Wet-joint techniques for the construction of precast concrete pipe rack structures in remote seismic zones

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- This paper presents a precast concrete solution for the construction of pipe rack structures for petrochemical plants in seismic zones.
- Experimental tests conducted on reduced-scale structures verified that the behavior of the prefabricated solution during an earthquake is comparable to, if not better than, that of the cast-in-place concrete solution.
- The proposed precast concrete solution is characterized by high strength and ductile behavior in the plastic range, and ease of in-plant fabrication and assembly on-site.

Oil and gas plants often require pipe rack structures to support the process pipelines. These structures are typically built with either steel or reinforced concrete elements or hybrid precast concrete beams and columns with steel bracing to provide the necessary stiffness and strength to resist lateral seismic loads.[1] While modularized pipe racks (which are the vast majority) are frequently made of steel, the structure type used for pipe racks is generally based on considerations related to cost; time for supply and erection of the structures; and ease of future maintenance, revamping, and expansions. In fire-prone areas, reinforced concrete elements are often selected for their intrinsic fire resistance, which reduces the cost and installation time related to fireproofing. With respect to cost and time, the choice between steel and reinforced concrete depends on the global and local markets and on the design solutions adopted.

Precast concrete construction offers several advantages—such as higher quality of components made in a manufacturing facility, lower costs, the ability to produce components in all weather conditions, and speed of construction—compared with traditional cast-in-place concrete techniques. However, cast-in-place structures traditionally had the advantage of continuous framing, which is better able to withstand the lateral loads produced by seismic activity. Prefabricated structures need to be specifically designed to guarantee monolithic behavior in seismic zones.

A number of solutions for precast concrete pipe racks have been proposed. Monolithic behavior has been achieved using cast-in-place concrete joints to connect the precast concrete beams and columns, with mechanical connectors between the precast concrete beams and columns (or with monolithic precast concrete frames for small pipe racks). Choosing the best technology for a precast concrete system is important, with the goal of finding a solution that can provide the required performance in terms of load-bearing capacity and ductility while minimizing construction time and cost.

This study presents a precast concrete solution developed for the pipe rack structures of a new petrochemical plant under construction in Central America (**Fig. 1**). The adopted solution was chosen for its high strength and ductility in the plastic range, its simple in-plant fabrication, and easy assembly on-site. Experimental tests conducted on reduced-scale structures confirmed that the structural behavior of the prefabricated solution during an earthquake is comparable to, if not better than, that of the corresponding cast-in-place concrete solution.

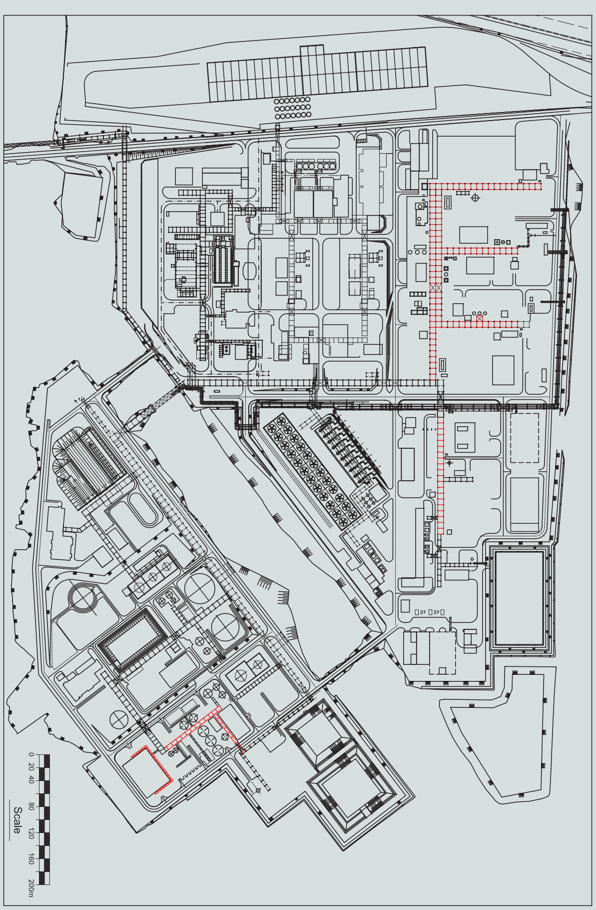


Figure 1. Site plan for a new petrochemical complex in Central America with precast concrete pipe racks in red. Note: 1 m = 3.28 ft.

Beam-to-column joints in precast reinforced concrete structures

Since the early 1990s, research has been conducted on beam-to-column joints in precast concrete structures. The importance of connection detailing for precast concrete structures subjected to seismic loads became apparent and various technical solutions have been proposed and tested.

The joint research project PRESSS (Precast Seismic Structural Systems) was conducted by researchers from the United States and Japan on the seismic design and performance of precast concrete structural systems.[2,3] United States researchers focused on ductile connections capable of protecting the precast concrete elements using the capacity design, whereas the Japanese program concentrated on the strong connection approach.

Restrepo et al.[4] tested different types of moment-resisting connections made with cast-in-place concrete located at the beam midspan or at the beam-to-column joint region. Their experimental results showed that connection details can be successfully designed and constructed to emulate cast-in-place concrete construction.

Priestley and MacRae[5] tested ungrouted, post-tensioned, precast concrete beam-to-column joint subassemblies under cyclic reversals of inelastic displacement to determine their seismic response. A good performance was recorded during the experimental tests, with only minor cosmetic damage detected up to drift ratios of 3%. Energy absorption of the hysteretic response, though small, was larger than expected. A low residual drift was observed after a severe earthquake.

Two full-scale beam-to-column connections designed according to the strong column–weak beam concept were tested by Alcocer et al.[6] Conventional mild steel reinforcing bars, rather than welding or special bolts, were used to achieve beam continuity. Test results showed that the performance of both beam-to-column connections was roughly 80% of that expected from monolithic reinforced concrete construction, with a ductile behavior due to hoop yielding.

Korkmaz and Tankut[7] tested 1/2.5-scale beam-to-beam connection subassemblies composed of a middle precast concrete beam placed between two cantilever beams connected to the columns under reversed cyclic loading. Lap splicing and welding were used to connect the top and the bottom reinforcement, respectively, using cast-in-place concrete to complete the connection. The results of the experimental tests allowed modification of the original connection detail for seismic use.

