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# Finite element analysis of precast hybrid-steel concrete connections under cyclic loading

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# Abstract

In this paper, a nonlinear finite element (FE) analysis of hybrid-steel concrete connections is presented. The detailed experimental results of the four full-scale hybrid-steel concrete connections with limited seismic detailing have been discussed in a different paper. However, due to the inherent complexity of beam–column joints and the unique features of the tested specimens, the experimental study was not comprehensive enough. Therefore, in this paper, an analytical investigation based on the FE models and using the DIANA software is presented. The FE models were validated using the experimental results of the hybrid-steel concrete connections tested in Nanyang Technological University, Singapore. The critical parameters influencing the joint's behaviour, such as the axial load on column, the connection plate thickness, and the continuation of beam bottom reinforcement, are varied, and their effects, especially implications on code specifications, are studied. (© 2007 Elsevier Ltd. All rights reserved.

Keywords: Finite element; Hybrid-steel concrete; Plane stress element; Cyclic loading; Geometric nonlinearity; Hysteresis loops

# 1. Introduction

In the last two decades, a lot of research has been devoted to the nonlinear analysis of reinforced concrete structures subjected to seismic forces. Reinforced concrete structures being the most commonly used structures need proper design and utmost care in the joint construction. During cyclic loading of a structure, the joint should be ductile enough and capable of dissipating large amounts of energy. A recent trend suggests that regions of low-to-moderate seismicity like Singapore, Eastern and Central parts of United States, Malaysia, etc. have witnessed a rise in construction activity with precast elements. It is already an established fact that precast structures are advantageous in terms of productivity, economy and quality control. However, the catastrophic failure of structures, particularly the joints during earthquakes, showed a possible drawback in the system. The information available on the seismic behaviour of the hybrid-steel concrete structures in the inelastic range is limited, thus necessitating the need for

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standard guidelines of seismic design of precast structures [1-3].

A beam normally rests on the column edges, thus coinciding with the inherent plastic hinging location. This makes the joint most vulnerable under seismic actions if the connections are not properly designed for the required strength and ductility. Vertical bearing failure may occur if the concrete of the column located above and below the beam is crushed, eventually leading to a rigid body rotation of the beam that takes place within the reinforced concrete column [4]. BS8110 [5], which is 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 [6–8], which contain some research findings since the 1970s, have been used.

To augment the ongoing research in precast technology construction, particularly the behaviour of connections in the inelastic range, some hybrid-steel concrete joints have been investigated in Nanyang Technological University, Singapore. These innovative hybrid-steel concrete connections make use of steel sections into the beam–column joint region to facilitate the connection of precast elements. In the experimental study, one cast-in-place and three hybrid specimens, whose connection configurations slightly differed from each other, were tested.

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However, due to unique features of the tested specimens and material heterogeneity, it was difficult to understand the complex seismic behaviour of beam–column connections. Furthermore, the effect of several influencing parameters such as plate thickness, axial load and continuity of beam bottom main reinforcement cannot be varied in a limited number of experiments. In order to quantify and decide the influence of critical design parameters, a robust numerical model is necessary. Therefore, this paper is aimed at developing and calibrating a nonlinear FE model, and further use it to investigate the behaviour of hybrid-steel concrete joints by varying the main control parameters.

Plastic strain in uniaxial stress direction

Minimum longitudinal reinforcement in beams

# 2. Test program

Nomenclature

 $A_{g}$ 

С

В

D

 $E_o$ 

 $f_{c'}$ 

 $f_t$ 

 $f_y$ 

 $G_F$ 

 $N^*$ 

 $w_u$ 

 $\sigma_{0.2}$ 

 $\sigma_u$ 

 $\varepsilon_u^{\rm cr}$ 

 $\varepsilon_u$ 

φ

 $\rho_{\rm min}$ 

 $\varepsilon_{\rm uniaxial}^{\ddot{p}}$ 

t

Gross area of section

Initial Young's modulus

Concrete tensile stress

Compressive stress

Overall width of cross-section (smaller dimen-

Overall depth of cross-section (larger dimension)

function of the plastic strain in the direction of

 $f_c(\varepsilon_{\text{uniaxial}}^p)$  Hardening or softening parameter as a

the uniaxial compression stress

Fracture energy of concrete

Axial compressive load

Ultimate crack opening

Static 0.2% proof stress

Ultimate strain in concrete

Elongation after fracture

Angle of internal friction

Static tensile strength

Thickness of steel section

Yield strength of reinforcement bars

Cohesion

sion)

