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Parametric investigation of 3D RC beam–column joint mechanics

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This paper presents the experimental and numerical findings of interior and exterior beam–column–slab joints under cyclic lateral loading in two orthogonal directions. Two full-scale interior and one exterior beam–column joint assemblies incorporating floor slabs were subjected to quasi-static cyclic loading and zero column axial loads. The test results from these three models, designed according to building code provisions for ductile moment-resisting frames, were satisfactory in terms of strength and ductility capacity. Parametric studies via the non-linear finite-element approach were performed to study the influence of various parameters on the strength and ductility of three-dimensional beam–column joints. The study confirmed the beneficial effects of incorporating floor slabs and transverse spandrel beams on the behaviour of beam–column joints. The presence of axial compressive load improved the joint shear capacity of the beam–column joints, but a threshold limit should be applied.

Notation

b_b	width of transverse beam
b_c	width of column
c	cohesion
f_t	tensile strength of concrete
h_s	slab depth
K_{test}	experimental stiffness
m_A	slab moments
P_i	theoretical downward force
T_{Xe}	membrane forces
V_i	theoretical ideal strength
V_s	storey shear
V_{sc}	storey shear when the width of the transverse beam equals the column width
ΔP	load applied to the beam ends
ΔV_s	increase in the storey shear force
$\Delta_{y,test}$	first yield displacement
μ	displacement ductility factor
φ	angle of internal friction

Introduction

The beam–column joint is one of the most critical regions of a structure when considering seismic-resistant design in moment-resisting frames. Traditionally, engineers have placed great emphasis on the design and detailing of beams and columns, while joint failures are an area of more recent concern.

Beam–column joints in moment-resisting frames are subjected to large shear forces due to lateral earthquake forces (ACI, 2008). The majority of the early studies were done on planar frames (Hanson and Conner, 1967), while some researchers (Becking-

sale, 1980) found that the performance of space frames is inferior to that of planar frames. As a result, studies (Durrani and Zerbe, 1987; French and Boroojerdi, 1989; Leon and Jirsa, 1986) began to include the effects of floor slabs and transverse beams to simulate the response of frame structures more realistically. It was found that, due to the confinement effect, the presence of transverse beams will limit the joint shear cracks and pull-out of the longitudinal beam bars (Ehsani and Wight, 1982). Other researchers (Paultre *et al.*, 1989; Rattray, 1986) concluded that the slab contribution increases the beam strength, which will lead to weak columns and strong beams, and thus alter the failure mode of the structure to a less ductile one.

Most beam–column joint tests have been done for the case of two-dimensional unidirectional loading, and the behaviour of three-dimensional (3D) joints is not fully understood.

Due to difficulties in applying column axial loads, it is common practice to ignore the effect of compressive axial loads in most of the investigations. In-depth information on the influence of other critical parameters, such as the slab presence and transverse spandrel beams, is also limited. Therefore, the first part of these study experimental investigations was undertaken, and in the second part the finite-element (FE) models were validated and parametric studies performed.

Description of test programme

Three full-scale reinforced concrete (RC) beam–column joint subassemblies, including transverse beams and the floor slab, were constructed and tested. The subassemblies were designed and detailed in accordance with the requirements of NZS 3101.1

(Standards New Zealand, 2006). Model 1D-I was a replica of a typical interior joint in a 3D one-way frame, and models 2D-I and 2D-E represented interior and exterior joints in a two-way frame. The test models were designed so as to simulate the full-scale joint subassembly in a frame with a beam span of 6 m and a storey height of 3.5 m.

All the models were 3D and consisted of the top and bottom columns, main beams and a floor slab. Models 2D-I and 2D-E included two transverse beams in the north–south direction. The main and transverse beams were concentrically connected to the column in all models.

Table 1 summarises the details of the test models, and Figures 1 and 2 show the reinforcement details and dimensions of all the models.

Material properties

The design compressive strength of the concrete in all the models was 30 MPa and the maximum aggregate size was 20 mm.

