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Seismic Performance of Precast Hybrid-Steel Concrete Connections

BING LI, SUDHAKAR A. KULKARNI, and CHEE LAI LEONG

School of Civil and Environmental Engineering, Nanyang Technological University, Singapore

This article presents experimental and analytical investigations of hybrid-steel concrete connections. In the experimental study, four full-scale specimens including one cast-in-place and three precast specimens were tested under cyclic load reversals. The performance of the specimens in terms of energy dissipating capacity, cracking patterns, and variation of strains along the main reinforcement is described. However, due to the inherent complexity of beam-column joints and the unique features of the tested specimens, the experimental investigation was not sufficient enough to fully understand the influence of several parameters. Therefore, an analytical investigation based on the FE models using DIANA software is presented. Validation of the FE models against the experimental results has shown a good agreement. The critical parameters influencing the joint's behavior such as the continuation of beam bottom reinforcement, column axial load, the size and embedded length of the angle sections are varied, and their effects including possible implications on code specifications are discussed.

Keywords Finite Element; Hybrid-Steel Concrete; Material Nonlinearity; Cyclic Loading; Geometric Nonlinearity; Hysteresis Loops

1. Introduction

For many years, precast concrete members have been known for the inherent benefits such as speed in productivity, considerable improvement in service and quality, and overall reduction in construction cost. A recent trend in low-to-moderate seismic regions like Singapore, Eastern, and Central parts of the United States, Malaysia, etc., has shown a steep hike in the usage of precast elements in construction. Although precast reinforced concrete (RC) elements can speed up the construction of structures, especially in high-rise buildings, proper design of joints and their execution during the construction is a matter of serious concern. The catastrophic failure of precast structures, particularly the joints during earthquakes, showed a possible drawback in the system. Major problems associated with precast structures during seismic events have been related to the low-energy dissipating capacity of precast elements and the ability of the overall structure to undergo large deformations without substantial loss of strength. In order to successfully achieve an innovative and robust connection system, Nanyang Technological University (NTU) Singapore has embarked upon a series of experiments on hybrid-steel concrete joints.

In precast structures, normally a beam rests on the column edges, thus coinciding with the inherent plastic hinge location. This kind of arrangement makes the beam-column joint the most vulnerable, especially under the action of seismic forces. The experimental study by Sheikh *et al.* [1989] showed that the crushing of concrete in

column located above and below the beam leads to vertical bearing failure, which may eventually cause rigid body rotation of the beam. Due to the limited information available on the seismic behavior of the hybrid-steel concrete structures in the inelastic range, some researchers [Hawkins and Englekirk, 1987; Englekirk, 1982; Dolan *et al.*, 1987] felt the necessity of standard guidelines for seismic design of precast structures. BS8110 [1997], the major code of practice used in Singapore, does not fully cover the specification for precast elements. To supplement this code, some other technical references on precast technology such as the PCI manuals and handbooks [PCI manual, 1973; PCI Design Handbooks, 1971; 1985], which contain some research findings since the 1970s, have been used. Khaloo and Parastesh [2003a, b] developed some models for the connection of precast beam and column members and performed experimental investigations to understand their behavior under cyclic load reversals. The test results indicated cracks at the beam-column junction at failure and the magnitude of storey shears carried by the specimens were also marginally low. An experimental investigation conducted by Korkmaz and Tankut [2005] to study the performance of precast beam-to-beam connections, which were fastened using welded plates at the bottom and lap splice for the top steel, exhibited a moderately satisfactory performance. The authors suggested a longer and better confined lap splicing including welding of bars for the cases of more heavily reinforced beams to improve the joint performance.

In order to supplement the scanty ongoing research on the hybrid-steel concrete joints, especially that on inelastic behavior under reversed cyclic loadings, NTU developed and tested a set of joints involving precast members fastened by using a variety of innovative connection techniques. These hybrid-steel concrete joints make use of steel sections into the beam-column joint region to facilitate the connection of precast elements. This article covers a comprehensive research involving experimental and FE numerical investigations. Test results of the experimental study consisting of one cast-in-place and three hybrid specimens, whose column-column connections slightly differed from each other, are summarized. Although the experimental investigation covered a variety of steel angle sections for column-column connections, several unique features of the tested specimens such as size, shape, reinforcement details, applied loading, etc., could not be varied. Furthermore, the effect of several key influencing parameters such as continuity of the bottom reinforcement in the beams, column axial load and the size and length of the angle sections used in column to column connection were difficult to comprehend due to a limited number of experimental specimens and the material heterogeneity. To enhance the understanding of the complex seismic behavior of hybrid-steel concrete beam column joints, and decide the influence of critical design parameters, an analytical investigation using a nonlinear FE tool is presented. The numerical investigation encompasses the development and calibration of a nonlinear FE model, and the study of hybrid-steel concrete joints by varying the key influencing parameters.

