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Nonlinear Simulation of Reinforced Concrete Moment Resisting Frames under Earthquakes

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Abstract: ***The main purpose of this paper is to study analytically the behavior of reinforced concrete frames under earthquakesand to develop a nonlinear finite element beam-column joint model and a moment resistingframe model. The analytical frame model was implemented using the commercial software SeismoStruct. All details of the reinforced concrete beam-column joint model used in the analysis were described and verified with the available experimental test results conducted by other researchers. To check the accuracy of the reinforced concrete frame model, it was verified with the dynamic time history test of three-bays, four-stories frame structure. The results proved that both beam-column joint and the moment resisting frame models can simulate the seismic behavior with accepted accuracy.***

Keywords: Beam-column joint, reinforced concrete, moment resisting frames, nonlinear analysis, finite element model

1. Introduction

Reinforced concrete moment resisting frame structures might fail by lateral instability caused by extreme earthquakes.the beam-column joint is the transmitter of action between a beam and a column and is one of the main characteristics of the reinforced concrete frame structures as it could transferdifferent actions across members and due to its critical behavior affecting the local and global behavior of a structure. The main reason of failure in the beam-column joist is shear, so it is too important to evaluate the shear capacity of the beam-column joint subjected to reversed loads. Many analytical and experimental investigations were conducted to study the behavior of the reinforced concrete beam-column joints and moment resisting frame structures. Yu [1]investigated experimentally five reinforced concrete interior and exterior beam-column joints under reversed horizontal cyclic loads. The results of this investigation were compared with the analytical joint model introduced by Yu [1]. This analytical model followed the numerical model presented by lowes et al. [2]- [3]. Pinho and Elnashai [4] presented apseudo-dynamic experimental test for a full scale tworeinforced concrete frame. The moment resisting frame consisted of three-bays and four-stories.Ali et al. [5] conducted an experimental investigation for one-bay, two stories reinforced concrete frame with one third scale. Also, an analytical model and approach were presented in his investigation. On the other hand, several analytical models were presented in recent decades using the analytical model presented by Lowes et. [2]- [3] for the analysis of beam-column joints such as Alam et al. [6], Fernandes et al. [7], Pan et al. [8], and for reinforced concrete frames like Naderpour and Mirrashid [9].

The main objective of this paper is to present a nonlinear beam-column joint model and a moment resisting frame model subjected to reversed cyclic loads using the commercial software SeismoStruct. To check the accuracy of the analytical models, the beam-column joint model and the moment resisting frame model were verified with the experiential results conducted by Yu [1] and Pinho and Elnashai [4], respectively.

2. Finite Element Model Component

The main important of the analysis of any analytical model is selecting and defining the elements and the components of the analytical model. This analytical study focused on development a nonlinearfinite element model using the commercial software SeismoStruct [10]for modelling to simulate theavailable tested beam-column joints and moment resisting frames. Different types of springs can be modelled by defining a link elementin SeismoStruct software, which is a zero-length element has a structural node at each end to define the force displacement or moment rotation relationship. To simulate the rotation of both beam and column cross-sections due to steel bar bond-slip, a bar-slip rotational springs located at the interface of beam-column joint region was used, as shown in Fig. 1. Also, a shear spring wasadded at the same locationto introduce the shear transfer. To introduce the inelastic shear response of the beam-column joint, a rotational spring was placed in the centerline of beam-column joint. For modelling beams and columns inside and outside the panel zone,the rigid elastic frame elements and the inelastic force-based frame elements were selected, respectively. In order to describe the mechanical properties of the material models of concrete and steel, the concrete model proposed by Mander et al. [11]and the uniaxial steel model introduced by Menegotto and Pinto model [12] were selected.Also, to define the unload-reload path and representing the degradation of the stiffness and strength during the cyclic loading,a response curve from the several models included in SeismoStruct were selected. In this analytical study the considered constitutive curves were:

- The modified Richard-Abbott curve, to present the moment rotation relationship of the rotational spring due to reinforcing bar slip at beam and column cross-section. This curve was introduced by Richard and Abbott [13] to predict a simulation curve for the response of connection subjected to monotonic loads. It was updated and modified to simulate cyclic loading and to include the pinching effect by Della Corte et al. [14] and Nogueiro et al. [15].

- The modified bilinear Takeda curve, to introduce the relation between force and displacement of the interface shear springs, and the moment rotation relationship of the rotational spring at of the joint core region. This model programmed based on the bilinear simplification of the original trilinear model of Takeda et al. [16] beside the hysteresis loop introduced by Otani [17], and the unloading path presented by Emori and Schonobrich [18].

