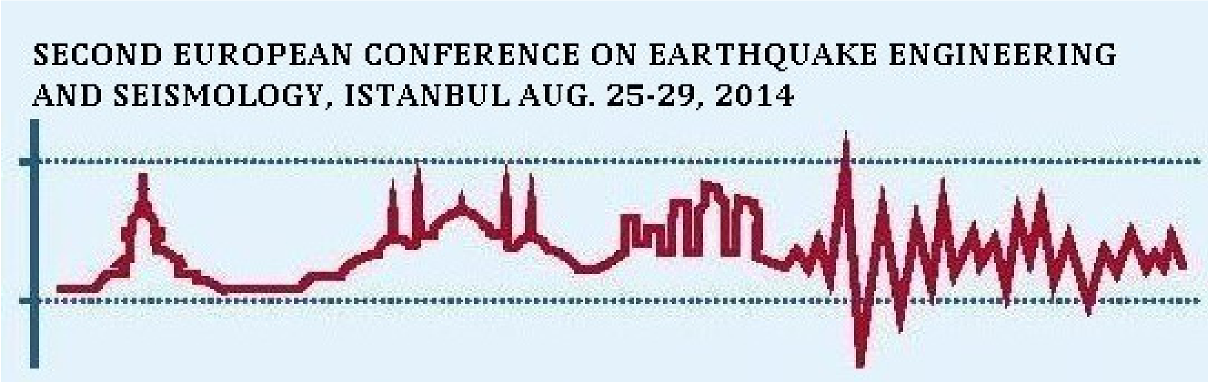

INFLUENCE OF JOINT MODELING ON SEISMIC EVALUATION OF NON-SEISMICALLY DESIGNED RC FRAME STRUCTURES

Akanshu SHARMA[1], R. ELIGEHAUSEN[2] and J. HOFMANN[3]

ABSTRACT

Under the action of seismic forces, joints of RC frame structures are subjected to large shear stresses in the core. The seismic performance of RC frame structures is often governed by the behavior of its beam-column connections. This is especially true for the case of non-seismically designed frame structures that were constructed following old and seismically poor detailing practices. For correct assessment of seismic performance of such structures, it is essential to consider inelastic behavior and distortion of the joint core. In this paper, the influence of joint modelling on seismic evaluation of non-seismically designed (NSD) RC frame structures is highlighted through examples at sub-assembly and structural level. It is clearly brought out that not considering the inelastic behaviour of beam-column joints can not only result in over-prediction of the failure loads, but also can lead to wrong prediction of failure modes and unrealistically high ductility. The examples clearly demonstrate the importance of modelling the joint inelastic behaviour in order to capture the realistic seismic behaviour of the NSD structures. Furthermore, it is shown that a relatively simple and practical lumped plasticity based model is able to capture the seismic response of the joints of the structures well.

INTRODUCTION

Traditionally, beam-column joints of the reinforced concrete (RC) frame structures were considered rigid, which was credited to the confining action of the members framing into the joint core. This approach works well to analyse structures subjected to gravity loads since the joint panel is not subjected to large forces in such cases. However, under the action of seismic forces, beam-column joints are subjected to large shear stresses in the core. These shear stresses in the joint are a result of moments of opposite signs on the member ends on either side of the joint core. Typically, high bond stress requirements are also imposed on reinforcement bars of beams framing into the joint. The axial and joint shear stresses result in principal tension and compression that leads to diagonal cracking and/or crushing of concrete in the joint core. These stresses in the joint core are resisted by the so-called strut and tie mechanism (Paulay and Priestley, 1992).

To prevent the shear failure of the joint core by diagonal tension, joint shear reinforcement is needed, which is therefore prescribed by the newer design codes. Moreover, these codes prescribe a large anchorage length of the bars terminating in case of exterior joints, so that a bond failure may be avoided. It is now generally accepted that the design and detailing requirements laid by new codes ensure a better seismic performance as compared to those designed as per old codes. This is especially true for the performance of beam-column joints of structures. However, a majority of the structures

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around the world were constructed before such codes came into force. The number of such non-seismically designed (NSD) structures is even larger in case of developing countries where the seismic designs are still not mandatory.