2. Test Program

2.1. Details of Specimens

The experimental program included a total of four specimens which were designated as Specimens JC1, JC2, JC3, and JC4. Specimen JC1 was a full-scale interior cast-in-place reinforced concrete beam-column joint (Fig. 1), while Specimens JC2, JC3, and JC4 were constructed assembling the precast elements using steel sections (Fig. 2). Brief details of the dimensions and reinforcement arrangement of Specimen JC1 are shown in Fig. 1. The specimen was designed and constructed following the code provisions of BS 8110 [1997].

FIGURE 1 Beam-column joint with cross section details.

A size of 400 mm × 400 mm was adopted for the columns, whereas the beams had a cross-section of 250 mm × 500 mm. Eight T20 bars were used as the main reinforcement in the columns, and four T20 bars at the top and three T20 bars at the bottom layers constituted for the main reinforcement in the beams. R10 bars with a spacing of 50 mm were used as the transverse reinforcement in both beams and columns. This unit was a replica of the critical joint in a moment-resisting frame. Figure 2 shows the connection details of Specimens JC2, JC3, and JC4. All the precast specimens had the same geometrical dimensions as Specimen JC1 and the reinforcement details of the beams and the upper columns were also identical to those of Specimen JC1. Unequal angles of size 200 × 100 × 12mm and partially embedded vertical and bottom steel plates of size 800 mm × 330 mm × 10 mm and 800 mm × 170 mm × 10 mm, respectively, were used to connect the beam and joint core. Vertical plates were connected the angle sections using four M24 size bolts, while two M16 bolts were applied to fasten the horizontal plate and angle section. The arrangement of the plates, angles and position of bolt holes are illustrated in Fig. 2. The column-to-column connection of all precast specimens was arranged by connecting four angle sections which were partially embedded in the columns. Figure 2 shows the arrangement of angle sections near the corners of the columns. To facilitate proper connection between the top and bottom columns, the size of the angle sections used in the upper and lower parts of the columns differed a bit (see Table 1).

2.2. Material Properties

The longitudinal reinforcement used in the beams and columns was 20 mm diameter deformed bars of Grade 460, while the transverse reinforcement employed for the beams and columns was 10 mm diameter mild steel of Grade 250. Test results obtained from samples of 20 mm and 10 mm diameter bars indicated an average yield stress of 492 MPa and 311 MPa, respectively. The concrete used for all specimens was of Grade 30. The

TABLE 2 Summary of tensile coupon test results

Section	Size D × B × t (mm)	E_o (GPa)	$\sigma_{0.2}$ (MPa)	σ_u (MPa)	ε_u (%)
Plate	170 × 330 × 10	199	328	619	49
Plate	800 × 330 × 10	195	320	615	53
Angles					
80 × 80 × 9		195	365	680	38
100 × 100 × 10		202	379	701	41
125 × 125 × 9		199	380	679	39

2.3. Instrumentation

A sufficient number of measuring devices were used in the experimental tests to record the strains and deformations. Strain gauges were placed on both beams and columns longitudinal reinforcements at selected locations within and around the joint region. The lateral displacements of the column top were measured using displacement transducers, while a range of transducers were installed within beam-column joint region to measure the flexural and shear deformations.

2.4. Test Setup

Figure 3 shows the test setup. While the loading arrangement, connection details, and test rig are shown in Fig. 3. Each of the test specimens was subjected to quasi-static load reversals that simulated earthquake loading. The bottom of the column was pinned to the strong floor of laboratory and ends of the beams were connected to this strong floor by steel links which permitted rotation and free horizontal movement of the beam but not the vertical movement, thus providing the vertical reactions to the beams. A reversible horizontal load was applied to the column using a double acting 1000 kN capacity hydraulic actuator. The cyclic loading history showing applied cycles versus the displacement ductility factor (DF) is shown in Fig. 4. In all tests, two cycles of horizontal loading up to $\pm 0.5P_i$ and $\pm 0.75P_i$ were initially applied, where P_i is the horizontal load at the top of the column associated with the theoretical flexure strength M_i being reached at the critical sections of the members, calculated using the conventional compressive stress block for the concrete with an extreme fiber concrete compressive strain of 0.0035 and the measured concrete compressive cylinder strength and the steel yield strength from

FIGURE 3 Typical test set-up.