A similar solution was proposed by Ong et al.,[8] who used the "design for disassembly" method to reuse the structural components after the structure was decommissioned, instead of demolishing and recycling the resulting debris.

Parastesh et al.[9] tested a new ductile moment-resisting beam-to-column connection achieved by a discontinuity in the column filled with the cast-in-place concrete. The authors consider this solution capable of providing good structural integrity in the connections with reduced construction time, no need for formwork or welding, and minimized cast-in-place concrete volume.

A recently completed projected, evaluated the influence of various parameters (for example, the type of mechanical connections and the presence of shear walls along with the framed structure) on the seismic behavior of a full-scale three-story precast concrete building.[10]

The current types of connections between precast concrete beams and columns can be classified as dry connections, wet connections, and hybrid connections. Vidjeapriya and Jaya[11] tested a dry connection made with steel elements and bolts. The authors conducted tests on two types of simple mechanical 1/3-scale concrete beam-to-column connections realized with a cleat angle with one or two stiffeners, subjected to reverse cyclic loading. The authors observed that the ultimate load-carrying capacity of the monolithic specimen was superior to that of the precast concrete specimens, though the latter was found to behave satisfactorily in terms of energy dissipation and ductility.

Wet connections typically comprise reinforcing bar splices and cast-in-place concrete. In some cases the use of steel fibers can contribute to the development of ductile moment-resisting connections designed to act as a plastic hinge during earthquakes.[12] A high-performance fiber-reinforced cement composite matrix was used to develop a high-energy-absorbing joint for precast, prestressed concrete structures in seismic zones, reducing the amount of transverse reinforcement in the connection by using steel fibers in the connection matrix.[13] Ultra-high-performance fiber-reinforced concretes were also used in conjunction with short reinforcement splice lengths to develop continuity connections between precast concrete elements, achieving safe in-place erection and assembly processes, reducing construction time, and avoiding the use of complex reinforcing details while maintaining high quality.[14]

Hybrid connections with mechanical fasteners and cast-in-place concrete were tested by Choi et al.[15] and Ong et al.[8] Cheok et al.[16] tested a hybrid connection made with mild reinforcing and posttensioning steel, where the mild reinforcing steel was used to dissipate energy by yielding and the posttensioning steel was used to provide shear resistance through friction developed at the beam-to-column joint.

Proposed beam-to-column connections

Figure 2 shows several solutions developed for beam-to-column connections in precast concrete moment-resisting frames of pipe racks.

Solutions involving wet joints and lap splices require scaffolding with the associated costs and time implications. In particular, a precast concrete pipe rack can be erected in a couple of months and the time required to complete the wet joints using scaffolding can be similar. Solutions that make use of connectors can reduce the erection time but necessitate skilled subcontractors to meet the required tolerances for the connectors. Many petrochemical complexes are

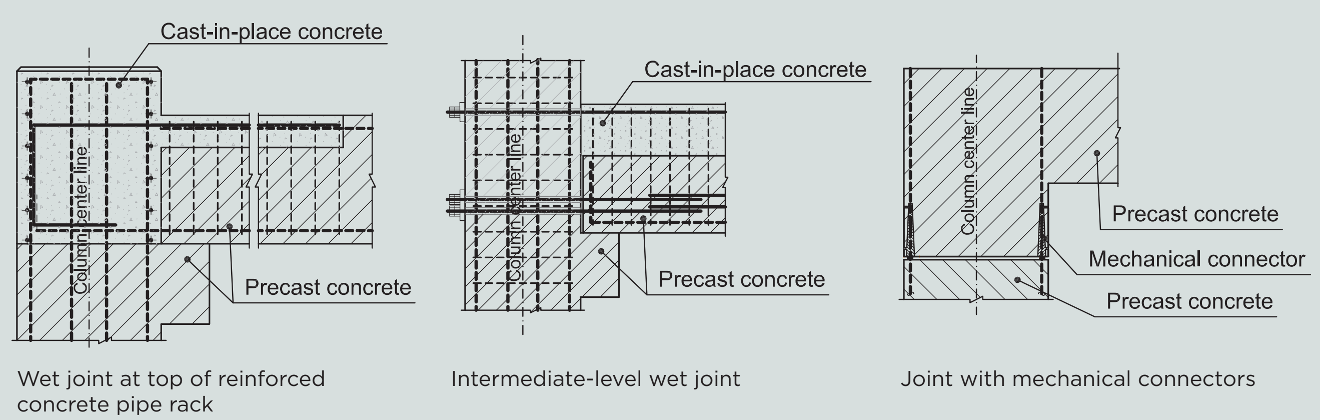


Figure 2. Beam-to-column connections developed previously.

built in countries with a low availability of skilled workers. Simplifying the solution thus avoids the necessity of using nonlocal workers and the related social and economic implications.

The proposed beam-to-column connection was developed with the following concepts in mind:

- can be safely and quickly executed
- does not use scaffolding
- uses traditional techniques (without connectors) that allow for the usual concrete construction tolerances
- can be put in place by low-skilled workers

Using traditional techniques pushed the design toward a wet joint solution, and the avoiding scaffolding required the use of a minimum volume of cast-in-place concrete and the installation of limited formwork.

The force transfer between the reinforcement in the precast concrete columns and beams inside the wet joints is usually accomplished with lap splicing. For the usual diameters of reinforcing bars in petrochemical applications, the lap splice can easily reach values up to 2 to 2.5 m (6.6 to 8.2 ft).

To reduce the volume of concrete to be cast in place as much as possible, the ends of the reinforcing bars were hooked, thereby limiting the required length of the splice. Breccolotti et al.[17] provides further details on this type of connection.