Grade 380 deformed steel bars (HD20, HD24 and HD28) with measured yield strengths ranging from 432 to 500 MPa were used as longitudinal reinforcement in the column section, while the longitudinal main steel in the beam sections consisted of grade 275 D24 and D20 bars of measured yield strengths of 283 and 300 MPa, respectively. The main reinforcing steel in the slab, which was included to increase the beam flexural strength at the joint, was grade 275 D10 bars of measured yield strength 326 MPa. The transverse shear reinforcement provided was grade 275 plain round bars of 10, 12 and 16 mm diameter. The steel properties are summarised in Table 2.

Test set-up

The loading rig (Figure 1(e)) was designed to allow unidirectional or bidirectional simulated seismic forces to be applied to the test models. Pins were provided at the top and bottom of the concrete column in order to enable the two ends to rotate in two perpendicular directions. The beam ends were also able to rotate and move laterally in the plane of the set-up frame. Two double-acting jacks were kept in the vertical direction at the beam ends in order to apply forces upwards or downwards.

The positive loading direction was determined as the force that would cause a counterclockwise rotation in the beams and a clockwise rotation in the column, as shown schematically in Figure 4(a).

Loading arrangement

Each model was subjected to quasi-static reversed cyclic loading. Model 1D-I was subjected to unidirectional loading, while bidirectional loading was imposed on models 2D-I and 2D-E, as depicted in Figure 3. The lateral force of about half of the theoretical ideal strength V_i was applied to the models in the first two load cycles. In the first half of the third load cycle, the first yield displacement $\Delta_{y, \text{test}}$ and the experimental stiffness K_{test} were determined based on the measured displacement at 75% of V_i and a linear extrapolation to V_i . Subsequent displacement-controlled cycles were imposed in the manner of increasing displacement ductility factor $\mu = \Delta/\Delta_y$. The models were tested under no axial load. The proposed bidirectional loading method permitted observations to be made on the behaviour of models in two separate directions. During the application of bidirectional loading, the drift angle in one direction was kept constant while the model was displaced in the orthogonal direction.

Member	Property	Model		
		1D-I	2D-I	2D-E
Slab	Thickness: mm	100	130	130
	East–west	Top bar, ρ : %	0.224	0.232
	North–south	Bottom bar, ρ' : %	0.224	0.252
		Top bar, ρ : %	0.447	0.377
		Bottom bar, ρ' : %	0.223	0.252
East–west beam	Size: mm	400 × 550	400 × 550	400 × 550
	Top bar, ρ : %	1.34	1.58	1.34
	Bottom bar, ρ' : %	0.77	0.77	0.77
North–south beam	Size: mm	None	400 × 575	300 × 575
	Top bar, ρ : %	None	1.57	1.3
	Bottom bar, ρ' : %	None	0.73	1.03
Column	Size: mm	600 × 550	600 × 600	550 × 500
	ρ : %	1.48	2.05	2.69
Joint	ρ : %	1.86	1.71	1.31

Table 1. Details of the models

Figure 1. Configuration and details of model 1D-I and the loading rig: (a) plan view; (b) dimensions and details of east-west elevation; (c) column section; (d) section 1-1; (e) loading rig

Figure 2. Configuration and details of (a) model 2D-I and (b) model 2D-E

Property	Grade 275					Grade 380				
Bar size	R10	R12	D10	R16	D16	D20	D24	HD20	HD24	HD28
Yield strength, f_y : MPa	315	320	326	330	318	300	283	482	500	432
Yield strain, ϵ_y	0.0014	0.0013	0.0018	0.0015	0.0014	0.0013	0.0013	0.0024	0.0024	0.0018
Ultimate strength, f_u : MPa	432	466	441	503	482	459	437	650	669	602

R10, plain round bar, 10 mm diameter; D20, deformed bar, 20 mm diameter; HD20, deformed high-strength bar, 20 mm diameter.

Table 2. Measured properties of the reinforcing steel in the test models

Experimental results

Model 1D-I

The storey shear versus horizontal displacement for model 1D-I is shown in Figure 4(a). The hysteresis loops show stable energy dissipation. The slight pinching in the early loading cycles reflects the major contribution of the beams to the overall behaviour of the test model. As observed during the test, the average stiffness in the positive (i.e. upward) bending case was usually higher than the negative (i.e. downward) bending stiffness, for both beams.