FIGURE 4 Cyclic loading and displacement history.

tests. The yield displacement of all test specimens was calculated using the stiffness at the interstorey horizontal displacement measured at $0.75P_i$.

3. Experimental Observations and Results

3.1. Energy Dissipating Capacity

Figure 5a shows the storey shear force versus DF of hysteresis loops of Specimen JC1. During the initial cycles, the specimen showed an elastic behavior with no pronounced pinching in hysteresis loops. With further increments in the loading cycles to a DF of 1.0, the specimen attained the first yield corresponding to a storey shear force of 99 kN. At a DF of 2.0, the specimen reached the maximum storey shear force of 158.4 kN, which was approximately 8% greater than its theoretical capacity. The specimen continued to carry a higher value of storey shears up to a 3.0%, and exhibited a substantially good amount in energy dissipating capacity. The test was terminated corresponding to a DF value of 4.1, although the specimen had not failed at that stage.

Figure 5b illustrates the storey shear force versus horizontal displacement hysteresis loops of Specimen JC2. In the positive loading direction, the first yield occurred at a displacement of around 12 mm, corresponding to a DF of 0.81. At this stage, the specimen also reached a storey shear of approximately 81 kN. In the subsequent loading cycles, the specimen reached the maximum capacities of approximately 154 kN, in the positive loading direction and 151 kN, in the negative loading direction corresponding to a DF of 3.0. Local buckling of column angle sections at higher cycles led to the premature buckling failure of the specimen. At the initial stage, the specimen exhibited similar storey shear carrying capacities when compared to Specimen JC1. The hysteresis loops also experienced a good pinching behavior compared with Specimen JC1.

The storey shear force versus horizontal displacement relationship of Specimen JC3 is presented in Fig. 5c. In the positive loading direction, the first yield occurred at a displacement of 13.8 mm corresponding to a DF of 0.82. The maximum capacities, 162.8 kN in the positive and 171 kN in the negative loading directions, were reached at DFs of 2.6 and 3.0, respectively. In the positive direction, when compared with Specimen JC1 the maximum capacity of Specimen JC3 was 11% higher, while in the negative direction,

FIGURE 5 Hysteresis loops of the specimens.

it was around 12% greater. This indicated the fact that the hybrid-steel joint improved the performance significantly with respect to storey shear capacity. The specimen experienced the stiffness degradation and a significant pinching behavior after the second cycle, corresponding to a DF of 0.9. During the last loading cycle corresponding to a DF of 3.0, the specimen reached the load carrying capacity of approximately 152 kN.

Figure 5 d shows the storey shear force versus horizontal displacement relationship of Specimen JC4. At a DF of 0.8, the specimen attained the first yield displacement of around 14 mm and the corresponding storey shear was 98 kN. With the enhancement in the loading cycles, corresponding at a DF of 1.6, the maximum capacities of 198 kN in the positive and 211 kN in the negative loading directions were reached. The specimen experienced a significant pinching in the hysteresis loops beginning from a DF of 0.8. After attaining the highest storey shear capacity, the specimen experienced minimal stiffness degradation before finally reaching the residual capacity of 165 kN that corresponded to a DF of 3.8.

3.2. Strain Profiles of Main Reinforcement

Figure 6 illustrates the strain variation plots of the top beam main reinforcement of Specimens JC1 to JC4. For Specimen JC1, the bending resistance was developed through the whole beam span mainly due to the yielding of the top and bottom main bars. The experimental results showed that these bars experienced yield displacement corresponding to a DF of approximately 2.0. The maximum strain values attained in the top and bottom bars were around 0.012 and 0.006, respectively. As shown by Figs. 6b-c, the main beam bars of Specimens JC2–JC4 did not show much yielding and remained in elastic limit for major part of the loading. Specimen JC2 failed slightly in a premature manner

FIGURE 6 Variation of strains along beam top bars.

with strains not reaching very high values. It was the buckling of one or more steel sections that caused the failure. On the other hand, a steady increase in the strains was particularly noticeable in Specimens JC3 and JC4 with the strains at some sections of column faces surpassing yield strains and penetrating through the joint cores. This trend indicated that these specimens successfully sustained the applied loads up to a DF of 3.0 and 4.0, respectively, with a reasonably good amount of energy dissipating capacity. The yielding in the column bars was also significant when a DF of 3.0 was reached.

3.3. Cracking Pattern

Figure 7 illustrates the final cracking pattern of all specimens. For Specimen JC1, cracks initiated at the intersection of beam and column regions, i.e., potential plastic hinge regions. With the increase in the loading runs, the cracks further propagated to other areas. A few diagonal cracks were also initiated corresponding to a DF of 2.0, and they further propagated along the diagonal directions, as the loading was progressed. When the DF reached a value of 4.0, the concrete cover began to crush forming an obvious diagonal compression strut.