Several past earthquakes have demonstrated the vulnerability of the beam-column joints of NSD structures under seismic excitation. The vulnerability of beam-column joints have also been confirmed through several experiments performed at beam-column joint sub-assembly level as well as at structural level. Therefore, in order to evaluate the seismic performance of such structures realistically, modelling of the inelastic behaviour of beam-column joints is essential. Though it is well-accepted that the beam-column joints, especially of non-seismically designed structures, behave inelastically during the earthquakes, still the analysis approach mainly revolves around considering concentrated plasticity at the member ends and assuming the joint core as rigid. This is due to the fact that the models available in literature generally are not simple enough to be used in commercial programs being at the same time able to predict the shear behavior of the joints reasonably. Moreover, the models either require large computational efforts so that they are not practically useful for analyzing the global structural behavior or they need a special element with various nodes and springs or a special purpose program to implement the joint nonlinearity. This makes it difficult for the designers and analysts to follow the recommended approaches using the commercial programs.

In this paper, the importance of joint modelling on the seismic evaluation of NSD structures is emphasized. It is demonstrated that if the joint inelasticity is not considered in the modelling approach, the performance as well as the failure modes of NSD structures will most likely be wrongly predicted. A relatively simple, practical and efficient model to simulate joint inelastic behaviour developed by Sharma et al. (2011) is presented and used to demonstrate the importance of joint modelling in NSD structures. The model is suitable for practical usage and can be easily and successfully be used with existing commercial programs without the need of any special sub-element.

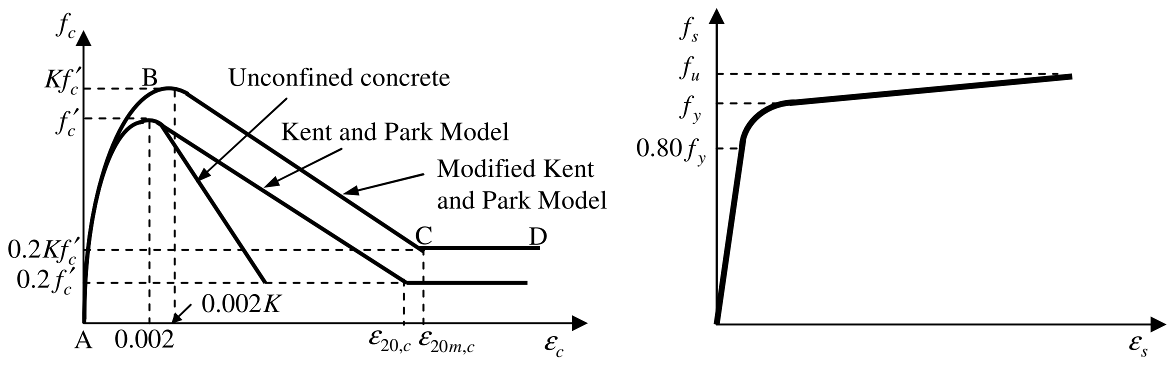
NUMERICAL MODELING OF MEMBERS AND JOINTS

The numerical modelling is performed within the framework of stiffness matrix analysis and lumped plasticity approach. The members (beams and columns) are modelled with 3D beam elements having six degrees of freedom at either end. This is the usual way of modelling structural members in popular commercial software such as SAP2000, STAADPro etc., to name a few. The possible inelastic behaviour at the critical locations is modelled using nonlinear springs. The various nonlinearities considered are: (i) flexural characteristics of the members (beam and column); (ii) shear characteristics of the members (beam and column); and (iii) shear behaviour of the joint core.

Flexural characteristics for members

The stress-strain characteristics of concrete confined by transverse reinforcement exhibits a more ductile behavior than its unconfined counterpart (Park and Paulay, 1975; Paulay and Priestley, 1992). Therefore, in order to generate moment-rotation characteristics for a section, the first step is to obtain the stress-strain curve for the confined concrete. In this work, the modified Kent and Park model (Park et al, 1982) (Fig. 1a) was followed mainly because it offers a good balance between simplicity and accuracy. The stress-strain characteristics for the reinforcement steel used in this work is considered to include strain hardening in the post yield portion of the curve (Fig. 1b). Same curve was followed for reinforcement bars in tension and compression. Once the stress-strain curves for steel and concrete are formulated, the moment-curvature characteristics of the section were derived using the standard procedure considering the equilibrium of forces and compatibility of strains.

The concrete strain at the extreme compression fiber, is assumed and the force equilibrium between compressive and tensile forces is established using iterative procedure to arrive at the correct neutral axis depth. The moment of resistance is then calculated by taking the moments of compressive and tensile forces about the centroid of the section and the corresponding curvature is obtained by dividing the extreme compression fiber strain by the neutral axis depth. In this work, the magnitude and point of application of compressive forces in concrete for various strain levels were calculated using the equivalent stress block approach (Park and Paulay, 1975).