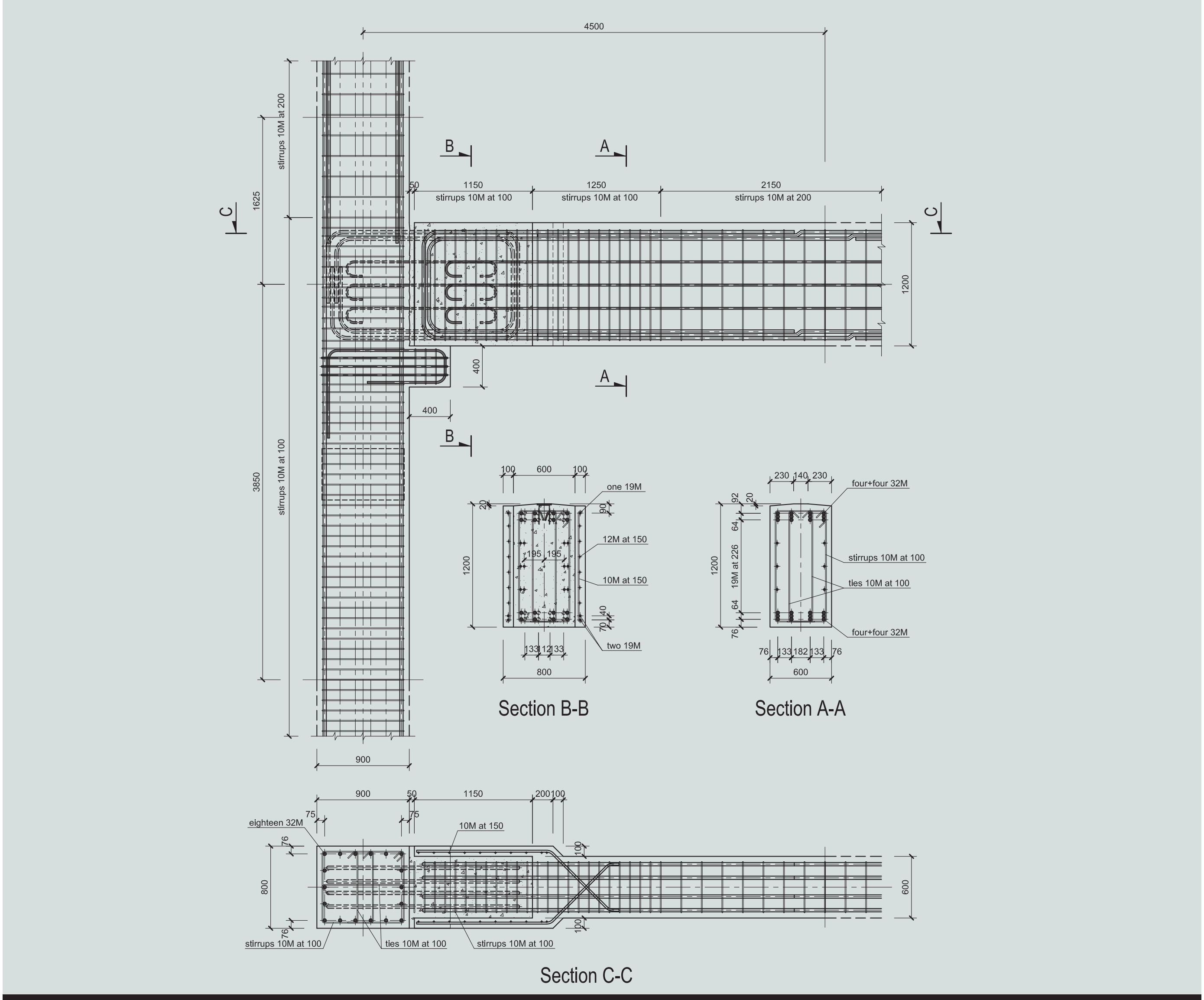


Figure 3. Reinforcement of the precast concrete beam-to-column joint. Note: All measurements are in millimeters. 10M = no. 3; 12M = no. 4; 19M = no. 6; 32M = no. 10; 1 mm = 0.0394 in.

To avoid handling formwork on-site, the precast concrete beam was shaped so that the lateral formwork of the joints was made of reinforced concrete panels cast together with the structural beam, which formed two pockets at its extremities. The precast concrete panels were designed to include a cross section of the joint that was filled with fresh concrete with the same dimensions as the beam cross section. Hook-shaped reinforcing bars protruding from the beam and stirrups grouped at the internal face of the precast concrete piece were contained in the joints. The reinforced concrete panels also serve as a temporary support for the beam when it is positioned on the two columns to be connected. The latter have short cantilevers that provide support for the precast concrete beam. Protruding hook-shaped steel bars, designed to take the forces coming from the longitudinal reinforcing bars of the beam, complete the design of the precast concrete columns.

Once the beam was in its final position, the stirrups were moved to their final position and the joints were filled with self-consolidating fiber-reinforced concrete. These simple operations can be executed using a aerial-lift. **Figure 3** shows the resulting design.

Structural design

One of the key factors for the success of projects involving the design and the construction of a petrochemical complex is the quick availability of concrete pipe racks able to support large pipes (up to 1320 mm [52 in.] in diameter) and to resist high lateral earthquake forces (up to 70% of vertical loads). In the present case, the structural design was conducted according to the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)*,[18] while the earthquake loads on the structures and foundations were evaluated in accordance with the *CFE Manual de Diseño por Sismo*.[19] This standard specifies that for pipe racks and process structures, the complete seismic load acting in one direction should be combined with 30% of the seismic load acting in the orthogonal direction with the most unfavorable combination of signs. The action of the seismic vertical component may instead be neglected. The CFE manual classifies structures according to their importance. The supports of equipment and piping containing toxic or flammable materials or considered vital for emergency situations are classified as group A. All of the other structures are classified as group B.

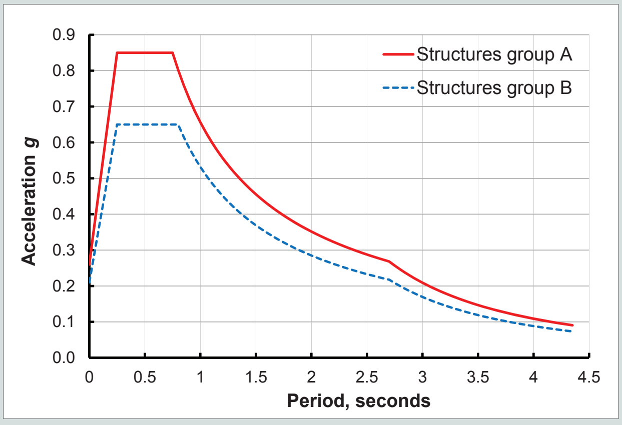


Figure 4. Elastic response spectra. Note: g = acceleration of gravity = 32.2 ft/s^2 = 9.81 m/s^2.