In Specimen JC2, firstly, cracks appeared close to the beam top regions, when the DF was 1.0 (Fig. 7a). Simultaneously, a few vertical cracks (interface cracks) between precast beam and cast in-situ concrete at beam-column joint area, and significantly dense cracks on the top and bottom of the columns close to the joint, were also formed. As the loading run was increased to a DF of 2.0, while the existing flexural cracks propagated rapidly, more additional flexural cracks were appeared at bottom of the beams, slightly away from the beam-column joint region. Besides, the specimen also experienced some diagonal cracks, while spalling of concrete began simultaneously, thereby reducing the specimen's load

carrying capacity substantially. The specimen failed abruptly at a DF of 2.5 due to the crushing of concrete in the column-to-column region; steel angles buckled away causing crushing of the concrete located in that region. At this stage, the specimen experienced a substantial reduction in its strength and the test was terminated.

Specimens JC3 and JC4 had similar crack patterns formation as JC2 as shown by Figs. 7c-d. The failure modes were quite analogous to Specimen JC2, with crushing of concrete in the joint region, while buckling of the angles was highly controlled due to increase in their size and thickness. In the meanwhile, Specimens JC3 and JC4 did not fail prematurely like Specimen JC2. From the beginning of the first cycle, initiation and propagation of cracks at the bottom and top of the beams was observed. Besides, the upper part of the column experienced a bit of diagonal cracking. In the final stage of loading, corresponding to a DF of 4.0, a small amount of spalling of concrete coupled with some minor diagonal cracks in the joint region was followed. In a way, the cracking pattern also resembled to that of Specimen JC1, where cracks took place both at the top and bottom beams while the column was stiff enough to resist the applied horizontal loading. The use of thicker and bigger size steel angle sections contributed successfully to an effective

a) JC1

b) JC2

FIGURE 7 Cracking patterns of the specimens.

c) JC3

d) JC4

FIGURE 7 (*Continued*).

column-to-column connection. Finally, the specimens were failed when concrete in the column-to-column connection area had crushed and spalled off corresponding to a DF of 3.5.

3.4. Decomposition of Lateral Displacement

Figure 8 explains the decomposition of displacement components of JC1–JC4, respectively. The beams initially showed a flexibility almost similar to columns when the load was applied on Specimen JC1. As the load was increased to a higher level, more flexibility was shown in column's components. This shows that the column-to-column connection of JC1 was inadequate to form a strong column, hence the high flexibility shown. This can also be confirmed with the premature failure of the specimen at the column connection region during test observation. Similarly, both JC2 and JC3 failed to form a strong precast column connection where premature took place at DFs of 2.5 and 3.0, respectively. A very high flexibility was shown in column's components prior to failure, which conformed to the cracking pattern in the test. To form a good beam-column joint to resist horizontal load, the combination of strong-column-weak-beam is essential. However, in JC1, JC2, and JC3, it is

FIGURE 8 Decomposition of lateral displacement.

noted that the column was weaker with a higher flexibility when the load was increased. Their respective beams were stiffer if compared to the columns which formed an unfavorable weak-column-strong-beam combination to resist horizontal load.

As for JC4, both the beam and column showed a stable flexibility throughout the test where the contribution of displacement was similar in both beam and column. Even though the column had a slightly higher flexibility when the test was stopped, the column-to-column connection was still found to serve as a good connection to resist horizontal load. No premature failure took place at the end of the test which confirmed that the proposed column-to-column connection in JC4 was capable of resisting the applied horizontal load. On the other hand, the contribution from the joint core in each specimen was not significant with a contribution ranged from 3–15%.

4. Finite Element Analysis

It is possible to more thoroughly evaluate the stresses and deformations in a structure using the FE analysis than can be done experimentally. The nonlinear analysis results in a

better understanding of the mechanical behavior of a structure during its loading to fracture. In the present study, the specimens were analyzed using the DIANA software [1991]. Two-dimensional (2D) plane stress elements were used to simulate the concrete and steel plates, while reinforcing bars were modeled as truss elements. In material modeling, the concrete models were based on nonlinear fracture mechanics to account for cracking, and plasticity models were used for the concrete in compression and steel reinforcement.