During upward loading, the response curves showed greater initial stiffness. The contribution of the top concrete, which is in compression, in flexural resistance of the structure was observed to be effective during the early stage of each load run.

Under downward bending, the peak strengths were higher in both beams due to the large number of additional slab tensile steel bars. The maximum strength attained at a ductility $\mu = 8$ for the west beam was 45% higher than the theoretical downward force $(-P_i)$.

Model 2D-I

Model 2D-I performed satisfactorily in terms of maintaining strength and ductility capacity, although its behaviour was not as good as for model 1D-I. The overall test observations were similar to those noted for model 1D-I – such as the formation of plastic hinges in beams at column faces, the formation of fine flexural cracks in columns, indicating that the columns remain in the elastic range, and the propagation of cracks in the floor slab.

The displacement corresponding to the first yield $\Delta_{y,\text{test}}$ was 16.5 mm, which is slightly higher than the yield displacement of 15.7 mm for model 1D-I. However, the experimental stiffness was 14.1 kN/mm, which is only 82.5% of the theoretical stiffness of 17.1 kN/mm.

The column shear versus displacement hysteresis loops for model 2D-I are plotted in Figure 4(a). Although there was a gradual degradation of stiffness, the storey shear remained constant and

the energy dissipation was stable. The inelastic response of this model was considered satisfactory. However, the comparison of models 1D-I and 2D-I clarifies that identically reinforced beam–column joints under one-way action would perform better than under two-way actions.

Model 2D-E

Plastic hinges formed at the end of three beams facing the column. Spalling and crushing of concrete was observed after a ductility $\mu = 8$, while bar buckling took place at ductility $\mu = 11$.

The column shear versus displacement for model 2D-E is plotted in Figure 4(a). Most of the features of the hysteresis responses are identical to those for model 2D-I. The first yield displacement $\Delta_{y,\text{test}}$ was determined to be 12.1 mm, while the theoretical stiffness was 9.5 kN/mm. The observed hysteresis response of the test model indicates that the storey shear forces kept increasing, except at the last stage of the loading.

The top and bottom bars in the east beam were anchored in the joint core by 90° standard hooks. The pattern of strain distributions (Figure 4(b)) confirms the spreading of inelastic tensile strains in the steel bars from the plastic hinge region towards the free end of the beam. In addition, the strain pattern showed that the embedment length was adequate to anchor the beam bars.

Finite-element analysis

General

The following sections present a 3D non-linear FE numerical investigation carried out on RC beam–column–slab joints in order to further enhance the understanding of the complex behaviour of the structural parts. As all the models were tested without any axial load applied on top of the column, the influence of this parameter remained inconclusive. In addition, it was impractical and uneconomical to investigate the effects of other parameters, such as the floor slab, transverse beams and the role of bidirectional loading, by means of experimental observations.

Figure 3. Quasi-static cyclic loading history: (a) model 1D-I, unidirectional loading; (b) model 2D-I, north–south loading; (c) model 2D-I, east–west loading; (d) model 2D-E, east–west loading; (e) model 2D-E, north–south loading

Figure 4. Verification of FE models. Comparison of hysteretic behaviour between the experimental and the finite-element analysis (FEA) results: (a) model 2D-I, east-west direction; (b) model 2D-I, north-south direction; (c) model 2D-E, east-west

direction; (d) model 2D-E, north-south direction; (e) model 1D-I. Strain profile of: (f) model 2D-I, north-south beam bar; (g) model 2D-E, east-west beam bar

Material modelling

Modelling of concrete

In order to simulate the characteristics and behaviour of the concrete material, the model should be comprehensive and consist of the concrete cracking behaviour, compression hardening and softening, the behaviour in tension, shear behaviour and lateral confinement influence (Okamura and Maekawa, 1991).

In the analysis a constant stress cut-off criterion for concrete cracking was used. The response of the concrete in compression was modelled as an elastic–plastic behaviour. The elastic state of the stress was limited by the Drucker–Prager yield surface, while isotropic hardening with an associated flow rule was used after the surface yielding. In the non-linear analysis program Diana (2012), the yield surface is evaluated using the current state of stress, the angle of internal friction ϕ and the cohesion c . A Poisson ratio of 0.15 was used in the analysis.