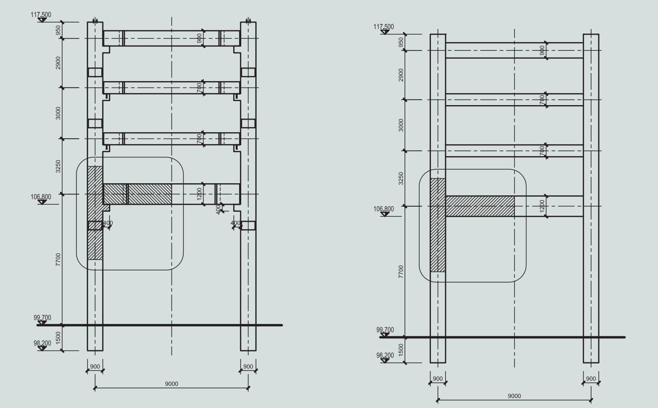
Tall structures and structures having stiffness, weight, or geometric irregularities shall be analyzed using dynamic lateral force procedures, including appropriate scaling of the results. In this case, the obtained dynamic base shear must be equal to at least 80% of the corresponding base shear obtained using static procedures.

The effective seismic weight of the process structures, pipe racks, shelters, and minor structures includes the total dead load and the operating weight of the permanent equipment. Floor live loads were disregarded, with the exception of storage areas, for which 25% of the live load was considered.

Figure 4 shows the elastic response spectrums used in the design of structures having a structural damping of 5%. The design response spectrums were obtained from the elastic response spectrums with a response reduction factor equal to three, as specified by the local standard for reinforced concrete structures.[20] Special attention was paid to the in-service conditions to limit the horizontal displacement due to wind action and to frequent low-intensity earthquakes.

Experimental investigations

To evaluate the structural behavior of the proposed beam-to-column connection, experimental tests on 1/3-scale models were designed and conducted. The structural performance of the proposed joint was also assessed by comparison with the behavior of a completely cast-in-place concrete joint. The test modules, that is, the laboratory specimens representing the characteristics of a typical configuration of intersecting beams and columns, were defined according to the provision of ACI's *Acceptance Criteria for Moment Frames Based on Structural Testing (ACI 374.1-05)*[21] for the most stressed connection of the moment frame. **Figure 5** shows a typical pipe rack for the precast concrete solution and a typical cast-in-place concrete frame. **Table 1** gives the mixture designs of the concrete used for the precast concrete elements and for the wet-joint cast. **Table 2** lists the effective concrete compressive strength f_c' obtained for these mixture designs by compressive tests. The design of the reduced-scale models was conducted using the theory of similitude with the scale factors in **Table 3** for the various mechanical variables.



Precast concrete | Cast-in-place concrete

Figure 5. Typical pipe rack for the precast concrete solution and cast-in-place concrete frame. Note: All measurements are in millimeters. 1 mm = 0.0394 in.

Table 1. Mixture designs for the cast-in-place, precast, and fiber-reinforced concretes

Material	Cast-in-place and precast concrete	Fiber-reinforced concrete
Portland cement type CEM I 52,5 R, kg/m^3	380	640
Fine aggregate, kg/m^3	940	583
Coarse aggregate, kg/m^3	850	800
Water, L/m^3	150	192
Hyperplasticizer, L/m^3	4.0	6.4
Fibers, kg/m^3	n/a	39

Note: n/a = not applicable. 1 kg/m^3 = 1.686 lb/yd^3; 1 L/m^3 = 0.026 oz/yd^3.

Table 2. Experimental concrete compressive strengths

Material	Sample	Concrete compressive strength, MPa	
		1 day	125 days
Cast-in-place and precast concrete	1	8.65	33.30
	2	6.36	30.40
	Average	7.50	31.85
Fiber-reinforced concrete	1	16.2	68.70
	2	17.8	70.20
	Average	17.0	69.45

Note: 1 MPa = 0.145 ksi.

Cast-in-place concrete frame

The cast-in-place reinforced concrete frame was obtained by assuming the same materials, dimensions, and reinforcement ratios in both the longitudinal and transversal directions, as the proposed precast concrete frame. **Figure 6** shows the structural details of the beam-to-column connection obtained with this assumption.

Experimental setup

After scaling the precast concrete and the cast-in-place concrete joints with the factors listed in Table 3, an experimental setup was developed (**Fig. 7**). It included a hydraulic jack placed between the reaction wall and the upper part of the column, which imposed the horizontal drift on the tested joint with a pinned connection.

The column base was restrained from horizontal displacement by a stiff steel frame anchored to the rigid reinforced concrete wall. A pinned connection allowed rotation to occur. The column was supported by a steel cylinder that provided the vertical reaction force without notable horizontal components. The beam was linked by a pinned connection to a steel frame fixed to the floor. The steel frame restrained the beam in the vertical direction, allowing simultaneous horizontal movement of the beam itself. No notable horizontal restraining force was thus applied to the end of the beam.

A second hydraulic ram actuator placed on the top of the column was used to apply a suitable compressive force to the column. The value of this force corresponds to the axial load induced in the column by the permanent loads in the overlying portion of the structure of the pipe rack reduced by a scale factor of nine to account for the scaling of the specimen. The reaction exerted by the jack was transmitted to the ground by two threaded steel rods. **Figure 8** shows the main dimensions of the specimens and of the experimental setup.

Table 3. Scale factors

Variables	Real	Model
Length L	L	$L/3$
Stress σ	σ	σ
Force F	F	$F/3^2$
Area of longitudinal reinforcement A_L	A_L	$A_L/3^2$
Area of transversal reinforcement per unit length A_T	A_T	$A_T/3$
Moment M	M	$M/3^3$