4.1. Modeling of Concrete

The analysis uses a constant stress cut-off criterion for cracking of the concrete. According to this model, a crack is assumed to have initiated perpendicular to the major principal stress if its value exceeds the tensile strength, independent of other principal stresses. Further details about the crack formation, its orientation, angle limitation between primary secondary cracks, etc., may be referred from Hajime and Kohichi [1991]. The fracture energy G_F and the tensile strength f_t were used to calculate the value of ultimate crack opening w_u . The fracture energy G_F of the concrete was calculated using a three-point bending test based upon the recommendations of RILEM 50-FMC [1985]. To simulate the softening effect of the concrete in tension after cracking, a bilinear tension stress-strain curve was used as shown in Fig. 9a in which ε_u^{cr} was taken as 0.001. The value was based on the assumption that the strain softening after failure reduces the stress linearly to zero at a total strain of about 10 times the strain at failure of concrete in tension, which is taken as 0.0001 based on authors' parametric studies. The uniaxial tensile strength of concrete f_t used in the analysis was determined from the compressive strength f_c according to CEB-FIP Model code [1993]:

$$f_t = 0.30(f_c')^{2/3}. \quad (1)$$

The response of the concrete in compression was taken into account by an elastic-plastic model. The elastic state of stress was limited by a Drucker-Prager yield surface. Isotropic hardening with an associated flow rule was used after yielding of the surface had occurred. The DIANA software evaluates the yield surface using the current state of stress, the angle of internal friction ϕ , and the cohesion c . As per the recommendations of the DIANA software manual [1991], the angle of internal friction in concrete can be approximated to be 30° . The cohesion c used in the analysis is given by formula as follows:

$$c = f_c(\varepsilon_{uniaxial}^p) \frac{1 - \sin \phi}{2 \cos \phi}, \quad (2)$$

where $f_c(\varepsilon_{uniaxial}^p)$ is the hardening or softening parameter as a function of the plastic strain in the direction of the uniaxial compression stress. CEB-FIP [1993] recommendations can be used to evaluate the post peak behavior of the concrete using cylinder compression strength tests. For the post peak behavior of concrete in compression, when concrete is unloaded/reloaded, the response is evaluated using the initial elastic stiffness (Fig. 9a). The unloading or reloading response of the post peak behavior, which is linearly elastic, is shown in Fig. 9b. A Poisson's ratio of 0.15 was used in the analysis.

4.2. Modeling of Reinforcement and Steel Plates

The bars were modeled with the DIANA options of either embedded reinforcements or according to the recommendations of separate truss elements. In the case of embedded

FIGURE 9 Material modeling.

reinforcement, the reinforcement does not have separate degrees of freedom. The strength and stiffness of the concrete elements were increased in the direction of the embedded reinforcement; the option assumes perfect bond between the reinforcement and the

surrounding concrete. However, in case of the reinforcing bars modeled as separate truss elements in combination with interface elements, the interaction between the reinforcement and the concrete was accounted. Figure 9c defines the stress-strain relationship for the reinforcing steel, which was modeled with an elasto-plastic curve.

The steel plates were modeled with 2D plane stress elements and were assigned the material properties of steel. The constitutive behavior of plate elements were modeled with von Mises yield criterion with isotropic strain hardening and an associated flow rule.

4.3. Bond Slip Law

The bond law used in the analysis is based on CEB-FIP Model Code [1993], as shown by Fig. 10. Equations of bond stress for different parts of the curve are described below:

$$\begin{aligned} \tau &= \tau_{\max} \left(\frac{s}{s_1} \right)^\alpha && \text{for } 0 \leq s \leq s_1 \\ \tau &= \tau_{\max} && \text{for } s_1 \leq s \leq s_2 \\ \tau &= \tau_{\max} - \left(\tau_{\max} - \tau_f \right) \frac{s-s_1}{s_3-s_2} && \text{for } s_2 \leq s \leq s_3 \\ \tau &= \tau_f && \text{for } s_3 \leq s \end{aligned} \quad (4)$$

The bond law model parameters depend on the properties of the bar surface, and can be referred from the CEB-FIP Model Code [1993].

4.4. Solution Algorithm

The Newton-Raphson method was initially applied to solve the nonlinear equations. After a gradual increase in load, the steps were followed by the arc-length technique combined with the line search method. The number of load steps required to minimize the work done by the unbalanced forces can be determined by adopting the line search method. Using the arc length method, it is possible to locate the descending part of the post-peak behavior and snap-back phenomenon as illustrated in Fig. 11. It is necessary to decide a suitable convergence or divergence criterion when the equilibrium position is accepted as converged state or need to be modified due to divergence. A maximum limit of 40 iterations was used for the convergence and the tolerance was taken as 0.001. From the analyses it was observed that the convergence generally occurred in less than five iterations. All the specimens were applied with quasi-static simulated seismic loading as shown in Fig. 4.