A linear tension softening curve was used to simulate the softening effect of the concrete in tension after cracking. The tensile strength of concrete f_t was calculated according to the *CEB-FIP Model Code 1990* (CEB-FIP, 2008) as

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

where f_c is the concrete compressive strength.

When the cracked concrete was unloaded in tension, the secant modulus was used to evaluate the stiffness; when the concrete was unloaded in compression, the initial stiffness was adopted for the stiffness calculations.

Modelling of reinforcement

The Von Mises yield criterion with isotropic strain hardening was used to characterise the constitutive behaviour of the reinforcement. The longitudinal bars in beams and columns were modelled as separate truss elements whereas other steel reinforcements were modelled as embedded bar elements in 3D solid elements. The available interface elements in the Diana library were used to connect the reinforcement (truss elements) to the original concrete elements. The bond element between the concrete and reinforcement is an interface element between a quadratic line and a quadratic brick solid element in 3D configuration. In the formulation, the concrete is treated as a 3D continuum element, while the truss and bond elements are assumed to be of constant strain and constant slip, respectively. The bond law used in the analysis is based on *CEB-FIP Model Code 1990* (CEB-FIP, 2008).

Geometry modelling

The Diana software was used for the FEA. Twenty-node 3D quadratic solid brick elements were used for the concrete, while the reinforcing bars were modelled using truss elements. The FE discretisation of models 1D-I, 2D-I and 2D-E is presented in Figure 7(a) – Type 2, 1 and 5 respectively.

Verification of FEA results

The results from the FEA were compared to those obtained from the experiment for the verification purpose in Figure 4. The comparisons show that the maximum storey shears in hysteretic loops are close to theoretical storey shear strengths in different loading cycles. As shown in Figures 4(a) to 4(e), for model 1D-I a few initial cycles of the FE simulation predicted storey shears slightly lower than those of the experimental hysteresis loop. In general, the comparison of analytical and experimental results showed that the lateral load–displacement hysteresis loops were similar for all models.

In order to verify the local behaviour of numerical models, the experimentally obtained strain profiles and the predicted values of the strain obtained in the FEA were compared. As illustrated in Figures 4(f) and 4(g) for the top layer beam bars, there was good correlation between the strain values.

Based on the well-established and verified models, further parametric study can be performed by varying the critical parameters.

Parametric study

After verifying all the numerical models against the experimental results, an extensive parametric investigation was performed to gain more information about the seismic behaviour of the 3D beam–column–slab joints. The following sections describe the application of the FE modelling technique to the investigation of the influence of the critical parameters, such as the bidirectional loading, axial load level, and the presence of a floor slab and transverse beams.

Influence of column axial load

Although analytical research has highlighted the important role of the axial load, previous experimental investigations on 3D beam–column joints have not considered the effect of this load on joint performance (Kurose, 1988; Leon and Jirsa, 1986; Shin and LaFave, 2004). However, some researchers (Li *et al.*, 2009) have concluded that the optimum enhancement in the storey shear occurs at an axial load level of $0.25f_c'A_g$.

In the present study, the same bidirectional loading history as applied in the experimental tests on the models was applied in FE simulations. The applied column axial load varied from $0.1f_c'A_g$ to $0.4f_c'A_g$. As observed, the storey shears of model 2D-E increased by around 3–5%, as the axial load was increased to $0.2f_c'A_g$. However, any further increase in the axial load reduces the storey shear. A similar trend was observed for the interior joint models 2D-I and 1D-I, for which the storey shear increased by around 5% and 7%, respectively, for an axial load of $0.2f_c'A_g$. Figure 5(a) shows the storey shears plotted against horizontal displacements for different levels of applied axial load on model 2D-I, and Figures 5(b) to 5(d) show the skeleton curve for storey shears plotted against horizontal displacements for the interior model 2D-I. It can be seen that the energy dissipation capacity of the model increases as the axial load increases up to $0.3f_c'A_g$.