A total of four specimens tested are briefly summarized in this section. One full-scale interior cast-in-place reinforced concrete beam–column joint M1 and three other full-scale interior precast beam–column joints M2, M3, and M4 were constructed and tested. The dimensions and reinforcement details of the reinforced concrete Specimen M1 are shown in Fig. 1. This unit was a replica of the critical joint regions in a moment-resisting frame. The specimen was designed and constructed according to BS 8110 [1]. The beam section was 250 mm by 500 mm, and the ratio of top beam bar to bottom beam bar was 4 to 3. The column cross section was 400 mm × 400 mm, and eight T20 reinforcements were used as the main reinforcement. The beam span was 4.0 m and the height of the column was 2.725 m. Fig. 2 shows the connection details of Specimen M2, M3 and M4. Precast Specimen M2 had the same geometrical dimensions as Specimen M1. The reinforcement details of the beams and the upper columns were identical to those of Specimen M1. However, a designed steel angle and plate connection was used to assemble the two precast beams to the joint core, while steel I-section to I-section connection was used to connect the upper and lower parts of the column (Fig. 2). Unequal angles of size  $200 \times 100 \times 12$  and partially embedded vertical and bottom steel plates of size 800 mm  $\times$  $330 \text{ mm} \times 10 \text{ mm}$  and  $800 \text{ mm} \times 170 \text{ mm} \times 10 \text{ mm}$  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 angel section. The arrangement of the plates and angles with bolt hole positions are illustrated in Fig. 2(a). The column-to-column connection of Specimen M2 was obtained by connecting two steel sections UC  $254 \times 254 \times 73$  which were embedded in the upper and lower columns. Splice plate of size 560 mm  $\times$  254 mm  $\times$  10 mm and twelve M24 size bolts were applied to connect the flanges, while splice plates of size 230 mm  $\times$  200 mm  $\times$  8 mm with six M24 bolts were used to connect the web. For precast Specimen M3, the dimensions, the reinforcement details of the beams and the upper column were identical to Specimens M1 and M2. Similar to Specimen M2, a connection consisting of steel angles and plates was used to join the precast beams and columns. On the other hand, steel square hollow sections (SHS) of size  $300 \times 300 \times 10$  mm were used to combine the upper and the lower columns. The SHS were properly connected using splice plates of size 390 mm  $\times$  240 mm  $\times$  10 mm and eight M24 size bolts, thereby ensuring the transfer of moment and shear between the two parts of the column (Fig. 2(b)). For Specimen M4, the beam column connection was kept as those used in Specimens M2 and M3, while UC  $254 \times 254 \times 73$  sections with plates and bolts were used to assemble the columns. The plates of size 260 mm  $\times$  260 mm  $\times$  10 mm were initially welded to UC sections and then joined by six M24 bolts (Fig. 2(b)).

# 3. Analytical model

# 3.1. Material properties

The longitudinal reinforcement of the beam and column was deformed bars of Grade 460, while the beam stirrups and column transverse ties were applied with Grade 250 bars. The concrete used for all specimens was of Grade 30. The slump value of the concrete mix was  $75 \pm 25$  mm. The average compressive strength of concrete calculated using the cube samples was found to be 28.9 MPa. Steel SHS, I-sections, and angle sections used in the construction of the specimens were confirmed to Grade E43. Average values of steel section properties were obtained from the samples of tensile coupon tests. The measured properties were the static 0.2% proof stress ( $\sigma_{0,2}$ ), the static tensile strength ( $\sigma_u$ ), the initial Young's modulus  $(E_{\rho})$  and the elongation after fracture  $(\varepsilon_{\mu})$ , which are presented in Table 1. Since the thickness of two splice plates or support plates used in the connection was greater than the main connected components such as flanges, web, etc., the properties



Fig. 1. Beam-column joint with cross section details.