FIGURE 10 Bond-slip law by CEB-FIP Code 1990.

FIGURE 11 Snap through buckling phenomenon.

5. Verification of Finite Element Model

5.1. Specimens Modeling

In order to validate the FE model, the predicted responses were compared with the experimental results. Figure 12 shows the FE modeling of the specimens. For the beams, truss elements used for modeling the reinforcement bars were discontinued at the face of the column. Steel plates used for connection at the joint, extended inside the beam at one side and abutted with the column face on the other side. Beam elements located in the left and right parts of the joint were modeled using plane stress elements. These elements were assigned with steel plate thickness and its material properties. The concrete on the front and rear side of these elements was neglected in the analysis as it

FIGURE 12 FE model with element details and boundary conditions.

was filled up after the connections were fastened. Elements of the column below the joint region were simulated using 2D concrete elements, contact elements, and truss elements. Areas of angle sections used for the connection of column to column connection were appropriately transferred to the truss elements. Intersections of the upper and lower parts of the columns were joined by using contact elements. 2D model assumption of treating the plate area equivalent to the truss element is approximate, though it has fairly validated the behavior of the joints. In the joint core region, the area of truss elements close to the boundary was increased appropriately to simulate appropriate contributions from the steel plate areas.

5.2. Load-Displacement Responses of Specimens

Comparison of the FE numerical and the experimentally observed responses of the specimens are presented in Fig. 5. As shown by Fig. 5a, the FE numerical model of Specimen JC1 seemed to have predicted a good response with respect to the experimental observations. Although the predicted displacements of the model for a few initial cycles were slightly higher, the later cycles' results predicted were in good agreement with the experimental observations. Figure 5b shows comparison of the FE numerical and the experimental results of Specimen JC2. The experimental results showed a bit of large initial displacement for a few initial cycles. During the initial stage, the FE numerical responses achieved higher story shears compared to the experimental observations. As shown by Fig. 5c, the FE numerical Specimen JC3 reached a DF of approximately 3.7, which was slightly higher when compared to its experimental value. Similar to Specimen JC2, the initial cycles of the experimental loops showed a bit of large initial displacements. This may be due to the fact that the connections might have had some initial gaps between the nuts and bolts at the fastening positions of the plates, which might have been slipped after the application of load leading to large initial displacements. Comparison of the hysteresis loops between the FE numerical and the experimental results of Specimen JC4 are presented in Fig. 5d. Beginning from the initial cycles, a good energy dissipating capacity was observed in both the loops. The story shears achieved in both the numerical and the experimental results showed a very good agreement. Specimen JC4 sustained a loading of approximately 4.0 DF with a stable and stiff behavior throughout all the cycles.

Comparison of strain profiles along the main reinforcement also showed a good agreement between the FE numerical and the experimental results. Analogous to the experimental observations, the predicted strains in the bars also kept the low strain values. Yielding of the plates was observed during the final cycles of the loading. It was also observed that the FE models witnessed almost analogous cracking patterns within the joint core, near the compression and tension faces of the columns and beams, beginning from an early stage. This was followed by widening of the cracks as the horizontal displacement was increased. From the aforementioned observations and predictions of both the global and local behaviors using the FE analysis, the use of FE modeling techniques can, therefore, be further extended to study the joint performance by varying different parameters.

6. Parametric Studies

In order to enhance the understanding of the structural response with hybrid connections, the FE modeling technique was applied by varying critical influencing parameters such as the continuity of bottom reinforcement in the beams, the column axial load and the

column angle size and thickness. The following sections present the key parametric investigations and their possible implications on code provisions.

6.1. Influence of Beam Bottom Reinforcement Continuity

In the experimental study, the precast elements were connected by the plates and angle sections, while the reinforcement at the bottom of the beams remained discontinued. Because of reinforcement discontinuity, higher stress levels were seen at the lower part of plate elements. This was obvious due to the reduction in the lever arm and subsequently, the decrease in the depth of the neutral axis. During the FE analysis it was also observed that at high DFs, the plate elements experienced a large deformation followed by yielding at some parts. In order to avoid the plate failure, which substantially reduces the flexural capacity of the beam, the effect of beam bottom reinforcement continuity was investigated. The continuity of bottom reinforcement was maintained by extending the truss elements of the beam bottom reinforcement. Figures 13–15 show the load-displacement plots of the specimens with a variation in reinforcement from 0.5 to 1 of the gross area A_g . This steel percentage varied was greater than the minimum longitudinal reinforcement for beams specified by NZS 3101 [1995]:

$$\rho_{\min} = \frac{\sqrt{f'_c}}{4f_y} \times 100. \quad (3)$$

As shown by Fig. 13, Specimen JC2 showed a hike in story shears approximately by 4 kN and 8 kN, respectively, for the reinforcement values 0.5 and 0.75 of A_g . However, no appreciable improvement in the story shear was observed as the reinforcement was further enhanced to 1 of A_g . Reinforcement continuity also showed a steady energy dissipation in the specimens as was observed from the hysteresis loops. Analogous trends

