photo courtesy: Kwan Henmi, Architecture and Planning.

analyses were performed on the various bracing frames. The design team concluded that post-yield rotations would be in the 1.0 to 1.2 percent range, and this is well below the experimentally confirmed limit state of 4 percent (see Fig. 15).

Most buildings tend to be rectangular in plan in which one dimension is often 2.5 to 3 times the other. This is a characteristic of the Paramount apartments. The design of this type of configuration must consider the torsional response, especially in the post-yield range.

When a building responds to earthquake excitations that drive it into the post-yield behavior range, the center of rigidity will gravitate to the stronger bracing element. For example, the eastern-most frame of the Paramount building would become the torsional pivot, and thus would cause the west frame to respond in an undesirable manner from a displacement perspective. Further, any imbalance in strength would limit the restoring force.

This concern was mitigated by balancing the strength and stiffness of the

Table 1. Breakdown of precast concrete components.

• 478	Precast Moment Frame Columns [3 x 3 x 18 ft]
• 732	Precast Moment Frame Beams [2 x 3 (12 to 24 ft long)]
• 68	Precast Gravity Columns [2 x 2 (40 ft long)]
• 641	Architectural Precast Panels [50 to 100 sq ft each]
• 312	Prestressed Beams [40 ft long (average)]
2231	Total Components

Note: 1 ft = 0.3048 m; 1 sq ft = 0.093 m².

east and west frames in both the elastic and post-yield behavior range. The balance attained is much more important than compliance with any prescribed strength objective. Thus, it is important to ensure that the balancing operation not reduce the available level of ductility in the weaker frame element (west frame in this case).

The design of the precast concrete frame systems is fairly straightforward. The design of a frame beam is described in Appendix B.

CONSTRUCTION HIGHLIGHTS

The fabrication, transportation and erection of the precast concrete components, together with the overall construction operation and schedule, was carried out by Pankow Builders. The precast components were fabricated in Corcoran, California, starting in January 2000. They were shipped to the project site by truck-trailer, a distance of about 200 miles (330 km). In all, 2231 precast pieces were produced (see Table 1).

Fig. 18 shows a mockup of an architectural precast panel assembly at the precasting yard. Fig. 19 shows a view of the architectural precast façade installed on the building. Figs. 20 and 21 show various erection phases of the building. Fig. 22 is a completed view of the building amidst the other highrises in San Francisco.

Concrete strengths were specified as follows:

Fig. 22. Finished view of The Paramount amidst other high rise buildings in San Francisco. Photo courtesy: Kwan Henmi, Architecture and Planning.

- For the columns: 6000 to 8000 psi (41 to 55 MPa).
- For the beams: 5000 psi (34 MPa).

Formwork for casting the architectural panels, beams and columns was provided by Hamilton Form Company, Inc.

The entire construction comprised a 26-month schedule. The foundations of the building were started on November 15, 2000, and subgrade work was completed in March of 2001.

The superstructure was completed in 16 months. Erection of the precast components started slowly but, as work progressed, an average production rate of $2^1/_2$ floors per month was attained.

A major time benefit was the rapid enclosure of each floor which made possible the installation of electrical and mechanical accessories as well as other fixtures. This made it possible for tenants to occupy the premises by October 26, 2001 – less than 2 years after the start of construction.

From the mat foundation to the eighth floor, a cast-in-place moment frame bracing and shear wall system was used. This was necessary because of the varying slab elevations and non-repetitive elements involved. Above the eighth floor, a perimeter precast moment frame was used.

The floor construction cycle, which ultimately took 5 days, started with the placement of half of the two-story precast column and ended with the raising of the flying forms by one story.

These forms were designed to support the weight of the precast beams. Most of the beams were hybrid and, as a consequence, were connected to the columns by a concentric post-tensioning system together with grouted mild steel bars.

The precast beams were typically post-tensioned by 19 – 0.6 in. (15.2 mm) diameter strands. Baumesh was used to facilitate the assembly of the column cages. Short spans, whose locations are marked by an asterisk in Fig. 3, were constructed using Dywidag Ductile Rods®, which were cast in the precast columns, and Threadbars®, which were turned out of the interior couplets and into the ductile rods. Note that precast panels served as the outside form.