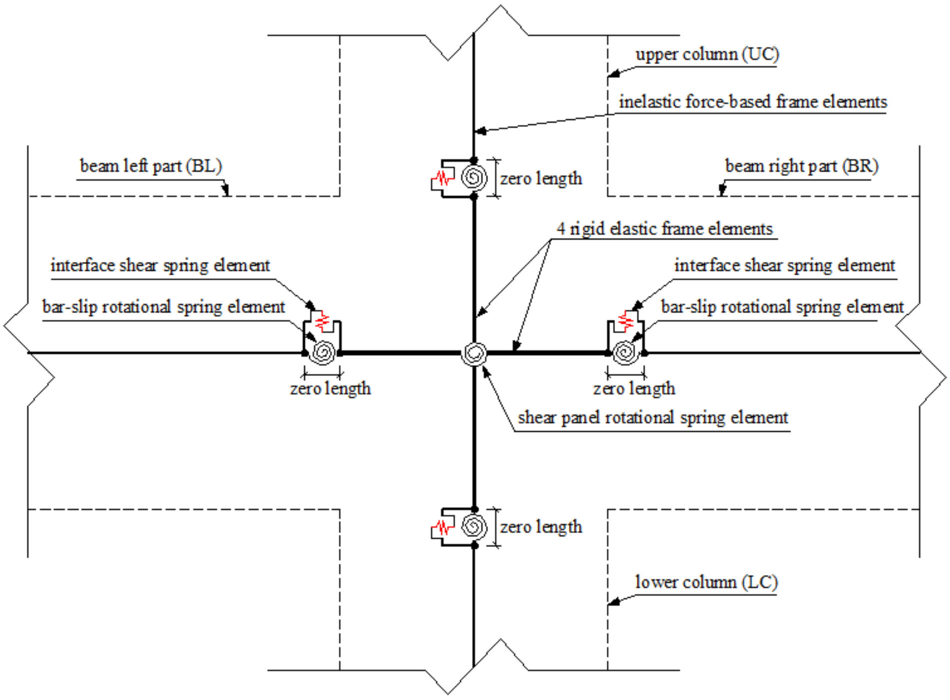

Figure 1: Details of the proposed RC beam-column joint model

2.1. Bar-Slip Spring

The bond slip mechanism refers to the movement of the beams and columns longitudinal steel reinforcement embedded in joint zone due to the bond strength deterioration of these bars. As stated in the previous research,the bond strength is a function of the material state of the anchored bar as well as of the concrete and transverse reinforcing steel in the vicinity of the reinforcing bar. To define the bar-slip springs, the bilinear moment rotation curve is defined by the critical points of yield and ultimate conditions (M_y, θ_y) and (M_u, θ_u). This moment rotation relationship was derived based on the bar stress and slip relationship by Eq. (1) introduced by Lowes et al. [3]. The average bond-strength values are listed in Table 1, based on the experimental data provided by Eligehausen [19]. The yielding (M_y) and the ultimate moments (M_u) were developed from the equilibrium between the tensile and compressive forces in the cross-section, which were calculatedusing a rectangular concrete stress block and the actual stress-strain relation of reinforcing steel bars. However, the yielding (θ_y) and the ultimate rotations (θ_u) were calculated from Eq. (2).

$$Slip = 2\frac{\tau_E}{E_s}\frac{l_{fs}^2}{d_b} \qquad for\ f_s < f_y \tag{1a}$$

$$Slip = 2\frac{\tau_E}{E_s}\frac{l_e^2}{d_b} + \frac{f_y l_y}{E_s} + 2\frac{\tau_Y}{E_{sh}}\frac{l_y^2}{d_b} \qquad for\ f_s \geq f_y \tag{1b}$$

$$l_{fs} = \frac{f_s}{\tau_{ET}}\frac{A_b}{\pi d_b},\ l_e = \frac{f_{ys}}{\tau_{ET}}\frac{A_b}{\pi d_b},\ l_y = \frac{f_s - f_y}{\tau_{YT}}\frac{A_b}{\pi d_b} \tag{1c}$$

$$\theta = Slip/(d - c) \tag{2b}$$

where, f_s is the steel stress at the interface of joint, f_y is the steel yield stress, E_s is the steel elastic modulus of elasticity, E_{sh} is the steel hardening modulus, τ_E is the bond strength for elastic steel, τ_{ET} is the bond strength for elastic steel in tension, τ_Y is the bond strength for yielded steel, τ_{YT} is the bond strength for yielded steel in tension, A_b is the nominal steel bar area, d_b is the nominal steel bar diameter, d is the effective depth of the cross-section and c is the neutral axis depth of the cross-section.