FIGURE 13 Load-displacement predictions with continuity in bottom reinforcement for Specimen JC2.

FIGURE 14 Load-displacement predictions with continuity in bottom reinforcement for Specimen JC3.

FIGURE 15 Load-displacement predictions with continuity in bottom reinforcement for Specimen JC4.

were also observed from the FE predictions of Specimens JC3 and JC4. As shown by Figs. 13 and 15, Specimen JC3 exhibited a hike in storey shears by approximately 5 and 9 kN, while Specimen JC4 showed as increase of 4 and 9 kN, when the reinforcement was varied by 0.5 and 0.75 of A_g , respectively. The aforementioned discussion clearly indicated that continuation of beam bottom reinforcement improved the performance of the joints with strength reaching a maximum increase of approximately 9, when the

reinforcement value was 0.75 of A_g . Besides, the stress distribution around the joint region exhibited a smooth distribution and a relatively lower level of magnitude.

6.2. Effect of Column Axial Loads on Joint Behavior

Column axial loading is a critical parameter in the investigation of beam-column joints, but its effect on seismic behavior of beam-column joints has not yet been fully understood. Investigations by Paulay [1989] have shown that axial force is beneficial to the joint shear resistance. Since the neutral axis depth in the column increases with axial compression load, a larger portion of the bond forces from the beam bars can be assumed to be transferred to the diagonal strut. Therefore, the concrete contribution to the joint shear resistance will be increased [NZS, 1995]. Pessiki *et al.* [1990] experimentally investigated two non ductile interior beam-column joints with different axial loading levels. However, both of these specimens failed due to the pullout of the embedded beam bottom bars instead of joint shear failure. Lin's investigations showed that axial compression in excess of $0.3 f'_c A_g$ became detrimental to the joints. In a study conducted by Fu *et al.* [2000], it was pointed out that if the shear was small, the increase of axial loads was favorable to the joints, whereas for high shears, the increase of axial loads was unfavorable. Li *et al.* [2003] found that for an oblong joint, an axial load less than $0.4 f'_c A_g$, was beneficial to the joint, while the axial compression load ranging between zero to $0.2 f'_c A_g$ enhanced the joint's performance deep wall-like column joints.

The present investigation was aimed to find the influence of axial loading on the seismic behavior of hybrid-steel concrete joints. In this study, all the specimens were reanalyzed by applying the different levels of axial loads on the column. The same loading histories as those used in the analysis of specimens without axial loading were applied, and the storey shear force versus horizontal displacement plots corresponding to different axial load levels were plotted for Specimens JC2-JC4 (Figs. 16–18). As shown by Fig. 16, Specimen JC2 attained a maximum value of storey shear when axial load ratio

FIGURE 16 Load-displacement predictions under different axial loading levels for Specimen JC2.

FIGURE 17 Load-displacement predictions under different axial loading levels for Specimen JC3.

FIGURE 18 Load-displacement predictions under different axial loading levels for Specimen JC4.

was $P/f'_c A_g = 0.3$. The FE analysis further showed that a further increase in axial load decreased storey shear force and the ultimate number of cycles reached by the specimens also reduced. This was clearly reflected when the program was terminated due to very small or negative value of one or more diagonal elements in stiffness matrix. Specimens JC3 and JC4 (Figs. 16–18) exhibited quite analogous trends when analyzed by applying column axial loads. The predictions of these specimens showed that there was reduction in the story shears and the ultimate number of cycles, after enhancement in the column

axial load ratio beyond 0.3. Therefore, the analysis results clearly suggested that the axial load ratio $P/f'_c A_g 0.3$ was beneficial to the joint's performance, while any increase in the axial load ratio $P/f'_c A_g 0.3$ was found to be detrimental joint behavior, with the reduction in the story shears and energy dissipating capacity.