Fig. 2. Typical connection details (a) Isoperimetric view showing the joint assembly; and (b) column to column connection details.

of the later were accounted in the FE analysis. The steel sections were connected using the high strength bolts of size M24 and M16 of Grades 8.8 or 10.8, respectively. The properties of the embedded plate were considered for the FE analysis because the thickness of two angle sections used in the connections was more than the plate.

# 3.2. Finite element modelling

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 behaviour of a structure during

Table 1 Summery of tensile coupon test results

Section	Size $D \times B \times t$ (mm)	E <sub>o</sub> (GPa)	σ <sub>0.2</sub> (MPa)	$\sigma_u$ (MPa)	е <sub>и</sub> (%)
SHS	$300 \times 300 \times 10$	202	330	688	48
Plate	$800 \times 330 \times 10$	195	320	615	53
UC					
$254 \times 254 \times 73$					
Web	$254 \times 254 \times 8.6$	197	379	705	39
Flange	$254\times254\times14.2$	201	366	679	41

its loading to fracture. In the present study, the specimens were analysed using the DIANA software [9]. Two-dimensional (2D) plane stress elements were applied to simulate the concrete and steel plates, while reinforcing bars were modelled as truss elements. In material modelling, the concrete models were based on nonlinear fracture mechanisms to account for cracking, and plasticity models were used for the concrete in compression and steel reinforcement.

# 3.3. Modelling 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 be initiated perpendicular to the major principal stress if its value exceeds the tensile strength, independent of other principal stresses. The orientation of the crack is then stored and the material response perpendicular to the crack is determined by a stress–strain relationship for the cracked material volume. Additional cracks may appear at the same location, but their formation to the existing cracks is greater than 15 deg. However, if the angle is less than that, the secondary cracks are assumed not to be generated even when the tensile stress has reached the fracture envelope [10].

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 threepoint bending test based upon the recommendations of RILEM 50-FMC [11]. To simulate the softening effect of the concrete in tension after cracking, a bilinear tension stress–strain curve was used as shown in Fig. 3(a) in which  $\varepsilon_u^{cr}$  is 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 typically 0.0001. The uniaxial tensile strength of concrete  $f_t$  used in the analysis was determined from the compressive strength  $f'_c$  according to the CEB-FIP Model code [12]:

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

When the cracked concrete is unloaded in tension, the secant modulus is used to evaluate the stiffness owing to the fact that the strain across the crack is linearly reduced to zero as the stress approaches to zero (Fig. 3(b)). However, when the concrete in compression is unloaded, the initial stiffness is adopted for the stiffness calculations (Fig. 3(a)).

The response of the concrete in compression was taken into account by an elastic-plastic model. The elastic state



Fig. 3. Material modelling.

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  $\phi$ , and the cohesion *c*. As per the recommendations of the DIANA software manual [9], 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:

$$c = f_c \left(\varepsilon_{\text{uniaxial}}^p\right) \frac{1 - \sin\phi}{2\cos\phi} \tag{2}$$

where  $f_c(\varepsilon_{\text{uniaxial}}^p)$  is the hardening or softening parameter as a function of the plastic strain in the direction of the uniaxial compression stress. Standard uniaxial tests on concrete cylinders were used to define the stress–strain relations up to the peak stress. CEB-FIP recommendations can be used to evaluate the postpeak behaviour of the concrete using cylinder compression strength tests [12]. A Poisson's ratio of 0.15 was used in the analysis.