Table 1: Average bond strengths [19].

Bar stress, fs	*Average bond strength (MPa)*
Tension, $f_s<f_y$	$\tau_{ET} = 1.8\sqrt{f_c}$
Tension, $f_s>f_y$	$\tau_{YT} = 0.4\sqrt{f_c}\ to\ 0.05\sqrt{f_c}$
Compression, $-f_s<f_y$	$\tau_{EC} = 2.2\sqrt{f_c}$
Compression, $-f_s>f_y$	$\tau_{YC} = 3.6\sqrt{f_c}$
* Note:f_c is the CompressiveStrength of concrete in MPa.	

2.2. Shear Spring

While the cracks at beam-column joint region widen due to cyclic loading, the shear transfer capacity across the crack surface decreases. This behavior was represented in the proposed analytical model by the interface shear spring based on the relation between the interface shear stress and slippage by Walraven [20], which was a function of crack width, concrete strength and the maximum diameter of aggregate. This model was used to describe the force-deformation envelope curve in the case of open concrete cracks. However, for the case of cracks are closed, the interface shear response was assumed linear stiffness for a relatively small crack width (ω) equal to 0.10 mm, as shown in Eq. (3). On the other hand, assuming a constant crack width (ω = 0.1 mm) is less reasonable during the cyclic loading because the crack width will increase after every load cycle. The hysteretic properties of the adopted one-dimensional constitutive curve will partly compensate this shortcoming.

$$\tau_{cr} = -f_{cu} + [1.8\omega^{-0.8} + (0.234\omega^{-0.707} - 0.2) \times f_{cu}] \times S \quad (3)$$

Where, τ_{cr} is the shear stress along the crack plane, f_{cu} is the cubic compressive strength of concrete, ω is the crack width, S is the slip at the beam-column joint.

2.3. Shear Joint Spring

In the current analytical study and to describe the shear panel strength of the reinforced concrete beam-column joint, the simplified approachproposed byKim and LaFave [21]were used, which considering the recommendations of ACI-352R-02 [22].The relation between shear stress and shear strain for the shear panel defined by four main point, as shown in Fig. 2. The crack, yield, ultimate and post ultimateshear stresses and the corresponding strains weredetermined by Equations (4) to (7).The coefficients of the crack, yield, and post ultimate shear stresses were 0.44, 0.89, and 0.90, respectively, and the coefficients of the crack, yield, and post ultimate shear strains were 0.02, 0.36, and 2.02, respectively. In this paper, only the yield and ultimate strength were considered in modelling and the other points were ignored, as shown in Fig. 4.b. The rotation of the beam-column joint (θ_j) is considered equal to the joint shear strain (γ_{xy}) andthe moment of the joint panel was calculated byEq. (8) [9].

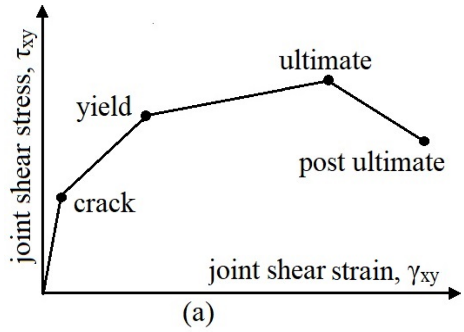

Figure 2: Joint shear stress-strain relationship presented by Kim and LaFave [21].

$$\tau_{xy} = \alpha_t \beta_t \eta_t \lambda_t (JI)^{0.15} \, (BI)^{0.30} \, (f_c^{'})^{0.75} \quad (4)$$

$$JI = (\rho_j f_{yj})/f_c^{'} \quad (5)$$

$$BI = (\rho_b f_{yb})/f_c^{'} \quad (6)$$

where, τ_{xy} is the joint shear stress, α_t is a parameter for the in-plane geometry: 1.0 for interior connections, 0.7 for exterior connections, and 0.4 for knee connections, β_t is a parameter for describing the out of plane geometry: 1.18 for joints two transverse beams and 1.0 for all other cases, η_t describes joint eccentricity: 1.0 for no eccentricity, λ_t is a constant value equal to 1.31, JI is the joint transverse reinforcement index, ρ_j is the volumetric joint transverse reinforcement ratio in the direction of loading, f_{yj} is the yield stress of joint transverse reinforcement, BI is the beam reinforcement index, ρ_b is the beam reinforcement ratio and f_{yb} is the yield stress of beam reinforcement.