(a) Confined concrete (Park et al., 1982) (b) Reinforcing steel

Figure 1. Constitutive laws used for concrete and reinforcing steel in this work

The generated moment-curvature characteristics were converted to moment-rotation characteristics using the following expressions for yield and ultimate rotations:

$$\theta_y = \int_0^L \varphi_y dx = \int_0^L \frac{M_y}{EI} dx \tag{1}$$

$$\theta_u = \theta_y + \left(\varphi_u - \varphi_y\right) l_p \tag{2}$$

where, L is the length of member between critical section and point of contra-flexure, φ_y and φ_u are the yield and ultimate curvature, θ_y and θ_u are yield and ultimate rotation, M_y is the yield moment, EI is flexural rigidity, and l_p is the plastic hinge length, which was calculated using the formulation suggested by Baker and Amarakone (1964) for confined concrete.

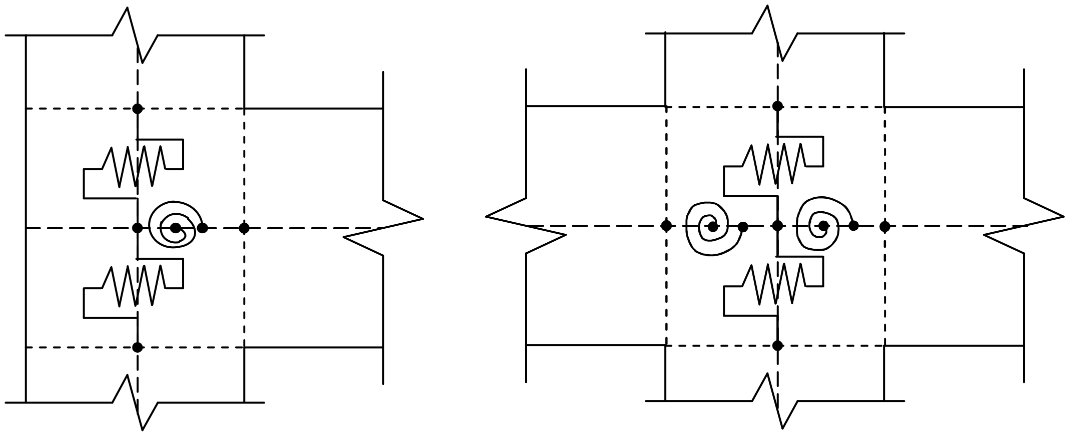
Shear hinge

To predict the shear force-deformation characteristics, an incremental analytical approach was followed (Watanabe and Lee, 1998), which is based on the truss mechanism. In this model, the stirrup strain is gradually increased with a small increment and the resisting shear at each step is calculated. The stress state is characterized by a biaxial stress field in the concrete and a uniaxial tension field in the shear reinforcement. The theoretical basis given by Kupfer and Bulicek (1991) for the equilibrium condition of stresses and compatibility condition of strains for the concrete element shown is followed. The equilibrium condition of stresses, compatibility condition of strains and constitutive laws are then used to obtain the complete shear force vs. deformation characteristics for the members. The method is straightforward and easily programmable. However, a detailed description of the approach is beyond the scope of this paper and details of the model can be found in (Watanabe and Lee, 1998).

Joint hinge

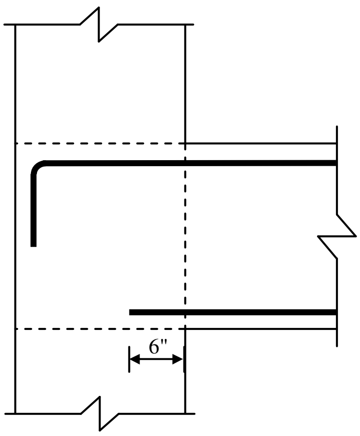
For NSD structures, it is very important to model the nonlinearities in the beam-column joints in order to capture the realistic seismic behavior. In this work, the joint model proposed by Sharma et al. (2011) is followed. The model uses limiting principal tensile stress in the joint as the failure criterion so that due consideration is given to the axial load on the column. The spring characteristics are based on the actual deformations taking place in the sub-assemblage due to joint shear distortion. For a planar exterior joint, two shear springs and one rotational spring are used to model the joint distortion (Fig. 2a), while for an interior joint, two shear springs and two rotational springs are used(Fig. 2b). The failure criteria used for the joint springs is based on the critical principle tensile stress criteria. For exterior joints, the critical principle tensile stress values depend on the anchorage detail of the beam longitudinal reinforcing bars. As an example, the curve for principal tensile stress vs. shear strain for a typical joint with top bar bent in and bottom bar straight with 150mm embedment is shown in Fig. 3. This curve is based on the recommendations of Priestley (1997) and evaluation of various test results as explained in Sharma (2013). The joint spring characteristics are derived using the relation shown in

Fig. 3 and equilibrium of the joints. The complete details are given in Sharma et al (2011) and Sharma (2013).