Figure 5. Influence of the axial load as determined in the FEA.
(a) The global behaviour of model 2D-I. Skeleton curves for: (b) model 2D-E; (c) model 2D-E without slab; (d) east-west loading of model 2D-I

In addition, the effect of axial loads on the behaviour of models without floor slabs was also investigated using FE simulations. Figures 5(b) to 5(d) show the storey shear plotted against horizontal displacement for model 2D-E and the same model without a floor slab. It was found that increasing the axial load up to $0.2f'_c A_g$ increased the storey shear by around 8%. Further increase in the axial load resulted in the joint maximum shear reduction, similar to the observed trend in the models with a floor slab.

The preceding discussion clearly shows that an axial load $0.2f'_c A_g$ will cause an optimal enhancement in strength of the models, with or without a floor slab.

Influence of bidirectional loading

Engindeniz (2008) concluded that the joint shear demand in exterior beam-column joints subjected to bidirectional loading is significantly higher than that predicted by unidirectional models. However, the 3D failure process has not been studied previously.

As described above, the strength and stiffness of the models reduced when bidirectional loading was applied, due to changes in the contribution of the slab reinforcements. In the current FE study, the behaviour of the models was investigated under unidirectional and bidirectional loading. The bidirectional loading sequence is the same as the one applied to models 2D-I and 2D-E, as explained previously. For unidirectional loading, only the loading in the direction of the main beam was applied in the model, with the displacements in the orthogonal directions omitted. Figure 6(a) shows the skeleton curve of storey shears plotted against horizontal displacements for the east-west beams in model 2D-I under two different loading scenarios. The hysteresis loops for the model 2D-I and 2D-E joints are presented in Figures 6(b) to 6(e) to compare the global behaviour of interior and exterior joints while subjected to unidirectional or bidirectional loading in the direction of the main beam. The results of the FEA show that applying bidirectional loading reduced the storey shear by around 5% and 7% in the exterior and interior beam-column-slab joints, respectively. In addition, the energy

Figure 6. Influence of the loading scenario (bidirectional or unidirectional loading) in the FEA. (a) Skeleton curve for north–south direction of model 2D-I. Global behaviour

of interior and exterior joints: (b) model 2D-I, bidirectional load; (c) model 2D-I, unidirectional load; (d) model 2D-E, bidirectional load; (e) model 2D-E, unidirectional load

dissipation capacity was considerably higher in the models subjected to unidirectional loading.

Influence of slab presence

The effect of slab presence on the load-transfer mechanism and the overall behaviour of beam–column joints has been the subject of several studies (Boroojerdi, 1986; Joglekar, 1985; Rattray, 1986). Some researchers (French and Moehle, 1991; Leon and Jirsa, 1986) concluded that the presence of a slab may increase the positive moment capacity of the system considerably, in addition to increasing the negative moment capacity due to the incorporation of slab-top reinforcements.

Pantazopoulou and French (2001) illustrated that the increase in strength of the structure is proportional to the force developed in

the longitudinal slab reinforcement within the effective slab width.

Figure 7(a) shows the FE models for interior and exterior joints that were used for the present parametric study. The FE models included the experimental models 2D-I and 2D-E and other combinations resulting from removing either the slab or the transverse beams. Figures 7(b) to 7(d) show the parametric study storey shears plotted against horizontal displacement results for all the FE models. FEAs of the interior joint models showed a 23% increase in the maximum storey shear in the positive loading direction and a 40% increase in the negative direction when the slab was included in the model, while in the case of exterior joints the increase in the storey shear was around 12% in the positive loading direction and 22% in the negative direction.

Figure 7. (a) The 3D FE models used in the parametric study. Results of the FEA: (b) interior joints; (c) exterior joints; (d) one-way models

These results are comparable with those found in previous studies on the behaviour of complete frames, which showed a strength difference of the order of 38–40% when slab participation was neglected (Pantazopoulou and French, 2001; Shahrooz and Moehele, 1987; US–Japan Research, 1988). The effect of the slab on the behaviour of interior models without transverse spandrel beams is presented in Figures 7(b) to 7(d). It can be seen that the presence of the slab increased the initial stiffness of the model significantly. In addition, the maximum shear capacity was increased by about 17% in the model with slab.