#### 3.4. Modelling of reinforcement and steel plates

The von Mises yield criterion with isotropic strain hardening and an associated flow rule were used to describe the constitutive behaviour of the reinforcement. The bars were modelled with the DIANA options of either embedded reinforcements or according to the recommendations of separate truss elements. In the case of embedded 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 bonding between the reinforcement and the surrounding concrete. However, in case of the reinforcing bars modelled as separate truss elements in combination with interface elements, the interaction between the reinforcement and the concrete was accounted for. Fig. 3(c) defines the stress-strain relationship for the reinforcing steel, which was modelled with an elasto-plastic curve.

The steel plates were modelled with 2D plane stress elements and were assigned the material properties of steel. The constitutive behaviour of plate elements were modelled with the von Mises yield criterion with isotropic strain hardening and an associated flow rule. Perfect bonding between the concrete and steel plates was assumed in the analysis.

#### 3.5. 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 minimise 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 behaviour and snap-back phenomenon as illustrated in Fig. 4. It is necessary to decide a suitable convergence or divergence criterion when the equilibrium position is accepted as a converged state or needs to be modified due to divergence.



Fig. 4. Snap through buckling phenomenon.



Fig. 5. Cyclic lateral loading and displacement history used in the tests.

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 5 iterations.

The analysis assumed the total Langrangian approach with small strains and large displacements. Initial investigations on some specimens showed the effect of geometric nonlinearity did not much vary the results since extremely large displacements did not occur. Hence, the geometric nonlinearity was neglected later in the analysis. All the specimens were applied with quasi-static simulated seismic loading as shown in Fig. 5. The first two cycles were load controlled and the remainder were displacement controlled.

#### 4. Verification of finite element model

#### 4.1. Specimens modelling

To verify the finite element model, the analytical results were compared with the experimental results. The specimens were modelled (Fig. 6) with a total of 512 elements: 320 truss elements and the remaining plane stress 2D elements. Concrete was modelled using 2D plane stress elements which were four-node isoparametric elements. On the other hand,



Fig. 6. Enlarged view of FE model at joint region with element details.

the reinforcing steel bars were modelled as two-node truss elements. Fig. 6 shows an enlarged joint core view with plate and truss elements. At the joint core region the area of truss elements close to the boundary such as 7, 8, 9, 10, 11, 12 etc. were increased appropriately to simulate their corresponding steel area contributions. The beam bottom bars were discontinued at the face of the column. Steel plates, which were used for the connection at the joint, extended inside the beam at one side and abutted with the column face on the other side. These plates were simulated as 2D plane stress elements. Elements 515-522, located in the left part of the beam adjacent to joint (Fig. 6), and their counterparts on the right part of beam were modelled as plate 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 was filled up after the connections were fastened. Four rows of 2D elements (i.e., elements 463, 464, 465, 466 etc.) at the bottom of the joint were treated as being connected by the steel plates and their equivalent area was transferred to the column main bars and transverse links. The area of truss elements of the region was appropriately increased to account for the effect of the flange of the steel I-section embedded inside the column for Specimens M2 and M4, and the wall of box-section for Specimen M3. The proportionate enhancement of these parts was necessary as they were perpendicular to the 2D analysis direction. The enhanced areas were approximately equal to the web area of steel I-section (UC 254  $\times$  254  $\times$  73) and wall area of SHS  $(300 \times 300 \times 10 \text{ mm})$ , respectively. During the FE investigations, the thickness of the beam connection steel plate was varied to study its influence on the energy dissipation and strength of the joints. Although the 2D model assumption of treating the plate area equivalent to the truss element is approximate, it has fairly validated the behaviour of the joints. Further improvements in the modelling can be tried with the options of solid, plane stress and truss elements, respectively, for concrete, steel plates and reinforcing bars.