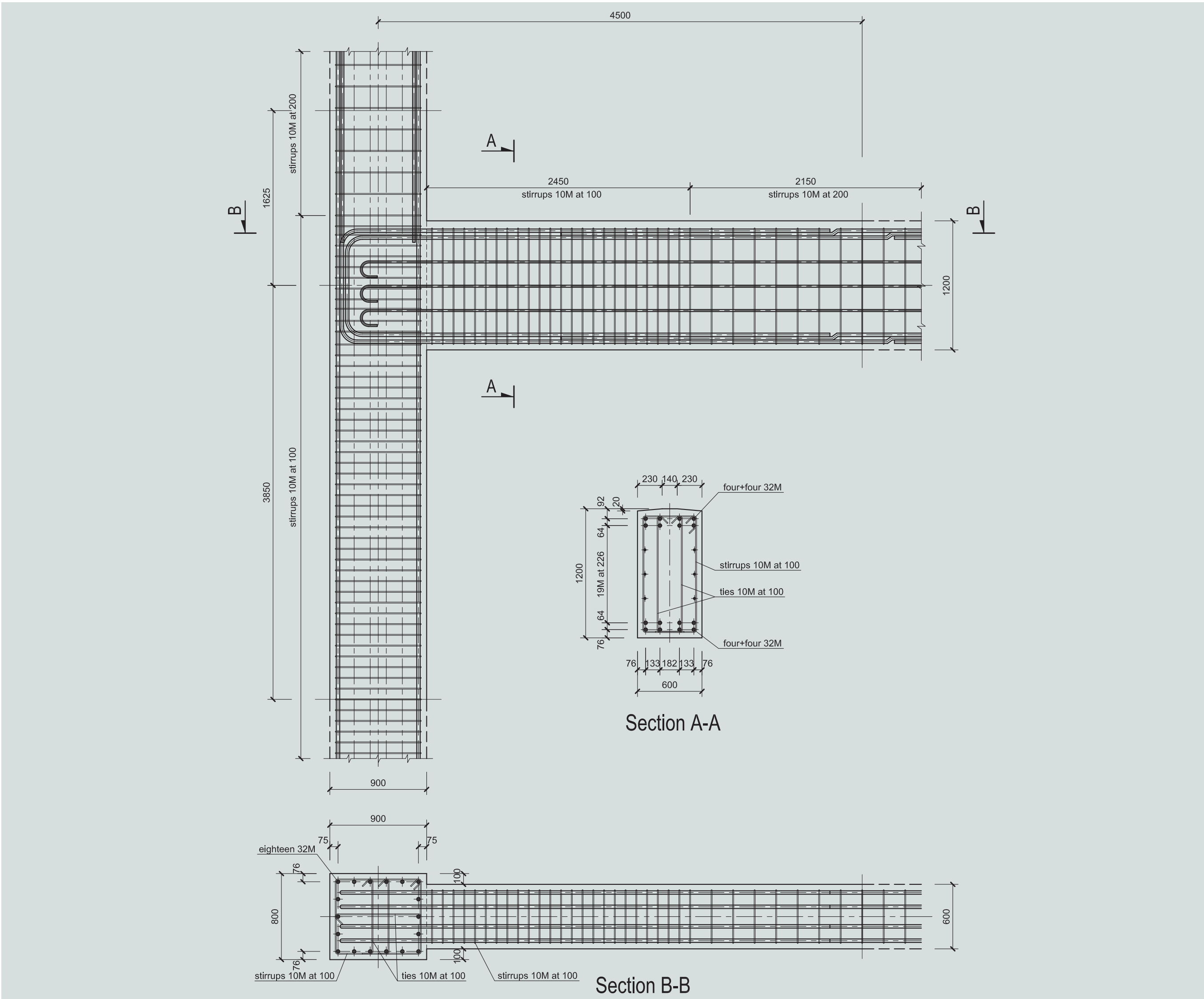


Figure 6. Reinforcement of the reference cast-in-place concrete beam-to-column joint. Note: All measurements are in millimeters. 10M = no. 3; 19M = no. 6; 32M = no. 10; 1 mm = 0.0394 in.

Figure 7. Reduced-scale experimental setup of the precast concrete solution.

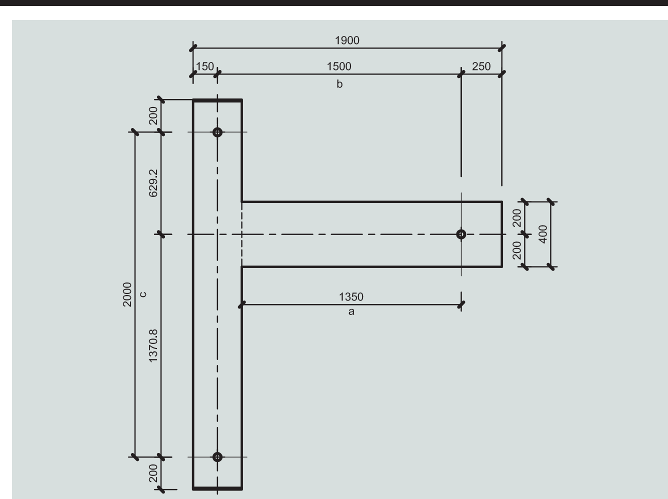


Figure 8. Dimensions and scheme of the reduced-scale specimens. Note: All measurements are in millimeters. 1 mm = 0.0394 in.

Sensors

Several sensors were applied to the tested joints to monitor their structural behavior and evaluate the stresses in the concrete and reinforcing steel. The applied drift was obtained as the difference between the readings of two displacement sensors placed on the bottom and top hinges of the column while a pressure transducer was used to measure the pressure in the hydraulic jack. To measure the concrete strain, vibrating wire strain gauges were embedded in the concrete in the upper and lower areas of the beam section near the joint, both in the cast-in-place and precast concrete joints (**Fig. 9**). These sensors were placed just outside the critical region for both the precast and cast-in-place concrete beams (**Fig. 10**) to avoid negative effects that might occur to the sensors had they been placed in the critical region. Similarly, vibrating wire strain gauges were arc welded to the lower and upper steel bars of the beam just outside the critical region and to the steel reinforcing bars inside the column in the precast concrete joint to verify the actual transmission of stress from the reinforcing bars of the beam to those integral with the precast concrete column. The choice to use vibrating wire strain sensors (rather than resistive strain gauges) was made taking the following critical factors into account:

- the need for waterproof, robust, reliable instrumentation able to withstand mechanical and thermal stresses during concrete casting (including the dense reinforcement grid close to the node)
- the need to measure the deformation of the concrete over a significantly long distance compared with the size of the aggregates
- the need for absolute immunity to electromagnetic interference caused by equipment inside the laboratory
- the need to filter the local thermal drift during long-term tests by equipping each vibrating wire strain gauge with an internal thermistor

Additional potentiometric linear variable displacement transducers were applied to the joint to measure its overall deformation. One transducer was located on the upper part of the beam section to detect the horizontal relative displacements between the upper outer layer of concrete and the outer concrete of the column. Similarly, another transducer was applied to the bottom part of the beam section. Finally, two transducers were placed in an X-shaped configuration on the lateral concrete surface of the beam and were fixed to the lateral surface of the column section.