(a) Exterior joint (b) Interior joint

Figure 2. Inelastic springs used to model the inelastic behaviour of beam-column joints



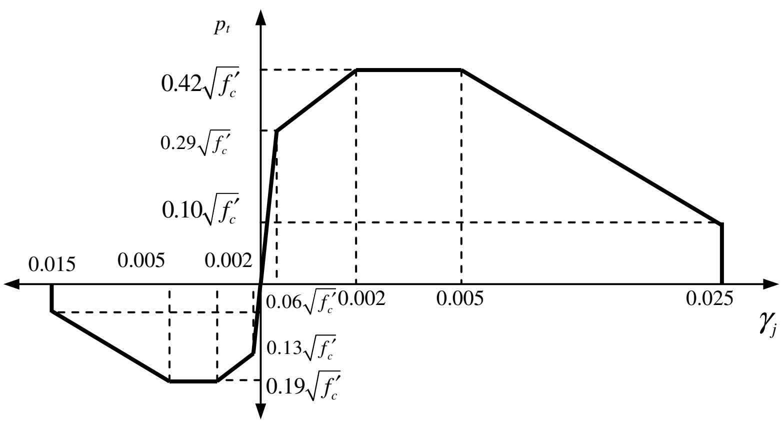


Figure 3(a). Beam bar anchorage details Figure 3(b). Failure criteria (principal tensile stress-shear deformation)

For a typical beam-column joint sub-assembly, e.g. an interior joint assembly, Fig. 4 shows the various springs used in the typical numerical model within frame analysis approach.

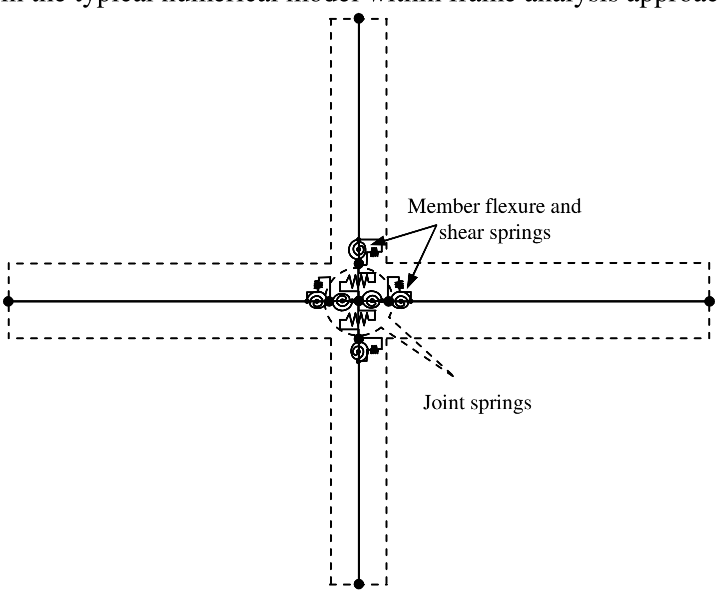


Figure 4. Inelastic springs used to model the inelastic behaviour of beam-column joints

It has been shown by experiments (Wong and Kuang, 2008) that the shear strength of the joints decreases with increasing aspect ratio (ratio of beam depth to column depth). The reason for the influence of joint aspect ratio on its strength can be explained with the argument that with larger aspect ratios, the angle of the concrete strut with horizontal is larger and hence the horizontal component of the diagonal strut that resists the tensile force from beam reinforcing bars is less. Therefore to maintain equilibrium, for same tensile force in the beam bars, higher compression force in the concrete strut is required. In this work, based on the experimental results by Wong and Kuang (2008), it is recommended (Sharma, 2013) to consider the effect of aspect ratio by multiplying the critical principal tensile stress values for cracking and/or failure by the factor k_α, given as:

$$k_\alpha = \frac{1}{\alpha} \tag{3}$$

Where, α is the joint aspect ratio defined as the ratio of beam depth to column.