#### 4.2. Load-displacement responses of specimens

The predicted and observed responses of the specimens are presented in Fig. 7. From Fig. 8(a) of Specimen M1 it can be seen that the analytical model seemed to have predicted a good response with respect to the experimental observations. Although the displacements of the analytical model for a few initial cycles were slightly higher, the later cycles' results predicted were in good agreement with the experimental counterparts. Specimen M1 achieved a displacement ductility factor (DF) of about 3.2 and pinching was observed in the loops. The loops were thin and quite similar to the experimental results. Global deformation of the specimen's joint core corresponding to a DF of 1.5 is given in Fig. 9(a). A large deformation of the joint core was observed at this stage. Fig. 7(b) shows the analytical and experimental results comparison for Specimen M2. From the experimental results it was seen that the specimen noticed a large initial displacement for many cycles. The specimen achieved good energy dissipation till a DF of approximately of 3.6. The global deformed shape of the specimen corresponding to a DF of 1.5 is given in Fig. 9(b). A moderate deformation of the joint core, and upper and lower parts of the column was seen from the figure. Specimen M3 reached a DF of approximately 3.0, slightly lesser when compared to its experimental values. Although the experimental loops showed large initial displacements their analytical counterparts always depicted steady displacements throughout. This may be due to the fact that the connections might have had some initial gaps in the plates, where the nuts and bolts were fastened, which might have slipped after the application of load leading to large initial displacements. The highest story shear carried by different loops in the experimental and the analytical results showed a good agreement. Similar to Specimen M2, a moderate deformation of the joint core was noticed when a DF value of 1.5 was reached, as shown in Fig. 9(c). Specimen M4 absorbed less energy showing a less number of cycles before failure (Fig. 7(d)). Its initial loops were similar to other specimens,



(c) Specimen M3.

Fig. 7. Predicted story shear forces versus horizontal displacements.

but the last two loops were slightly fatter showing a higher level of energy dissipation compared with other specimens. But the highest horizontal displacement noticed was around 60 mm which was smaller when compared to other specimens. The specimen showed a good energy dissipation and strength. From Fig. 7(d) it can be seen that the finite element model seemed to have predicted well the experimental observations. Fig. 9(d)shows the global deformation of the specimen at a DF of 1.5. Comparison of predicted story shear forces versus the ductility factors for different specimens is presented in Fig. 8. A good correlation was also observed between the analytical and the experimental story shears.

Fig. 10 shows the major principal strain distribution for Specimen M2 at different ductility factors. It was observed that the maximum strains were concentred at the joint core and the connection plates. Although the intensity of the strain and its distribution varied for different DFs, it always spread from the corners thereby showing a large deformation of the joint in shear. As the DF was increased from 1.5 to 3.0, the elements across the diagonal of the joint and the adjacent plate regions

witnessed a high strain leading to large deformations. It was also observed that the specimen witnessed extensive cracking 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. Similar trends were observed in other specimens.

#### 4.3. Discussion of results

Comparison of the analytical and experimental results of all the specimens showed that the lateral load-displacement hysteresis loops obtained from the FE analyses were quite similar to the experimental observations. Besides, the failure modes and the ultimate ductility capacities correlated well with the experimental results. The FE analyses also showed that results of the deformations and cracking patterns matched well with the experimental observations. From the aforementioned observations and predictions of both the global and local behaviours using the FE analysis, the use of FE modelling



(c) Specimen M3.

(d) Specimen M4.

Fig. 9. Deformed shapes of the specimens.



Fig. 10. Principal strain distributions of Specimen M3 at different DFs.

techniques can, therefore, be further extended to study the joint performance by varying different parameters.

# 5. Parametric studies

# 5.1. General

To further improve the understanding of the structural response with hybrid connections, the finite element modelling technique was applied by varying critical influencing parameters such as the axial load, the connection plate thickness and the continuity of beam bottom reinforcement. The following sections present the key parametric investigations and their implications on code provisions.

# 5.2. Influence of axial loads on behaviour of beam-column joints

Axial loading is a critical parameter in the studies of beam-column joints, but its effect on seismic behaviour of beam-column joints has not been fully understood. Previous investigations have shown that axial force is beneficial to the joint shear resistance [13]. 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 [14]. Pessiki et al. [15] experimentally investigated two nonductile 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. [16], it was pointed out that if the shear was small, the increase of axial loads was favourable to the joints, whereas for high shears, the increase of axial loads was unfavourable. Li et al. [17] 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 for deep wall-like column joints.