Signals from the sensors were recorded using a double system of measurement with two data acquisition units:

- One unit had four 4.8 kHz carrier-frequency channels, ±25,000 digits, with a sampling rate up to 9600 S/s/ch (samples for second and for each channel), and full- and half-bridge strain gauges. This unit was used to acquire the data of the strain gauge sensor, two pressure transducers, and one load cell.
- The other unit had 10 to 30 V wide-range input channels, 15-bit resolution, with a sampling rate up to 25 Hz (70 Hz)/channel. This unit was used to acquire the data of six vibrating wire strain gauges, six thermistors, and four potentiometric linear variable displacement transducers.

The two data-acquisition units were synchronized by a digital line and configured for a data scan rate of 1 Hz.

Test program

The two joint specimens were subjected to a sequence of displacement-controlled cycles according to the provisions of ACI 374.1-05. The drift sequence (**Fig. 11**) was established in accordance with the following rules:

- The initial drift ratio must be within the essentially linear elastic response range.

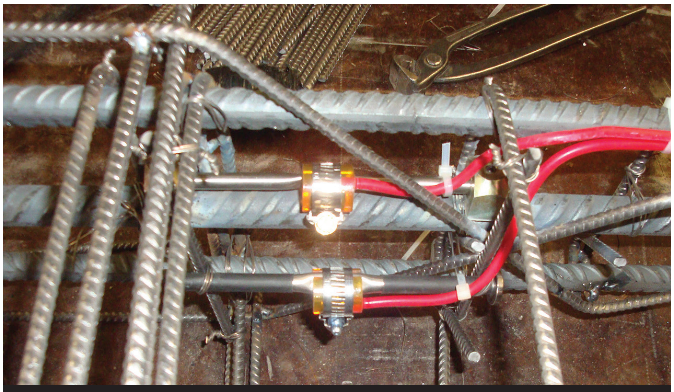


Figure 9. Vibrating wire strain gauge for concrete strain and steel strain (welded).

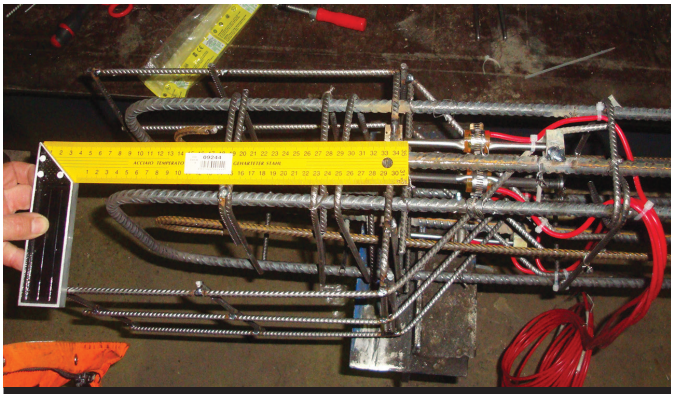


Figure 10. Precast concrete beam equipped with vibrating wire strain gauges.

- Subsequent drift ratios must not be less than one and one-quarter times and not more than one and one-half times the previous drift ratio.

- Three fully reversed cycles must be applied for each drift ratio value.

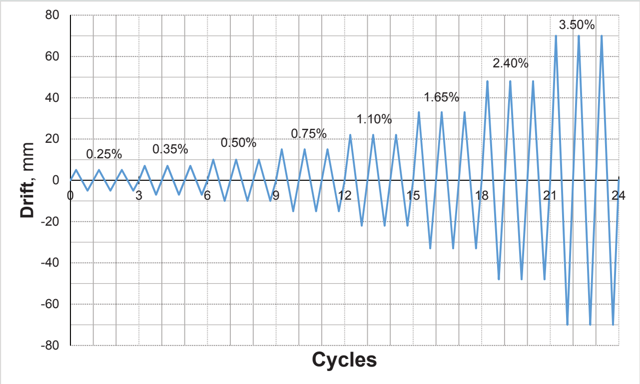


Figure 11. Test sequence of the imposed lateral displacement. Note: 1 mm = 0.0394 in.

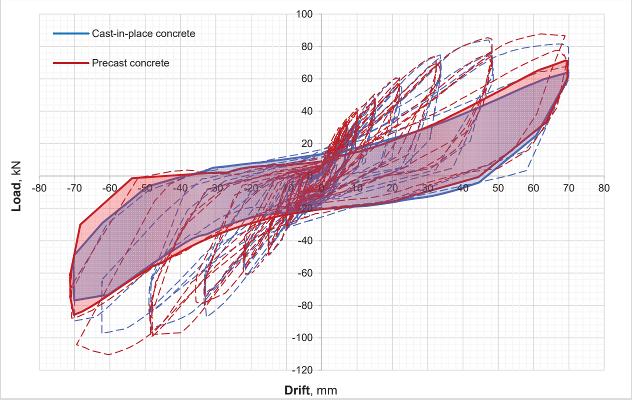


Figure 12. Drift response versus force of the cast-in-place (blue curve) and the precast concrete (red curve) specimens. Note: solid lines = third cycles at a drift value of 0.035; dashed lines = previous cycles. 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

The tests were conducted with a gradually increasing drift ratio until it reached a value of ±70 mm (2.8 in.), corresponding to a drift ratio of 3.5% for the scaled specimen.

Comparison between cast-in-place and precast concrete solutions

Figure 12 shows the overall behavior of the two joints. The graph contains the entire load-drift cycling for the cast-in-place and precast concrete specimens. The overall behavior of the two joint types is similar to the precast concrete solution, which seems to possess slightly greater stiffness and strength. Conversely, no detailed comparison can be made for the crack pattern in the beam critical region. It is, in fact, hidden by the lateral concrete panels in the precast concrete joint that do not allow the main cracks to reach the lateral external surface.

The response of each sample complied with the acceptance criteria of ACI 374.1-05. In fact, for cycling at the 0.035 drift level, the characteristics of the third complete cycle satisfied the following conditions:

- The peak force for both loading directions is greater than $0.75E_{max}$ (where E_{max} is the maximum lateral resistance) for the same loading direction.

- The relative energy dissipation is greater than 1/8.

- The secant stiffness from a drift ratio of -0.035 to a drift ratio of +0.035 is greater than 0.05 times the stiffness for the initial drift ratio.

Tables 4 and **5** summarize the actual values of these figures for the cast-in-place concrete specimen and for the precast concrete specimen, respectively.