$$\gamma_{xy} = \alpha_{\gamma t} \beta_{\gamma t} \eta_{\gamma t} \lambda_{\gamma t} \, BI \, (JI)^{0.10} \left(\frac{\tau_{xy}}{f_c^{'}}\right)^{-1.75} \quad (7)$$

where,γ_{xy} is the joint shear strain, $\alpha_{\gamma t}$ is a parameter for the in-plane geometry: 1.0 for interior connections, 0.328 for exterior connections, and 0.093 for knee connections, $\beta_{\gamma t}$ is a parameter for describing the out of plane geometry: 1.40 for joints two transverse beams and 1.0 for all other cases, $\eta_{\gamma t}$ describes joint eccentricity: 1.0 for no eccentricity, $\lambda_{\gamma t}$ is a constant value equal to 0.0055.

$$M_j = \frac{\tau_{xy} . h_c . b_j}{\frac{(1 - b_j/L_b)}{j_d} - \frac{\alpha}{L_c}} \quad (8)$$

where, M_j is the moment of the joint, b_j is the effective width of the joint panel, h_c is the depth of the column cross-section, j_d is the moment arm of the beam, L_b is the total beam length between the contra flexure points, L_c is the sum of the lower and upper column height, and α is a constant value equal to 2 for the top floor joints and 1 for other joints.

3. Finite Element Model Verification

To check the accuracy of the proposed analytical model, the beam-column joint model and the moment resisting frame model were verified with the experiential results conducted by the previously available experimental tests. Two exterior and interior reinforced concrete beam-column joint and one three-dimensional reinforced concrete frame were considered. The modified Richard-Abbott and the modified bilinear Takeda curves were used in the analysis to define the rotational and shear spring elements.All empirical parameters of the Richard-Abbott curve were defined according toNogueiro et al. [15].

3.1 Reinforced Concrete Beam-Column Joints

Two experimental reinforced concrete beam-column joints were considered and modelled in this section. These interior (A1) and the exterior (B2) joints were experimentally tested by Yu [1].The reinforcementand detailsof these beam-column joints are shown in Fig. 3.Also, the geometry and details of theanalytical joint models are shown in Fig. 4. First, the beam-column joint model verified usinga horizontal monotonic displacement loading in each direction till the failure of joints. After that, the model was subjected to a prescribed reversed horizontal displacement cyclic loading. All specified parameters for the interface bar-slip spring and the shear spring are listed in Table 2 and Table 3, respectively. Fig 5 presents the moment rotation curve for the shearjoint spring. The comparison between the obtained results obtained from the joint model and the experiments for the beam-column joints A1 and B2 are shown in Fig. 6. The results confirmed the accuracy of the proposed model. The results indicated that the monotonic loading results and the cyclic loading results of the model were identically till yield at 3% drift ratio. After that the cyclic response deviated from the monotonic loading response due to stiffness degradation and strength deterioration especially at the exterior connection.

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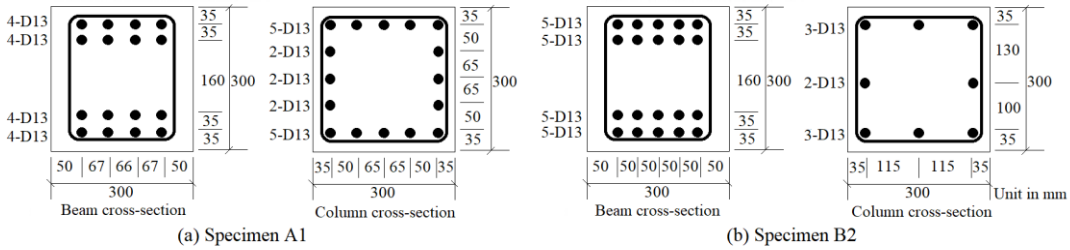


Figure 3: Reinforcement and details of beams and columns for the interior joint A1 and the exterior joint B2 tested by Yu [1].