In this study, the influence of axial loading on the seismic behaviour of hybrid-steel concrete joints is investigated. The specimens were analysed using the DIANA package by applying the axial loads. The same loading histories as those used in the analysis of specimens without axial loading were applied, and the story shear force versus horizontal displacement plots corresponding to different axial load levels were plotted for Specimens M2-M4 (Figs. 11-13). From Fig. 11 it can be seen that Specimen M2 attained an optimum value of ultimate story shear when axial load ratio was  $N^*/A_g f'_c = 0.3$ . A further increase in axial load decreased story shear force and the ultimate number of cycles reached by the specimens also reduced. Similar trends were observed for Specimens M3 and M4 (Figs. 12 and 13), with the reduction in story shear and ultimate number of cycles after enhancement in axial load ratio beyond 0.3 (i.e.,  $N^*/A_g f'_c >$ 0.3). Therefore, the analysis results suggested that the axial load ratio  $N^*/A_g f'_c \le 0.3$  was beneficial to the joint's performance. However, the axial load ratio  $N^*/A_g f'_c > 0.3$  was found to be detrimental as it reduces the story shear and energy dissipation of the joint.



Fig. 11. Load-displacement predictions under different axial loading levels for Specimen M2.



Fig. 12. Load-displacement predictions under different axial loading levels for Specimen M3.



Fig. 13. Load-displacement predictions under different axial loading levels for Specimen M4.

### 5.3. Influence of connection plate thickness

Connection plates adjacent to the joint play a key role in transferring the moment and shear between the column and beams and hence, its successful design is very important. In the experimental investigations 10 mm thick plate was used for the connections. In this study, the connection plate thickness was varied and its effect on energy dissipation was studied. Figs. 14-16 show the predictions of story shears of the specimens for different thickness of connection plates. As seen from Fig. 14, Specimen M2 showed an increase in story shears by 4%, 7% and 11% when the plate thicknesses are changed to 12 mm, 14 mm and 16 mm plates, respectively. The Specimen also achieved an improvement in energy dissipation up to 14 mm of plate thicknesses. However, when the plate thickness was enhanced to 16 mm, though the specimen initially carried a higher value of story shears, the ultimate number of cycles was reduced. One of the reasons for the reduction in number of cycles attained was the due to the specimen failure at other parts. Despite the fact that the concrete in the front and back regions of connection plate was neglected in the analysis, satisfactory energy dissipation was observed. This clearly indicates that the concrete which was filled after connecting the joint plates was not effective in resisting the stresses. Moreover, the bond between the connection plates and precast elements may not have been perfect, and/or might have further reduced during the initial few cycles of loading. Replacement of the top corner plate elements adjacent to the joint region by truss elements for the reinforcements and 2D elements for concrete was also found to be satisfactory in energy dissipation. It was observed that enhancement of plate thickness not only increased the energy dissipation, it also helped in smooth distribution and reduction of the maximum principal stresses. Specimen M3 also showed similar trends when the plate thickness was varied (Fig. 15). The increase in story shears by 3%, 8% and 11% was noticed when the plate thickness was enhanced to 12 mm, 14 mm and 16 mm, respectively. There was no substantial increase in energy dissipation with the enhancement of plate thickness beyond 16 mm, though a few initial cycles showed higher story shears. Similar trend was also seen Specimen M4 (Fig. 16) with the exceptions that the effect of energy dissipation and strength enhancement almost ceased at 14 mm plate thickness. From the aforementioned comparison, it is clear that with the ductility level of the joint remaining the same, a better energy dissipation and higher ultimate strength was observed in the specimens with the increase in connection plate thickness. The specimens showed an optimum benefit of around 11% strength enhancement when the plate thickness was 14 mm.