Hence, for a joint having aspect ratio α, the critical principal tensile stress corresponding to a particular limit state (e.g. 1st cracking) can be obtained as:

$$p_{t,\alpha} = k_\alpha p_{t(\alpha=1)} \tag{4}$$

NUMERICAL ANALYSIS OF JOINT SUB-ASSEMBLIES

In order to demonstrate the importance of joint modelling, the analysis was performed on the beam-column joint sub-assemblies of NSD structures. The modelling was performed within the framework of lumped plasticity approach and matrix analysis using commercial software SAP2000. In all the cases, the numerical analysis was performed (i) by only considering nonlinearities at the member ends, while considering rigid joint (no joint model), and (ii) by modelling inelastic joint shear behaviour in addition to the nonlinearities at the member ends (joint model). The numerical modeling technique was applied and verified on various joint sub-assemblies tested by different researchers.

Analysis of exterior tests by Genesio and Sharma (2010)

Genesio and Sharma (2010) performed experiments on full-scale exterior RC beam-column joints under cyclic loads. Here, the comparison of experimental results for two joints with different beam bar anchorage in the joint panel as shown in Fig. 5 is provided. Joint JT1-1 had both top and bottom beam bars bent into the joint core, while in case of joint JT3-1, the bottom beam bar was embedded straight with 150 mm embedment into the joint. The average compressive strength of concrete on testing day, obtained by performing tests on 150 mm size cubes for JT1-1 and JT3-1 were obtained as 31.75MPa and 30.44MPa respectively. The average yield and ultimate strength for reinforcing bars were 550 MPa and 663 MPa respectively. The joints were tested in erect position, with column vertical. Both ends of the columns were held by elastic hinge sub-assembly. No axial load was applied on the column in any case.

The joints had an aspect ratio of 1.33. Therefore, the critical principle tensile stresses were multiplied by the factor to consider the reduction in critical joint stress due to aspect ratio (eqs. 3 and 4). The comparison of experimental and numerical results for joint JT1-1 is given in Fig. 6. The analysis results follow the experimental results very closely, especially in the negative direction of loading. In the experiment, the peak load in the positive direction was obtained as 79.9 kN and that in the negative direction was 61.5 kN. Such significant difference in the experimental peak loads is quite rare for joints having symmetric beam reinforcement and no axial load and may be attributed to certain local in-homogeneity. However, the peak load was obtained as 62.5kN for either side in the analysis. It can be observed, that the model considering joint as rigid (no joint model) predicts a very high failure load as well as ductility for the joint. Thus, the analysis considering joint as rigid leads to unsafe results, while the analysis considering joint inelastic behavior leads to good prediction of the response of the joint.

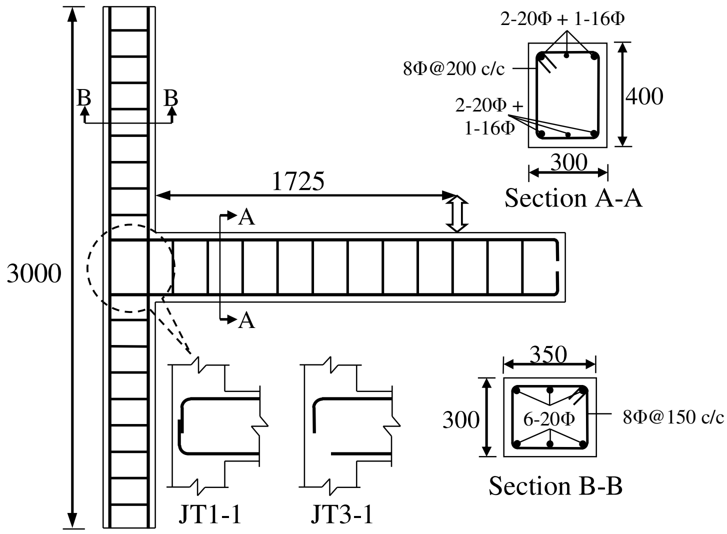


Figure 5. Details of beam-column joints tested by Genesio and Sharma (2010)