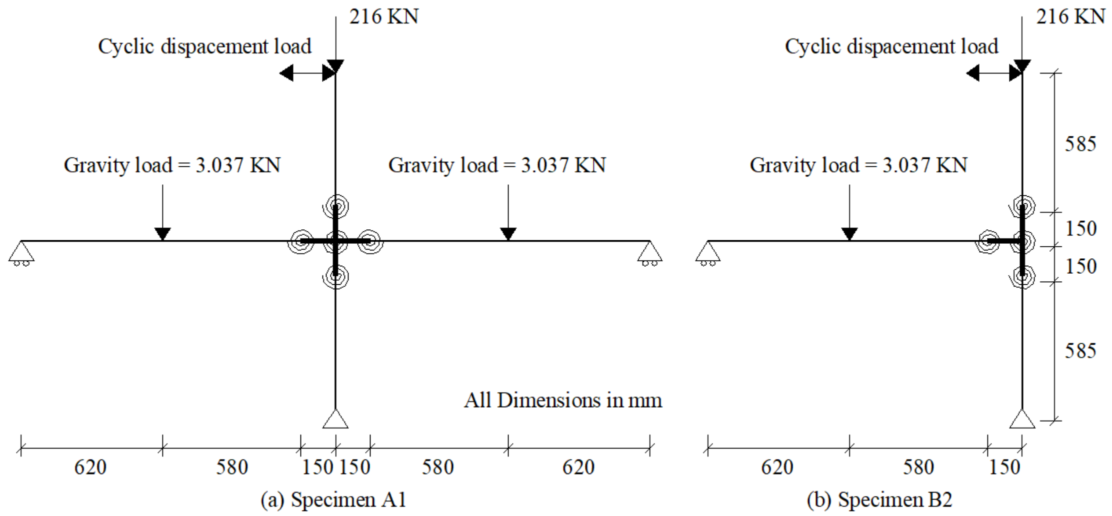


Figure 4: Analytical modelfor the interior joint A1 and the exterior joint B2tested by Yu [1].

Table 2: Parameters for the interface bar-slip spring using the modified Richard-Abbott curve

Specified parameters	*A1*	*B2*	*Specified parameters*	*A1*	*B2*
K_a, K_{ap} (KN.m/rad)	106570	124590	K_d, K_{dp} (KN.m/rad)	106570	124590
M_a (KN.m)	85.0	99.36	M_d (KN.m)	85.0	99.36
K_{pa}, K_{pap} (KN.m/rad)	1811.2	1944.9	K_{pd}, K_{pdp} (KN.m/rad)	1811.2	1944.9
N_a, N_{ap}, C_a	1.0	1.0	N_d, N_{dp}, C_d	1.0	1.0
M_{ap} (KN.m)	8.50	9.936	M_{dp} (KN.m)	8.50	9.936
t_{1a}	6	6	t_{1d}	6	6
t_{2a}	0.30	0.30	t_{2d}	0.30	0.30
i_{Ka}, i_{Ma}, H_a	0.0	0.0	i_{Kd}, i_{Md}, H_d	0.0	0.0
E_{max-a} (rad)	0.10	0.10	E_{max-d} (rad)	0.10	0.10

Table 3: Parameters for the interface shear spring using the bilinear Takeda curve.

Specimens	*F_y (KN)*	*K_y (KN/mm)*	*α*	*β_0*	*β_1*
A1 – B2	405	4800	0.000001	0.00001	1.0

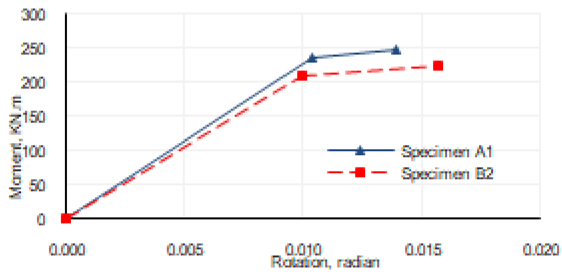


Figure 5: Moment rotation curves of the shear joint spring for the interior joint A1 and the exterior joint B2.