In order to investigate the effect of slab depth on the behaviour of the structure, several FE models, including various slab depths ranging from 100 to 180 mm, were studied (Figure 8(a)). The slab reinforcement ratio was kept constant in all the FE models. The global behaviour of these models was compared with that of the model with no slab (Figure 8(b)). It can be seen that, in general, models with a thicker slab had a higher storey shear force, and the energy dissipation capacity reduced as the slab depth increased. The relationship between the enhancement of storey shear and slab depth in interior and exterior joints is illustrated in Figure 8(c). The storey shear increase in interior joints varied between 30% and 50% in the positive loading direction and between 8% and 35% in the negative loading direction. The storey shear increase in exterior joints ranged from 0.5% to 19% in the positive loading direction and from 11% to 18% in the negative loading direction. The relationship between the average storey shear and the horizontal displacement for the positive and negative loading directions is illustrated in Figure 8(c). The following equations are proposed for finding the increase in storey shear in the interior and exterior joints due to the presence of a slab

$$\Delta V_s (\%) = 0.177h_s + 1.788;$$

$$100 \text{ mm} \leq h_s \leq 180 \text{ mm}$$

2. (interior joints)

$$\Delta V_s (\%) = 0.103h_s - 4.0;$$

$$100 \text{ mm} \leq h_s \leq 180 \text{ mm}$$

3. (exterior joints)

where ΔV_s is the increase in the storey shear force and h_s is the slab depth.

The parametric study illustrated that the role of slab participation is significantly greater in interior than exterior joints. This is expected because stiffer diaphragms, as in the case of interior joints, will develop larger forces in their plane of action in order to resist the imposed deflections.

Figure 8. Analysis for different slab depths: (a) finite-element models; (b) global behaviour of interior joints; (c) influence of slab depth on joint shear

Influence of spandrel transverse beams

Several studies have been undertaken to investigate the influence of transverse beams on the behaviour of joints (Di Franco, 1993). Hatamoto *et al.* (1991) concluded that increasing transverse beam reinforcement will not enhance the total storey shear force. However, the strength of transverse beams in torsion is critical in exterior beam column joints, as a cracked transverse beam will not be able to transfer slab forces to the joint region, which results in stiffness loss in the model. Thus, in the present study, the effect of different types of spandrel transverse beam was investigated.

It can be clearly observed in Figures 7(b) to 7(d) that when the FE model consisted of a slab with no transverse spandrel beams,

the storey shear decreased by 11% compared with the model having both a floor slab and transverse beams. This is consistent with the slab participation actions, as shown in the free body diagram in Figure 10(b). The slab moments m_A and membrane forces T_{Xe} that are induced by the applied load to the beam ends ΔP are transferred to the joint core region by the mechanism shown in Figure 10(b). Hence, the role of the strength and torsional resistance of transverse beams is critical in order to increase the load-transfer capacity of the beam–column–slab subassembly. If the spandrel beam is absent in the exterior joint, there is no mechanism to transfer the slab forces in the transverse boundary, except in the region of the column support. In that case, the contribution of the slab is not significant outside the effective width, which is determined based on the column size.

As illustrated in Figures 7(b) to 7 (d), the FE investigations indicated that in the bare frame models (with no floor slabs) the presence of transverse beams did not enhance the storey shears dramatically, which confirms the above-mentioned role of transverse beams in forming the load-carrying mechanism.

The FE models shown in Figure 9(a) were analysed, and their behaviour is compared in Figure 9(b). Model E1 is the exterior joint model used in the experiment, while models E2 to E5 have transverse beams with larger widths. Models E6 to E8 have the same beam width as E1, but with larger beam depths. The reinforcement ratio was kept the same in all the models.

The results of the analysis show that increasing the width of the transverse beams resulted in a higher maximum storey shear force, while increasing the depth of the transverse beams did not enhance the strength or stiffness significantly. Figure 9(c) illustrates the influence of the width of the transverse beam on the storey shear capacity of interior and exterior joints. It can be that in an exterior model with a transverse beam width equal to the column width, reducing the beam width by half reduced the storey shears by more than 10% and 20% in the negative and positive loading directions, respectively. The behaviour of interior beam-column joints seems to be irrelevant to the width of transverse beams.