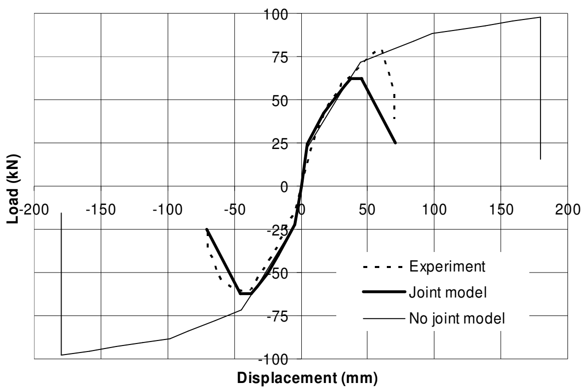


Figure 6. Results for joint JT1-1

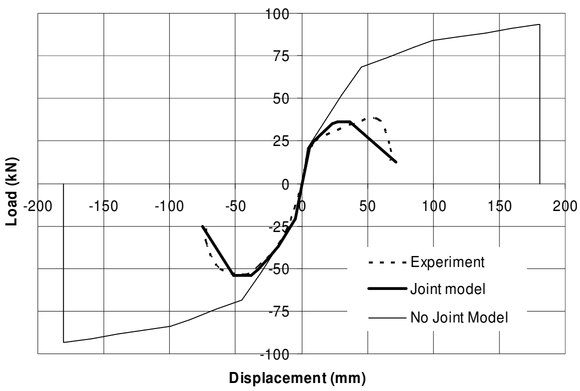


Figure 7. Results for joint JT3-1

The comparison of experimental and numerical results for joint JT3-1 is given in Fig. 7. Due to un-symmetric embedment of beam reinforcing bars into the joint, the peak loads in the experiment for positive and negative loading directions were obtained as 38.0kN and 53.5kN respectively. This effect can be accounted for in the joint model, through different failure criteria for different beam bar embedment (see Fig. 3). The comparison of numerical and experimental results clearly show the suitability of the model to predict the joint behavior realistically. The peak load in the analysis with joint model was obtained as 36.4kN and 54.2kN for positive and negative loading direction respectively, which is very close to experimental results. On the other hand, the analysis performed considering the joint as rigid (no joint model) cannot consider the influence of unsymmetric embedment and hence the peak load in this case is obtained as 93.3kN for either side, which is very unsafe. Again, a very high ductility for the joint sub-assembly is predicted.

It is important to note that in case of no joint model, the failure mode predicted for both the joints is identical, which in this case is beam flexure failure. In principle, the load-displacement curve is also identical for the two joints. The results of the numerical results clearly bring out the importance of considering the joint inelastic behavior to obtain realistic predictions and that ignoring joint inelasticity leads to unsafe results.

Analysis of exterior tests by Genesio and Sharma (2010)

Dhakal et al (2005) performed experiments on gravity designed interior beam-column joints as shown in Fig. 8, that were part of frames designed according to the British standard BS8110. The geometrical dimensions and reinforcement details of the C1 and C4 type specimens are illustrated in Fig. 8. Both the specimens were without any vertical or lateral hoops inside the joint core. Standard compression test results on cylinders showed that the average compressive strength of concrete was

31.6 MPa for the C1 type specimens and 32.7 MPa for the C4 type specimens. All specimens were subjected to an axial compression of 10% axial capacity.

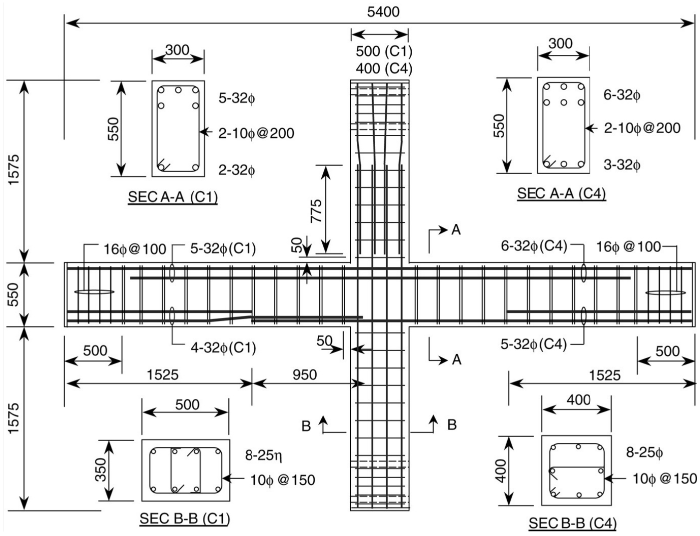


Figure 8. Details of beam-column joints tested by Dhakal et al. (2005)

The comparison of experimental and numerical results for joint C1 tested by Dhakal et al. (2005) is given in Fig. 9. Again, to visualize the significance of joint modelling, a comparison is given with the analysis results when the model did not have springs to model the joint shear behaviour. The peak load from the experiment was obtained as 225 kN for both up and down directions. In the analysis, the peak load was obtained as 224 kN for both up and down directions using joint model, which is very close to the experiment, while the same was obtained as 332 kN for both up and down directions without using joint model. Thus, the model without joint springs again yielded results on highly unsafe side for both the directions.