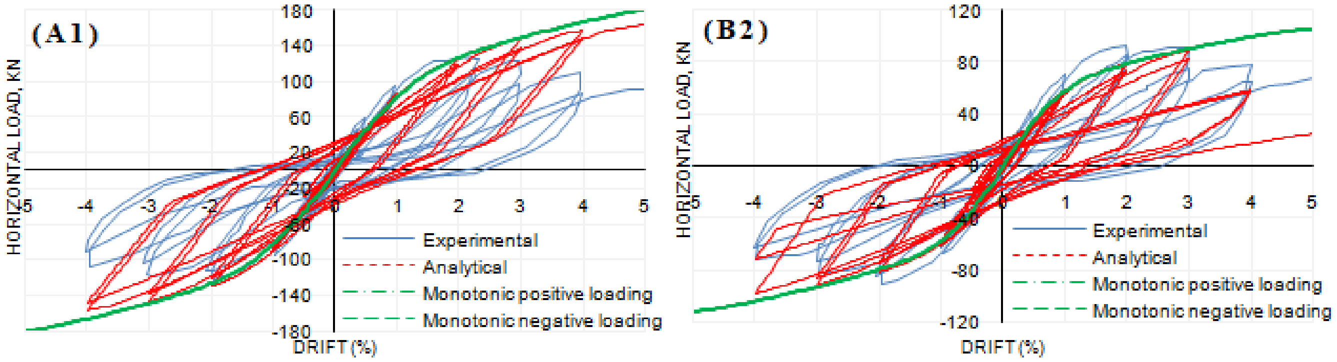
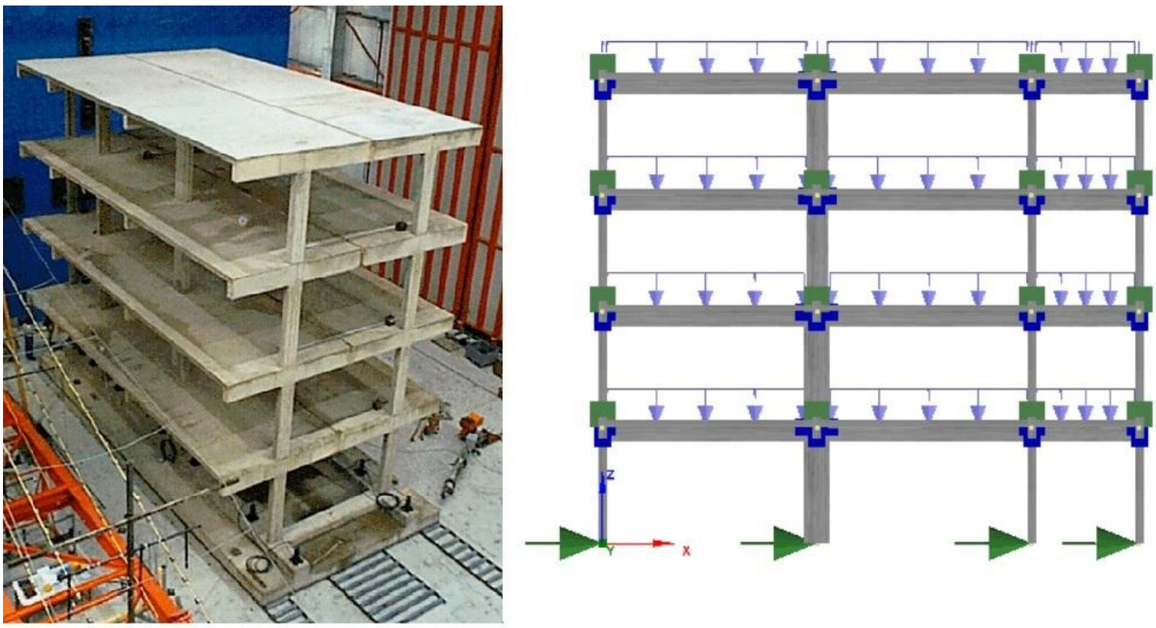


Figure 6: Comparison of the hysteretic and monotonic load drift curves for the interior joint A1 and the exterior joint B2

3.2 Reinforced Concrete Moment Resisting Frames

In this section and to verify the moment resisting frame model, a three-dimensional three-bays and four-stories reinforced concrete frame structure was used. This reinforced concrete frame was one of two full scale frames experimentally tested under apseudo-dynamic load by Pinho and Elnashai [4], as shown in Fig. 7. The dynamic time history load consisted of two seismic shocks with duration 15 seconds and a rest time between the two loads with duration 35 seconds, as shown in Fig. 8. All the details of the frameare shown in Figures9 and 10. The specified parameters and the properties of the interface bar-slip spring, the interface shear spring, and the shear joint spring were calculated as described in the previous sections. It should be noted that, the second seismic shock was stopped in the experimental test due to the major damage of one column in the frame structure. The comparison between the experimentalresults and the output model results are shown in Fig.10. The output top displacement of the finite element frame model in the two seismic shocks are shown in Figures 11 and 12. The results proved that the proposed model could simulate the seismic behavior of the moment resisting frame with accepted accuracy.



(a) (b)

Figure 7: (a) Frame in fixed position for test [4]; (b) Finite element frame model.