Two tower cranes were used to erect the precast members and relocate the flying forms. Despite the prototypical nature of the project, the contractor reported that the erection and assembly process went very smoothly, and that noise and pollution were very minimal.

One of the few problems that needed to be resolved was the corner conditions where, in the absence of a more viable solution, two exterior stressing assemblies would need to be placed. Offset assemblies were tested and proved to be effective. However, they were abandoned in favor of an "around the corner stressing."

The author developed a piece of hardware consisting of bent pipe sections, restrained by straps connected to an anchoring angle. The stressing program involved a significant amount of testing to ensure that strands could be placed and stressed without damage.

The adopted program involved sequentially tensioning the strand groups in increments from both ends. Columns were connected using Splice Sleeves® (supplied by Splice Sleeve North America, Inc.), which are capable of developing the breaking strength of the bar.

The total cost of the project was $92.7 million. The precast cost was $8.9 million. Based on gross square footage of rentable area, this amounts to $140 per sq ft. This cost figure is fairly good considering the building's location and seismic environment.

CONCLUDING REMARKS

The construction of this innovative project flowed smoothly and was completed on schedule. The credit for this achievement lies with the Pankow project management team. Assembly procedures were developed in the precasting yard and carefully documented, as were the procedures for ensuring high quality.

An even wider use of precast components is possible. The floor systems, for example, could have been constructed using pretopped hollow-core slabs or double tees. All that is really required is imagination, teamwork, and careful planning.

From a design perspective, the performance and the assembly advantage associated with the development of yielding precast concrete frame systems is clear. The emphasis to date, insofar as code development is concerned, has been on emulative assemblies where precast concrete is assembled so as to perform as though it were cast-in-place concrete. The codification process must strive to encourage the yielding approach and avoid the restrictive provisions that dominate current codes.

This design philosophy is consistent with the performance objectives of the next generation of seismic codes. Such a course is particularly important because the structural systems used on the Paramount apartment building are just examples of what can be accomplished. Hopefully, they will engender a whole new approach to construction.

Precast building assemblies must continue to be explored, and these should logically include bearing wall systems.[10,11] Contractors and especially precast contractors must be willing to spend the time to explore alternative systems, for the immediate return seems attractive and the future looks extremely bright.

EPILOGUE

In retrospect, the author finds it hard to believe that during the span of his professional engineering career, Precast Concrete, as a seismic bracing system, has evolved from a prohibited structural system to a system of preferred choice. The successful completion of this building, as well as other structures currently under construction, attests to this metamorphosis. Clearly, the curved façade and sculpted finish do not fit the traditional stoic image of structural precast concrete and attest to the fact that structural precast concrete has truly come of age.

POST-SCRIPT

Since its completion last year, The Paramount has received many accolades from the design community and visitors from around the world. In June of this year, a jury of peers judging the 2002 PCI Design Awards Program, bestowed upon the structure the Harry H. Edwards Industry Advancement Award. The jury comments were as follows:

"The successful completion of this 39-story precast, prestressed concrete building brings to fruition the culmination of a ten-year research effort in which the best minds developed an innovative seismic lateral force resisting system. This structure is a classic example of combining the knowledge and wisdom learned from academia, engineering, architecture and construction in a landmark project. The path is now open for others to apply this technology with precast/prestressed concrete structures in high seismic areas."

CREDITS

Owner: Third and Mission Associates, Inc., Irvine, California

Architect of Record: Kwan Henmi, Architecture and Planning, San Francisco, California

Design Architect: Elkus Manfredi, Boston, Massachusetts

Structural Engineers: Robert Englekirk Consulting Structural Engineers, Inc., Los Angeles, California