Furthermore, the test modules attained a lateral resistance greater than the test module nominal lateral resistance E_n before their drift ratio exceeded the allowable story drift limitation according to the relevant standards.[22,23] Equations (1) and (2) calculate the nominal resistance of the full-

Table 4. Reference and actual values for the acceptance criteria of cast-in-place concrete specimen

Variables	Third cycle	Reference	Ratio
Positive peak force, kN	64.0	85.1	0.752
Negative peak force, kN	-77.1	-96.3	0.801
Energy dissipation, kJ	5.63	15.03	0.374
Secant stiffness, kN/mm	1.008	3.247	0.310

Note: 1 kJ = 738 ft-lb; 1 kN = 0.225 kip; 1 kN/mm = 5.710 kip/in.

Table 5. Reference and actual values for the acceptance criteria of precast concrete specimen

Variables	Third cycle	Reference	Ratio
Positive peak force, kN	71.3	87.8	0.812
Negative peak force, kN	-85.6	-110.4	0.775
Energy dissipation, kJ	5.76	17.98	0.320
Secant stiffness, kN/mm	1.129	4.014	0.281

Note: 1 kJ = 738 ft-lb; 1 kN = 0.225 kip; 1 kN/mm = 5.710 kip/in.

sized beam critical section and the corresponding strength of the scaled specimen, respectively.

$$M_n = \left[\beta_1 f_c^{'} b y^2 \left(1-0.5\beta_1\right) + \frac{A_s^{'} E_s \varepsilon_{cu} \left(y-c\right)^2}{y} + f_y A_s \left(h-c-y\right)\right]$$

$$= 2946 \text{ kN-m (2173 kip-ft)} \quad (1)$$

where

M_n = nominal moment capacity

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

b = beam width

y = neutral axis depth

$A_s^{'}$ = area of compression reinforcement

E_s = reinforcement modulus of elasticity

ε_{cu} = maximum concrete strain

c = cover to the centroid of the longitudinal bars

f_y = reinforcement yield strength

A_s = area of tension reinforcement

h = beam height

$$M_{n,red} = \frac{M_n}{SF^3} = \frac{2946}{3^3} = 109.1 \text{ kN-m (80.47 kip-ft)} \quad (2)$$

where

$M_{n,red}$ = nominal moment capacity of the scaled specimen

SF = scale factor of the test specimens

The lateral resistance E_n can be evaluated as the horizontal force that produces a bending moment on the critical section of the beam equal to $M_{n,red}$. It can be calculated using Eq. (3) according to the scheme in Fig. 8.

$$E_n = \left(\frac{M_{n,red}}{a}\right)\left(\frac{b}{c}\right) = \left(\frac{109.1}{1.35}\right)\left(\frac{1.50}{2.00}\right) \quad (3)$$

$$= 60.6 \text{ kN (13.6 kip)}$$

The allowable story drift limitation for the structure under investigation can be conservatively assumed to be equal to that of buildings in Risk Category III with interior walls, partitions, ceilings, and exterior wall systems designed to accommodate the story drifts.

$$\delta = 0.20 h_{sx} = 0.020 \times 2000 = 40 \text{ mm (1.6 in.)}$$

where

δ = allowable story drift

h_{sx} = interstory height

This value corresponds to a horizontal load in the range of 80 to 100 kN (18 to 22 kip) that is greater than E_n.

Finally, the maximum lateral resistance E_{max} recorded in the test did not exceed λE_n, where λ is the specified overstrength factor for the tested structure. The National Earthquake Hazards Reduction Program[24] assumes an overstrength factor λ of 3 for ordinary reinforced concrete moment frames.

E_{max} = 110 kN-m < λE_n = 3 × 60.6 kN-m = 181.8 kN-m (134.1 kip-ft)

Availability of precast concrete pipe rack structures

The ability to independently erect any single element (beam or column) of the precast concrete structure greatly simplified the construction of the pipe racks. Moreover, reduction of the joint length achieved with the hoop splice allowed the casting operations of column-to-beam connections to be done quickly and safely using only an aerial-lift. This dramatically reduced the time needed to complete the erection of the structure because no scaffolding was required. **Figure 13** shows the difference in the time required for erection between a typical cast-in-place concrete joint as described previously and the innovative cast-in-place concrete joint based on data from past projects of the same size. The graph shows that the construction time was reduced by roughly 50%. **Figure 14** shows the construction process with the erection of the columns and the placing of the precast concrete beams.

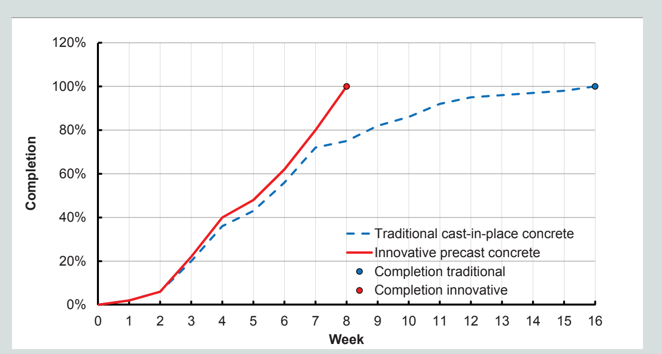


Figure 13. Construction time comparison between traditional and innovative cast-in-place concrete joints.

Conclusion

This paper presents a technique to use wet beam-to-column connections for precast concrete frames. The technique relies on the prefabrication of beams and columns with protruding hook-shaped bars that are connected in place with fiber-reinforced concrete. Experimental tests on reduced-scale specimens verified the acceptance criteria of ACI 374.1-05. The authors compared the structural behavior of a beam-to-column subassembly created with this technique with that of an equivalent cast-in-place concrete beam-to-column joint. The results of these tests met the acceptance criteria and showed that the two solutions exhibited similar structural behavior. The proposed solution achieved slightly greater strength and stiffness than the cast-in-place concrete solution, without relevant modifications to the joint ductility. This technique allows the economical and reliable applicability of monolithic reinforced concrete frames that can be achieved with this technique.

Acknowledgments

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References

1. Vaquero, S. F., D. R. Correa, and S. F. Wolkomirski. 2013. "Precast Concrete, Steel-Braced, Hybrid Pipe Rack Structures." *PCI Journal* 58 (4): 55–67.

Figure 14. Erection of the precast concrete columns and beams.

2. Priestley, M. 1991. "Overview of PRESSS Research Program." *PCI Journal* 36 (4): 50–57.

3. Priestley, M., S. Sritharan, J. Conley, and S. Pampanin. 1991. "Preliminary Results and Conclusions from the PRESSS Five-Story Precast Concrete Test Building." *PCI Journal* 44 (6): 42–67.