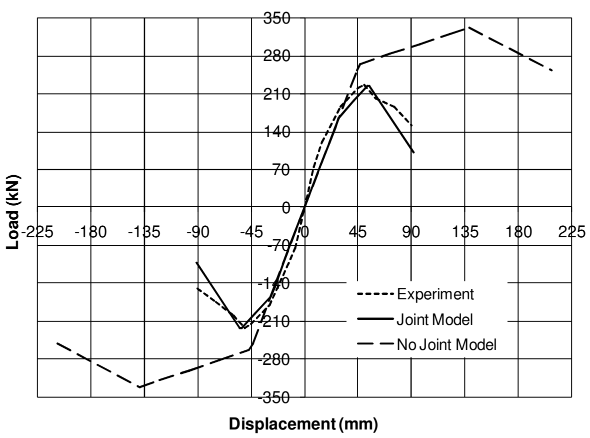


Figure 9. Results for joint C1

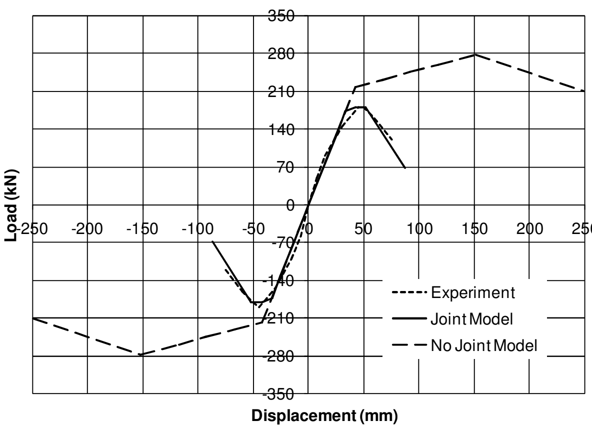


Figure 10. Results for joint C4

Joint C4 had an aspect ratio of 1.375 (Fig. 8). Therefore, the critical principle tensile stresses were multiplied by the factor to consider the reduction in critical joint stress due to aspect ratio. The comparison of experimental and numerical results is given in Fig. 10. The peak load from the experiment was obtained as 181 kN for both and down directions. In the analysis, the peak load was obtained as 179 kN for both up and down directions using joint model, which is again very close to the experimental results. The peak load considering joint as rigid (no joint model) was obtained as 280 kN for both up and down directions. This further proves the validity and importance of modelling the joint using the proposed joint model. Also, it is shown that the model without joint springs may yield results on highly unsafe side.

NUMERICAL ANALYSIS OF NSD STRUCTURES

Similar to beam-column joint sub-assemblies, the analysis of the NSD structures was performed with and without considering the joint shear behaviour. A full scale 3D structure tested under lateral monotonic pushover loads by Sharma et al. (2013) was numerically analysed (Fig. 11). The structure was constructed following the non-seismic detailing practice and no hoop reinforcement was provided in the joint core (Fig. 11). The failure patterns displayed the vulnerability of RC buildings with non-conforming detailing which tend to fail in undesirable failure mechanisms, such as joint shear failures, bond failures, etc. The complete details can be obtained from Sharma et al. (2013). In this section, the numerical modeling aspects of the same will be provided and the importance of joint modeling at structural level will be highlighted.