General Contractor: Pankow Residential Builders II, Ltd., Altadena, California

Precaster: Mid-State Precast, L.P., Corcoran, California

REFERENCES

1. Englekirk, R. E., "An Innovative Design Solution for Precast Prestressed Concrete Buildings in High Seismic Zones," PCI JOURNAL, V. 41, No. 4, July-August 1996, pp. 44-53.
2. Englekirk, R. E., "An Evaluation of the State of the Art in the Design and Construction of Prefabricated Buildings in Seismically Active Areas of the United States," Earthquake Resistant Reinforced Building Construction, June 1978.
3. Englekirk, R. E., and Selna, L.G., "Criteria for the Design of Prefabricated Concrete Systems," ATC Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads, April 27-29, 1981, pp. 639-656.
4. Englekirk, R. E., "Overview of the ATC Seminar on Design of Prefabricated Concrete Buildings for Earthquake Loads," PCI JOURNAL, V. 27, No. 1, January-February 1982, pp. 80-96.
5. Cheok, G. S., and Lew, H. S., "Model Precast Concrete Beam-to-Column Connections Subject to Cyclic Loading," PCI JOURNAL, V. 38, No. 4, July-August 1993, pp. 80-92.
6. ACI Innovative Task Group 1 and Collaborators, "Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary," ACI Proposed Provisional Standard (ACI T1.1-XX), American Concrete Institute, Farmington Hills, MI, undated.
7. Warcholik, G., and Priestley, M. J. N., "Structural Systems Research Project: High Strength Concrete Joints Tests – Series 3, Report No. TR-98/12," University of California, San Diego, July 1998.
8. Englekirk, R. E., *Seismic Design of Concrete and Precast Concrete Structures to a Performance Criterion,* John Wiley & Sons, Inc., New York, NY, 2003.
9. Nawy, E. G., *Prestressed Concrete: A Fundamental Approach,* Third Edition, Prentice Hall, Upper Saddle River, NJ, 2000.
10. Rahman, A. M., and Restrepo, J. I., "Earthquake Resistant Precast Concrete Buildings: Seismic Performance of Cantilever Walls Prestressed Using Unbonded Tendons," Civil Engineering Research Report No. 2000-5, University of Canterbury, Christchurch, New Zealand, August 2000.
11. Holden, T. J., "A Comparison of the Seismic Performance of Precast Wall Construction: Emulation and Hybrid Approaches," Civil Engineering Research Report No. 2001-4, University of Canterbury, Christchurch, New Zealand, April 2001.
12. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-99)," American Concrete Institute, Farmington Hills, MI, 1999.

APPENDIX A — NOTATION

a = depth of rectangular stress block
A_c = area of concrete
A_{ps} = area of prestressed reinforcement in tension zone
A_s = area of non-prestressed tension reinforcement
c = distance from extreme compression fiber to neutral axis
d = distance from extreme compression fiber to centroid of non-prestressed tension reinforcement
d' = distance from extreme compression fiber in centroid of compression reinforcement
f_{se} = effective stress in prestressed reinforcement (after allowance for all prestress losses)
f_{psp} = increment of post-yield stress in prestressing tendon
f_y = specified yield strength of non-prestressed reinforcement
F_{sn} = force in reinforcement at nominal strength
ℓ_{ps} = extent of reinforcement capable of sustaining post-yield strain
M_n = nominal moment strength
T_{pse} = effective prestressing force
T_y = tensile yield strength of ductile rod
δ_{ps} = post-yield elongation of rod or strand
θ = story drift or beam rotation
λ_o = overstrength factor

APPENDIX B — FRAME DESIGN

The design and analysis of the Hybrid and DDC Systems is fairly simple. Consider the Hybrid test assembly described in Fig. B1. These Hybrid beams were reinforced with nine $^1/_2$ in. (12.7 mm) diameter, 270 ksi (1862 MPa) strands (concentric unbonded post-tensioning) and three No. 6 (Grade 60) mild steel reinforcing bars bonded in the top and bottom of the beam. The grouted interface is 16 in. wide and 20 in. deep (406 x 610 mm). The nominal flexural strength is developed from the effective force in the strand and the yield strength of the mild reinforcing steel.