4. Restrepo, J. I., R. Park, and A. H. Buchanan. 1995. "Test on Connections of Earthquake Resisting Precast Reinforced Concrete Perimeter Frames of Buildings." *PCI Journal* 40 (4): 44–61.

5. Priestley, M., and G. MacRae. 1996. "Seismic Tests of Precast Beam-to-Column Joint Subassemblages with Unbonded Tendons." *PCI Journal* 41 (1): 64–80.

6. Alcocer, S., R. Carranza, D. Perez-Navarrete, and R. Martinez. 2002. "Seismic Tests of Beam-to-Column Connections in a Precast Concrete Frame." *PCI Journal* 47 (3): 70–89.

7. Korkmaz, H. H., and T. Tankut. 2005. "Performance of a Precast Concrete Beam-to-Beam Connection Subject to Reversed Cyclic Loading." *Engineering Structures* 27 (9): 1392–1407.

8. Ong, K., Z. S. Lin, L. R. Chandra, C. T. Tam, and S. D. Pang. 2013. "Experimental Investigation of a DfD Moment-Resisting Beam-Column Connection." *Engineering Structures* 56: 1676–1683.

9. Parastesh, H., I. Hajirasouliha, and R. Ramezani. 2014. "A New Ductile Moment-Resisting Connection for Precast Concrete Frames in Seismic Regions: An Experimental Investigation." *Engineering Structures* 70: 144–157.

10. Bournas, D., P. Negro, F. Molina, B. Viaccoz, and G. Magonette. 2012. *Pseudodynamic Testing of the SAFECAST 3-Storey Precast Concrete Building*. JRC (Joint Research Centre) technical report. Luxembourg: European Commission JRC.

11. Vidjeapriya, R., and K. Jaya. 2013. "Experimental Study on Two Simple Mechanical Precast Beam-Column Connections under Reverse Cyclic Loading." *Journal of Performance of Constructed Facilities* 27 (4): 402–414.

12. Naaman, A. E., J. K. Wight, and H. Abdou. 1987. "SIFCON Connections for Seismic Resistant Frames." *Concrete International* 9 (11): 34–39.

13. Vasconez, R., A. Naaman, and J. Wight. “Behavior of HPFRC Connections for Precast Concrete Frames under Reversed Cyclic Loading.” *PCI Journal* 43 (6): 58–71.

14. Maya, L., C. Zanuy, L. Albajar, C. Lopez, and J. Portabella. “Experimental Assessment of Connections for Precast Concrete Frames Using Ultra High Performance Fibre Reinforced Concrete.” *Construction and Building Materials* 48: 173–186.

15. Choi, H.-K., Y.-C. Choi, and C.-S. Choi. 2013. “Development and Testing of Precast Concrete Beam-to-Column Connections.” *Engineering Structures* 56: 1820–1835.

16. Cheok, G., W. Stone, and S. Kunnath. 1998. “Seismic Response of Precast Concrete Frames with Hybrid Connections.” *ACI Structural Journal* 95 (5): 527–538.

17. Breccolotti, M., S. Gentile, M. Tommasini, A. L. Materazzi, M. F. Bonfigli, B. Pasqualini, V. Colone, and M. Gianesini. 2016. “Beam-Column Joints in Continuous RC Frames: Comparison between Cast-in-Situ and Precast Solutions.” *Engineering Structures* 127: 129–144.

18. ACI (American Concrete Institute) Committee 318. 2008. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)*. Farmington Hills, MI: ACI.

19. Comisión Federal de Electricidad. 2008. *Manual de Diseño de Obras Civiles: Diseño por Sismo*. Mexico: Comisión Federal de Electricidad.

20. Administración Pública del Distrito Federal. 2004. “Normas Técnicas Complementarias para Diseño y Construcción de Estructuras de Concreto.” *Gaceta Oficial del Distrito Federal* I (103-bis): 88–194.

21. ACI Committee 374. 2001. *Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary*. (ACI 374.1-05). Farmington Hills, MI: ACI.

22. International Code Council. 2009. *International Building Code 2009*. Whittier, CA: International Conference of Building Officials.

23. ASCE (American Society of Civil Engineers). 2010. *Minimum Design Loads for Buildings and Other Structures*. ASCE/SEI 7-10. Reston, VA: ASCE.

24. Building Seismic Safety Council. 2004. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions and Part 2: Commentary (FEMA 450)*. Washington, DC: FEMA (Federal Emergency Management Agency).

Notation

A_L = area of longitudinal reinforcement

A_s = area of tension reinforcement

A_s' = area of compression reinforcement

A_T = area of transverse reinforcement per unit length

b = beam width

c = cover to the centroid of the longitudinal bars

E_{max} = maximum lateral resistance

E_n = test module nominal lateral resistance

E_s = reinforcement modulus of elasticity

f_c' = 28-day concrete cylinder characteristic compressive strength

f_y = reinforcement yield strength

F = force

g = acceleration due to gravity

h = beam height

h_{sx} = interstory height

L = length

M = moment

M_n = nominal moment capacity

$M_{n,red}$ = nominal moment capacity of the scaled specimen

SF = scale factor

y = neutral axis depth

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

δ = allowable story drift

ε_{cu} = maximum concrete strain

λ = overstrength factor

σ = stress

About the authors

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Abstract

This paper presents a precast concrete solution developed for the construction of pipe rack structures in petrochemical plants. It is based on the prefabrication of concrete columns and beams connected on-site by a wet joint. The solution is characterized by high strength and ductile behavior in the plastic range, and ease of in-plant fabrication and on-site assembly. Experimental tests conducted on reduced-scale structures verified that the behavior of the prefabricated solution during an earthquake is comparable to, if not better than, that of the corresponding cast-in-place concrete solution awhile fulfilling the requirement of the American Concrete Institute's (ACI's) *Acceptance Criteria for Moment Frames Based on Structural Testing* (ACI 374.1-05). The proposed precast concrete solution economically mimics the behavior of monolithic reinforced concrete frames. A case study of several pipe rack structures in a remote seismic zone has been presented to underscore the benefits achievable with this technique in terms of duration and safety of the construction process.

Keywords

Beam, column, connection, earthquake, moment frame, petrochemical plant, pipe rack, reinforcement, seismic, wet joint.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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