6.3. Influence of Connection Angle Size and Length

Connection angles in the column to column connection region play a key role in transferring the column axial load and moment between the column parts and hence, its successful design extremely important. In the experimental investigations, angle sections of different size and thickness were tried to understand their influence on the story shear and energy dissipating capacities. Although the experimental studies roughly indicated some clues about the size of angles for the column to column connections, because of the limited number of tested specimens the influence of length, size, and thickness was not fully understood. Besides, some experimental specimens experienced a moderate buckling of the angles due to local instability in the column to column connection region, the reason for which was also not known. Therefore, this study is intended to elucidate more information about the performance of joints by varying certain key parameters related to angle sections. The specimens were analyzed by varying the embedded length and size of the angle sections. In the FE modeling, the areas of the angle sections were simulated to the nearby truss elements by considering their equivalent areas, and contact elements were used at the intersection between the connection of the top and bottom columns. To investigate the stress distribution and embedded length effects thoroughly, the nearby the column to column connection region was modeled using a fine mesh. Table 3 shows the comparison the maximum story shears carried by the specimens with different size of angle sections. In the experimental set up, the angles were embedded inside the columns by 60 times the connection angle thickness (t). It may be noted that the specimen with an angle size of $100 \times 100 \times 10$ mm and an embedded length of $70t$ proved to be better in carrying the story shears. It also exhibited a good energy dissipating capacity. On the other hand, the specimen with $125 \times 125 \times 9$ mm angle sections for column to column section showed a marginal enhancement in story shears when compared to the angle section of size $100 \times 100 \times 10$ mm. Although the increase in the story shear was only about two, and the energy dissipating capacity of both these specimens were almost similar. From the aforementioned comparison, it was clear that the connection angle size and its embedded length strongly influenced the story shears and energy dissipating capacity of the specimens. Angle section $100 \times 100 \times 10$ mm with an embedded length of $70t$ resulted in higher storey shears followed by an optimum energy dissipating capacity.

TABLE 3 FE predictions of the column to column angle sections

Angle sections, $D \times B \times t$ (mm $\times$ mm $\times$ mm)		Maximum story shear capacity for different embedded lengths, kN	
In top column	In bottom column	$50t$	$70t$
$90 \times 90 \times 10$	$90 \times 90 \times 10$	167.1	171.6
$100 \times 100 \times 10$	$90 \times 90 \times 10$	175.5	177.9
$100 \times 100 \times 10$	$100 \times 100 \times 10$	196.5	204.6
$125 \times 125 \times 9$	$125 \times 125 \times 9$	199.5	206.8

7. Conclusions

The hybrid-steel concrete connection for seismic behavior was investigated using the experimental and the FE numerical models. Based on the observations and results from these studies, the following conclusions can be drawn:

1. Experimental observations showed that precast specimens under cyclic loading experienced no abrupt damage within the joint core region and therefore, the final failure was not controlled by the capacity of the joint core. The precast specimens' performance was quite good exhibiting adequate ductile behavior under the seismic loading and it also agreed well with the cast-in-place specimen. Embedment of the steel sections in joint greatly enhanced the strength of the joint core with the specimens carrying story shears up to a DF of 3.5.
2. Joint region detected no severe damage in the form of plastic hinge, even when the specimens were loaded to a very high DF of 3.5. Beam-to-column connection of the precast specimens was proved to be sufficiently stiff and ductile, thereby effectively resisting both shear forces and bending moments.
3. Joint core regions of the precast specimens were adequately confined by the incorporated steel sections, providing a significantly high degree of restraint and reducing the joint core deformation under the reversed cyclic loading.
4. Experimental and FE numerical investigations showed that size and thickness of steel angle sections strongly influenced the storey shears generated from the cyclic loading. Specimens with inadequate angle size and thickness experienced premature failure at the connection region with buckling of the angle sections followed by crushing of concrete. A higher story shear followed by an optimum energy dissipating capacity was obtained using the angle $100 \times 100 \times 10$ mm and an embedded length of 70 times the angle thickness.
5. The FE numerical investigation showed that continuation of reinforcement in the bottom of the beams enhanced the maximum story shears carried by the specimens and helped to decrease the joint degradation at higher ductility factors. Predicted results indicated an optimum hike of approximately 9 in story shears when the bottom reinforcement in beams was 0.75 of A_g . The FE analysis further showed that any increase in reinforcement beyond 1 of A_g did not significantly increase the strength in terms of story shears.
6. The FE numerical results showed that column axial load was beneficial to the joint's performance. Axial load ratio $P/f'_c A_g = 0.3$ produced optimum results with an enhancement in the story shears by 9 and also, improving the energy dissipating capacity. However, any increase in axial load ratio beyond $P/f'_c A_g = 0.3$ adversely affected the joint's performance.

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