The following properties of materials are assumed:

$$A_{ps} = 1.38 \text{ sq in. } (890 \text{ mm}^2)$$

$$f_{se} = 162 \text{ ksi } (1117 \text{ MPa})$$

$$T_{se} = A_{ps} f_{se} = 223 \text{ kips } (1032 \text{ kN})$$

$$A_s = 1.32 \text{ sq in. } (852 \text{ mm}^2)$$

$$F_{sn} = A_s f_y = 79.2 \text{ kips } (352 \text{ kN})$$

$$a = \frac{T_{pse}}{0.85 f'_c b} = \frac{223}{0.85(5)(16)} = 3.3 \text{ in. } (84 \text{ mm})$$

$$M_n = T_{se}\left(\frac{h}{2} - \frac{a}{2}\right) + F_{sn}(d - d')$$

$$= 223\left(\frac{20}{2} - \frac{3.3}{2}\right) + 79.2(16.5)$$

$$= 1855 + 1307$$

$$= 3172 \text{ kip-in. } (358 \text{ kN-m})$$

The resultant column shear would be:

$$V_{col} = \frac{2M_n\left(\frac{\ell_b}{\ell_{bc}}\right)}{h_x} = \frac{2(3172)\left(\frac{72}{62}\right)}{117.5} = 62.7 \text{ kips } (279 \text{ kN})$$

Two important design objectives are to:

1. Provide a restoring force in the post-tensioning strand that exceeds that which is developed by the mild reinforcing steel. This will tend to restore the frame to its original pre-earthquake position ($\Delta \cong 0$).

$$M_{nps} > M_{ns}$$

$$1865 > 1307 \text{ kip-in. } (148 \text{ kN-m})$$

2. Furnish a post-tensioning force on the order of 700 to 1000 psi (4.83 to 6.90 MPa).

$$\frac{T_{se}}{A_c} = \frac{223}{16(20)} = 0.7 \text{ ksi } (4.83 \text{ MPa})$$

By achieving these two design objectives, it becomes easy to develop an initial frame beam size.

Post-yield strain states in the prestressing strands can also be easily checked. Assume that one wants to check the strand strain state at a subassembly drift of 2 percent.

Start by assuming a neutral axis depth of 5 in. (127 mm). The neutral axis depth must consider the additional strength provided by the strand elongation and, depending on the extent of the rotation, the strain hardening in the mild steel. Consider the rotation described in Fig. B2.

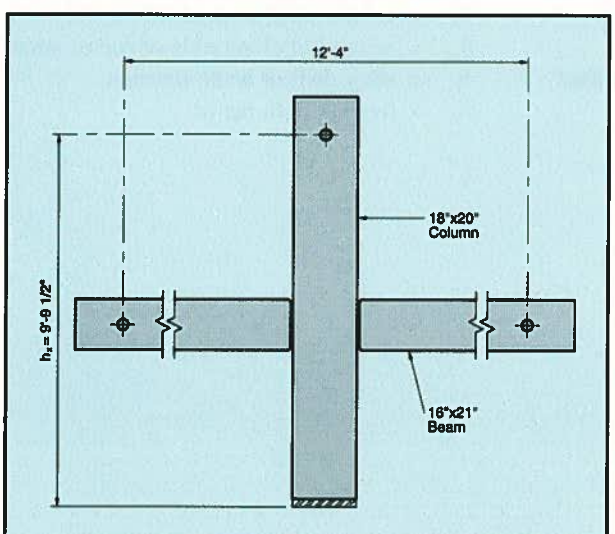


Fig. B1. Interior hybrid beam test assembly.

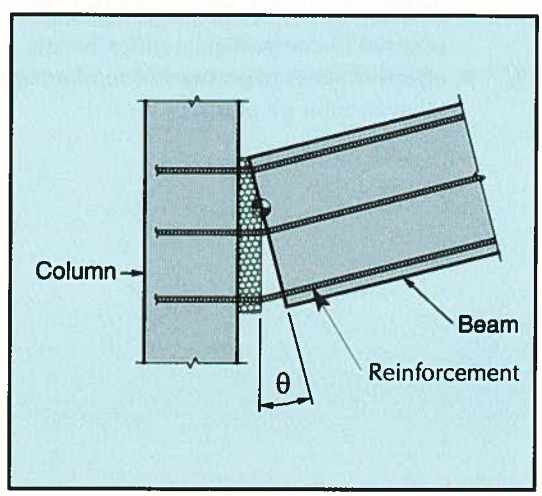


Fig. B2. Seismic induced movement (exaggerated) of precast frame components.