Figure 9. Analysis for different transverse beam geometries:
(a) finite-element models; (b) global behaviour of exterior joints;
(c) influence of transverse beam width on joint shear

The following equation is proposed for relating the ratio of story shears when the transverse beam width is different from the column width in exterior joints

$$4. \quad \frac{V_s}{V_{sc}} = 0.337 \frac{b_b}{b_c} + 0.653; 0.5b_c \leq b_b \leq 1.5b_c$$

where V_s is the storey shear, V_{sc} is the storey shear when the width of transverse beam is equal to the column width, b_b is the transverse beam width and b_c is the column width (the dimension of the column in the face of the transverse beams).

Confinement effect of spandrel beams and floor slabs

Some researchers believe that a one-way exterior beam–column joint with three exposed surfaces is the most vulnerable joint configuration in the case of applied cyclic loading, due to the lack of confinement in the joint region. In order to investigate the confinement effect of floor slabs and transverse beams, the behaviour of FE models when joint hoops are not included was compared. The global hysteresis behaviour of exterior and interior models with no joint hoops is shown in Figure 10(a). It can be seen that the exterior models without transverse beams (types 6 and 8) show the most deficient performance in terms of capacity and ductility, which implies a critical confinement effect of

Figure 10. (a) Comparison of global behaviour of exterior and interior joint models without joint hoops; (b) role of slab flange participation

transverse beams in exterior joints. For a bare frame exterior joint without joint hoops, incorporating either transverse beams or a floor slab will increase the maximum storey shear by around 6.5%, while the presence of both transverse beams and a floor slab will increase the joint strength by 16%. The presence of joint hoops will add another 4.5% to the maximum storey shear force, and will enhance the energy dissipation capacity.

Although the maximum storey shears in the models without transverse beams (type 6) and without a floor slab (type 7) were almost identical, strength degradation was severe in the exterior model without transverse beams and joint hoops.

For interior joints without joint hoops, including transverse beams and a floor slab can improve the maximum storey shear force of the bare frame by around 34%, while including joint hoops will increase the maximum storey shear by another 3%, which shows the significant confinement effect of the transverse beams and floor slab in interior joints.

Conclusions

The seismic behaviour of 3D beam-column joints was evaluated using experimental and numerical approaches. Based on these investigations, the following conclusions are drawn

- (a) The results of the FEA clearly showed the influence of the axial load on the behaviour of 3D joints. This effect was not considered in the experimental programme. It was observed that at an axial load of $0.2f_c'A_g$ exterior joint models experienced an optimum enhancement in the storey shear by 3–5%, while the maximum storey shear force increased by around 5–7% in interior joints.
- (b) It was observed that the effect of the axial load on the models without floor slabs was slightly higher, as the optimum axial load of $0.2f_c'A_g$ increased the storey shear by around 8% in the beam-column joint bare frame model.
- (c) According to the numerical investigations, it was observed that applying 3D cyclic loading reduced the maximum storey shear by around 5% and 7% for exterior and interior joints, respectively, compared with the application of unidirectional loading. The energy dissipation capacity is considerably higher in the models subjected to unidirectional loading.
- (d) The FE simulations showed that including floor slabs in beam-column joint models increased the maximum storey shear by around 25% in interior joints and by around 10% in exterior joints. However, the increase in story shear is linearly related to the slab depth. In addition, including a floor slab increased the initial stiffness of the structure, and this was more significant in one-way models (without spandrel beams).
- (e) The role of transverse beams is critical in exterior beam-column joints modelled with a slab. If transverse beams were present in the exterior model, the maximum storey shear increased by about 12%, showing the vital role of spandrel beams in transferring slab reinforcement tension forces to the

joint core region. The increase in storey shear was also related to the width of spandrel transverse beams. The effect of transverse beams in exterior models without a slab and in interior models was not significant.

- (f) The FEAs demonstrated that the presence of a floor slab and transverse beams in the joints modelled without joint hoops increased the maximum storey shear by 16% and 34% in exterior and interior joints, respectively, due to the beneficial confinement effect. In this case, the joint strength was only 3% and 4.5% smaller than that of the interior and exterior models including joint hoops.

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