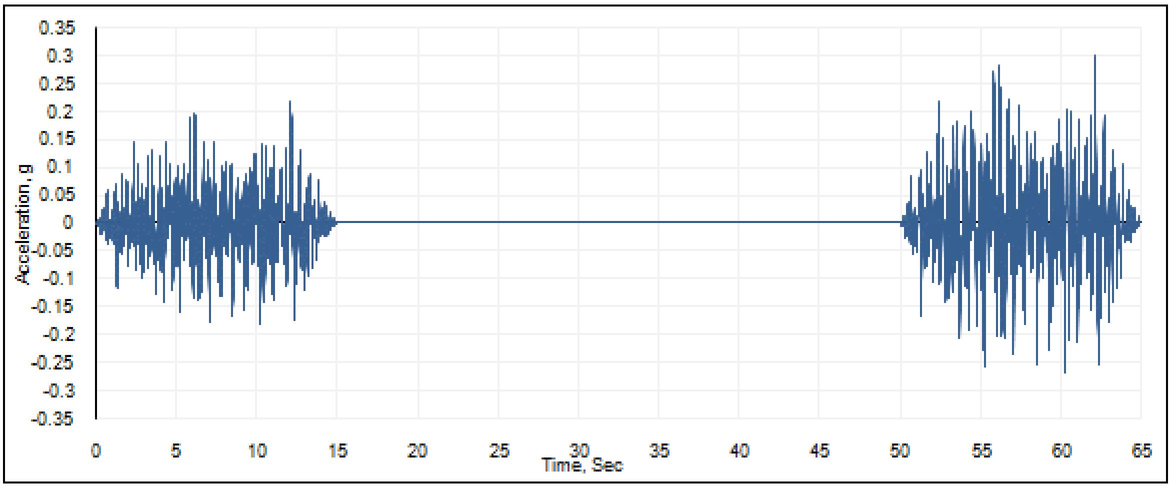


Figure 8: Dynamic time history load [4].

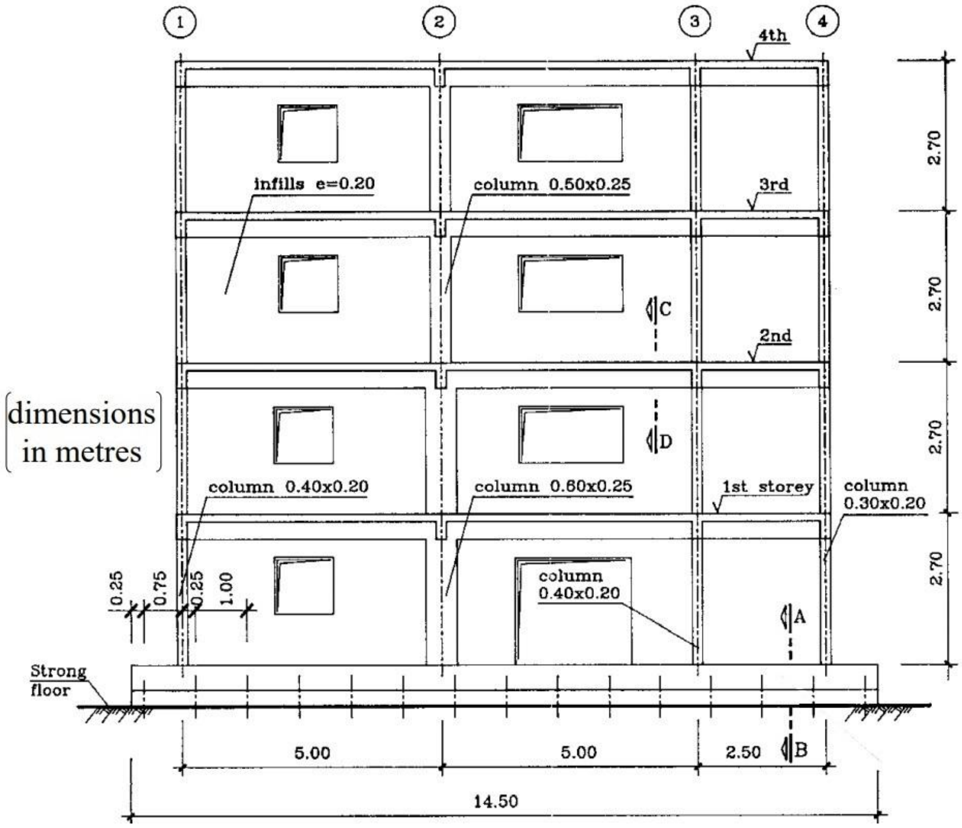


Figure 9: Elevation view of the tested reinforced concrete frame [4]