#### 5.4. Influence of beam bottom reinforcement continuity

In the experimental study of hybrid-steel concrete beamcolumn connections, the precast beams and the columns were connected by plates, while the reinforcement at the beam bottom remained discontinued. Because of reinforcement discontinuity, higher stress levels were seen at the lower part



Fig. 14. Effect of connection plate thickness for Specimen M2.



Fig. 15. Effect of connection plate thickness for Specimen M3.



Fig. 16. Effect of connection plate thickness for Specimen M4.

of plate elements. This was obvious due to the reduction in the lever arm and the neutral axis depth. At high DFs, a large deformation of bottom plate elements followed by yielding was noticed in the analyses. In order to avoid the plate failure, which substantially reduces the flexural capacity of the beam, the effect beam bottom reinforcement continuity was investigated.



Fig. 17. Influence of beam bottom reinforcement below connection plates for Specimen M2.



Fig. 18. Influence of beam bottom reinforcement below connection plates for Specimen M3.

The continuity of bottom reinforcement was maintained by extending the truss elements of the beam bottom reinforcement. Figs. 17–19 show the load–displacement plots of the specimens with variation in reinforcement from 0.5% to 1% of the gross area  $A_g$ . It may be noted that the steel percentages varied in this study were greater than the minimum longitudinal reinforcement for beams specified by NZS 3101 [14]:

$$\rho_{\min} = \frac{\sqrt{f_c'}}{4f_v} \times 100. \tag{3}$$

As seen from Fig. 17, Specimen M2 witnessed a hike in story shears approximately by 4% and 9%, 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 enhanced by 1% of  $A_g$ . Steady energy dissipation in the hysteresis loops was also seen with reinforcement continuity. Figs. 18 and 19 show that the story shears of Specimen M3 were enhanced approximately by 4% and 8%, and that for Specimen M4 the increase was around 4% and 7%, when the reinforcement was varied by 0.5% and 0.75% of  $A_g$ , respectively. From the above discussion, it is clear that continuation of beam bottom reinforcement improved the



Fig. 19. Influence of beam bottom reinforcement below connection plates for Specimen M4.

performance of the joints with strength reaching an optimum value of approximately 8%, when the reinforcement value was 0.75% of  $A_g$ . Besides, a smooth stress distribution adjacent to the joint region and higher energy dissipation were observed.

# 6. Conclusions

The hybrid-steel concrete connection for seismic behaviour was studied using the numerical models. Finite element analysis was employed as a numerical tool to investigate the behaviour of joints. Concrete was modelled using 2D elements, whereas truss elements are employed for steel bars. The connection plates are modelled as 2D elements with steel properties and the DIANA software was used as the modelling tool. Comparisons with the experimental results indicated that the finite element models used in this study were suitable, and the corresponding investigation results were reliable. The predicted results matched well with the experimental observations. The connection plate modelled using 2D plate elements and neglecting the concrete on either side of it showed a satisfactory performance in the structural analysis. Based on the parametric study results, the following conclusions can be drawn:

- (1) The FE results showed that axial load was beneficial to the joint's performance. Axial load ratios  $N^*/A_g f'_c = 0$  to 0.3, influenced energy dissipation and story shears of the joints adding in a better behaviour. However, an axial load ratio beyond  $N^*/A_g f'_c > 0.3$  was detrimental to the joint's performance.
- (2) Connecting plate thickness at joint influenced the energy dissipation and deflections during the cyclic loading. The increase in plate thickness gradually increased the energy dissipation and strength of the joint. With ductility of the joint remaining the same, the specimens showed an optimum benefit of around 11% enhancement in strength followed by better energy dissipation when plate thickness was 14 mm. However, any increase in thickness beyond 14 mm, showed no marked improvement in energy

dissipation, and it also reduced the ultimate number of load cycles attained.

(3) Continuation of beam bottom reinforcement increased the ultimate strength of the specimens and reached an optimum value of approximately 8% when the reinforcement was 0.75% of  $A_g$ . The specimens also showed good energy distribution and smooth stress distribution. It was observed form the FE analysis that beyond 1% of  $A_g$ , the advantage almost ceased with no further enhancement in strength.

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