Fig. B3. DDC reinforced beam frame.

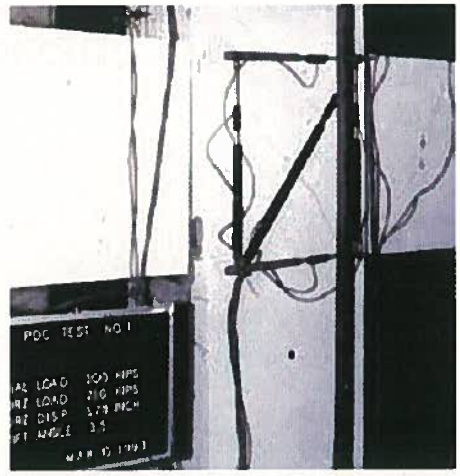


Fig. B4. Post-test condition of beam-to-column frame.

$$\theta = 0.02 \text{ radian}$$

$$\delta_{ps} = \theta\left(\frac{h}{2} - c\right)$$
$$= 0.02(10 - 5)$$
$$= 0.1 \text{ in. } (2.54 \text{ mm})$$

The total strand elongation developed in both beams is 0.2 in. (5.1 mm). The increase in strain (ε_{psp}) is:

$$\varepsilon_{psp} = \frac{2\delta_{ps}}{\ell_{ps}}$$
$$= \frac{0.2}{148}$$
$$= 0.00135 \text{ in. per in.}$$

The associated increase in the post-tensioning force is:

$$\Delta f_{ps} = \varepsilon_{psp} E_{ps}$$
$$= 0.00135(28,000)$$
$$= 37.8 \text{ ksi } (261 \text{ MPa})$$
$$f_{ps,2\%} = f_{pse} + \Delta f_{ps}$$
$$= 162 + 37.8$$
$$\cong 200 \text{ ksi } (1379 \text{ MPa})$$

This stress is significantly less than f_{psu} [230 ksi (91586 MPa)].

Obviously, space does not permit a full development of the design of the Hybrid System, but this will be available in a book which will be published by Wiley[8] in early 2003. In addition, a design example is contained in Reference 9.

The design of the DDC System is even simpler. Each ductile rod can develop a yield force of 141 kips (627 kN). The frame beam used in the Paramount building (see Fig. B3) would be designed as follows:

$$T_y = 141\, N$$

where N is the number of ductile rods, which in this case, is 4 (see Fig. B3).

$$M_a = T_y(d - d')$$

For the 36 in. (914 mm) deep frame beam of Fig. B3, the nominal flexural strength is:

$$M_n = 564\,(2.5)$$
$$= 1410 \text{ kip-ft } (1912 \text{ kN-m})$$

Since the plastic hinge region occurs within the column, the shear strength of the beam may include the contribution of the concrete because the beam is not damaged by post-yield rotations (see Fig. B4).

The strain state expected in the ductile rods of the beams of Fig. B3 at a drift of 2 percent would be estimated in the following manner:

$$a = \frac{\lambda_0 T_y - T_y}{0.85 f'_c\, b}$$
$$= \frac{(1.25)(564) - 564}{0.85(5)(20)}$$
$$= 1.68 \text{ in. } (43 \text{ mm})$$

Therefore, the neutral axis must be at a depth of about 6 in. (152 mm) in order to yield the compression rods:

$$\delta_{rod} = \theta_p(d - c)$$
$$= 0.02(33 - 6)$$
$$= 0.54 \text{ in. } (14 \text{ mm})$$

$$\varepsilon_{p,rod} = \frac{\delta_{rod}}{\ell_p}$$
$$= \frac{0.54}{9}$$
$$= 0.06 \text{ in. per in.}$$

This strain is well within the strain capabilities of the ductile rod.[8]

Note that the design of the beam-to-column joints for either the Hybrid or DDC Systems can follow the provisions in the ACI Building Code.[12]