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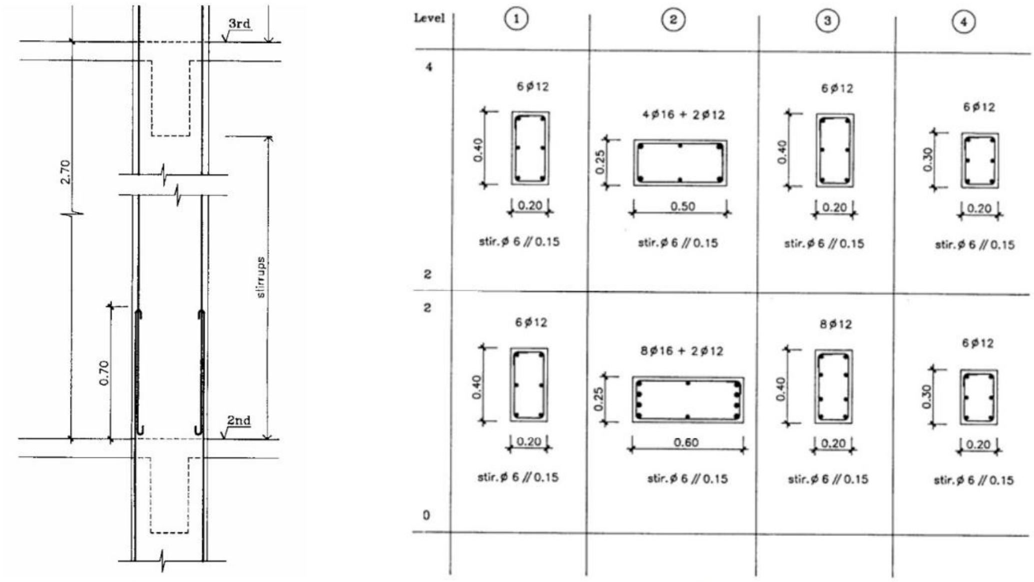

(a) (b)

Figure 10: Reinforcement details the tested frame [4]. (a) lap-splicing detail; (b) cross-section characteristics.

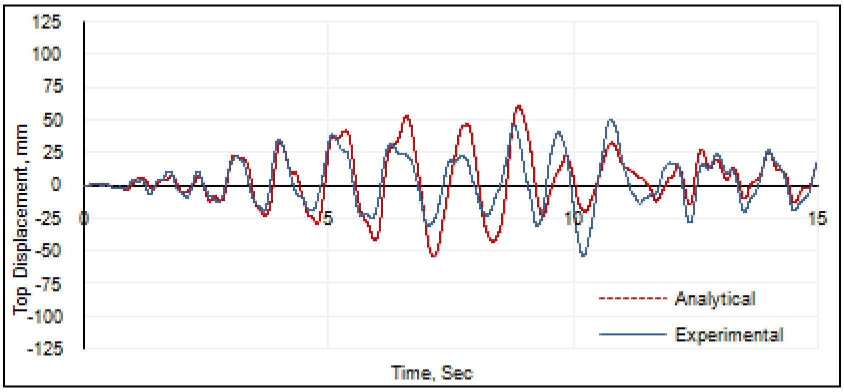

Figure 11: Results of the finite element framemodel at the first seismic shock

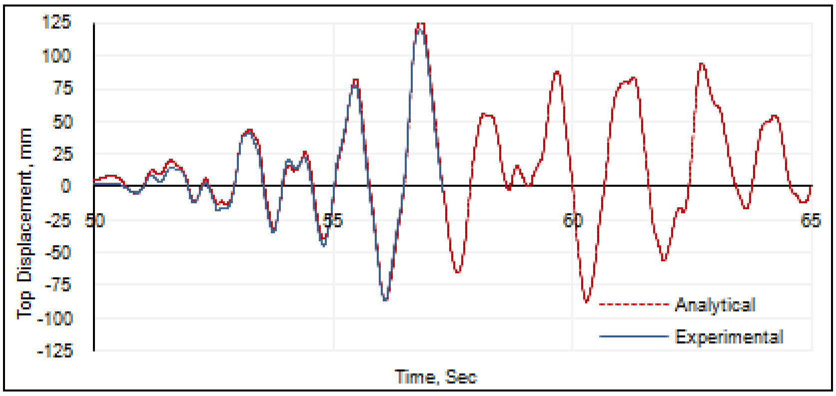

Figure 12: Results of the finite element frame model at the second seismic shock

4. Conclusions

The main purpose of this paperwas to present a nonlinear beam-column joint model and a moment resisting frame model subjected to reversed cyclic loads using the commercial software SeismoStruct. To check the accuracy of the analytical models, the beam-column joint model and the moment resisting frame model were verified with the experiential results conducted by Yu [1] and Pinho and Elnashai [4], respectively. From the results obtained from the finite element models presented in this study, the following conclusions may be drawn:

- The finite element beam-column joint model using the commercial software SeismoStruct was able to simulate the monotonic and the cyclic behavior of the reinforced concrete beam-column joints. Also, it was able to predict the stiffness degradation and the strength deterioration ofthe tested joints.
- The obtained results from the finite element frame model proved that the model could simulate the seismic behavior of the moment resisting frame with accepted accuracy.

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