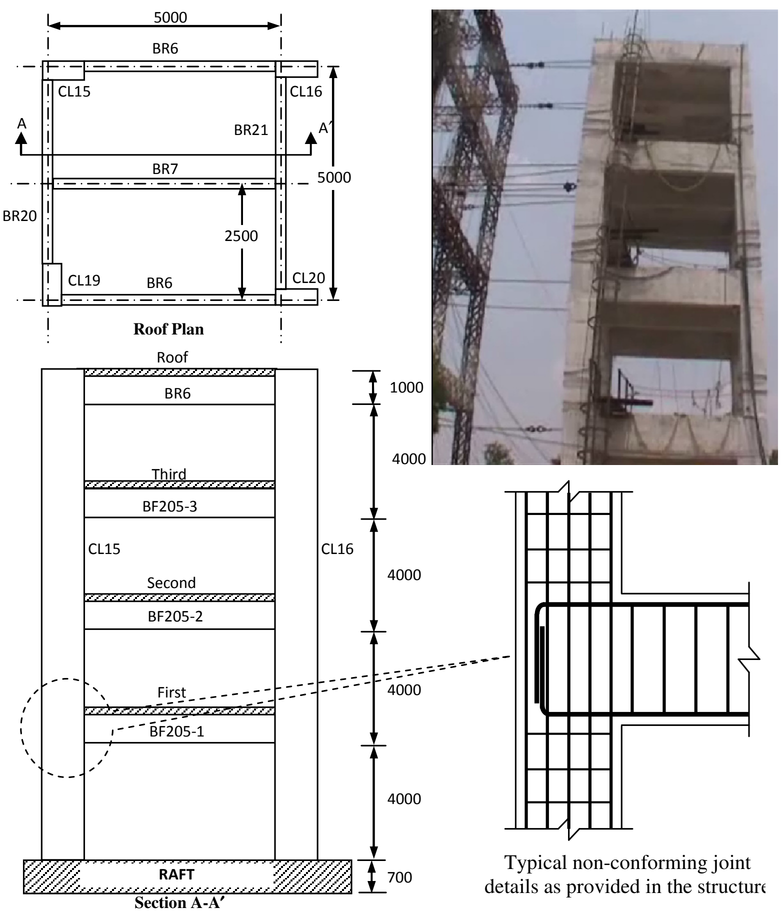


Figure 11. Details of full scale NSD structure tested by Sharma et al. (2013)

Fig. 12 shows the nonlinear springs modelled on a typical joint of the structure. In order to have a comparison among modelling techniques, three cases were analysed, with different types of nonlinear hinges models: Model 1, with flexural and shear hinges only; Model 2, with torsional hinges along with flexural and shear hinges; and Model 3, with joint characteristics along with torsional, flexural and shear hinges. Fig. 13 shows the comparison of experimental and analytical results for the examined cases. It can be observed that models where joint shear behaviour is not considered over-predict the response of the structure. After considering the joint characteristics, torsional effects, moment and shear characteristics the analysis using third model predicted very well the load-deformation behaviour of the structure. The results of analysis at the structural level again highlight the importance of joint modelling for NSD structures subjected to seismic loads.

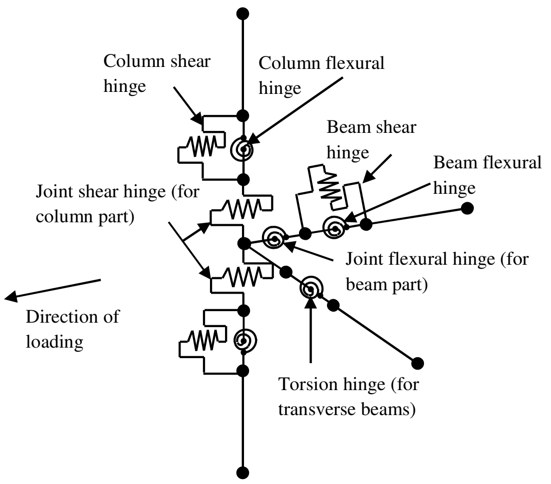


Figure 12. Nonlinear springs at a typical joint

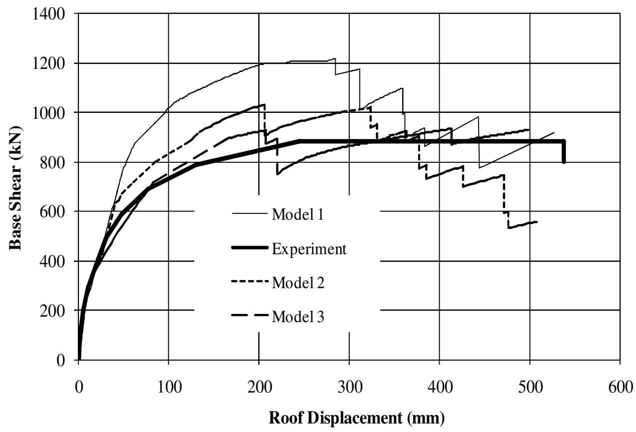


Figure 13. Comparison of results for the structure

Fig. 14 presents a comparison of the experimentally observed and numerically simulated deflected shape of the structure for each analysis case, with respect to the point when the structure reaches the first peak. Since the computational models and the experimental setup reach peak base shear at different displacement, for better comparison of the deflected shape, the actual values of the storey displacement were normalized with respect to roof displacement. It can be seen that the numerically obtained displacement shape for Models 1 and 2 display a parabolic shape for the structure and do not match the experimentally observed profile. This discrepancy is attributed to the rigid behavior of the joints. However, in the experiment, due to the failure at joint levels, the displacement of the roof level was much less than would be expected in the case of shear building behavior. In order to simulate this phenomenon, modeling of joint nonlinearities becomes extremely important and therefore explains why the deflected shape obtained from Model 3 matches closely the experimentally observed one.

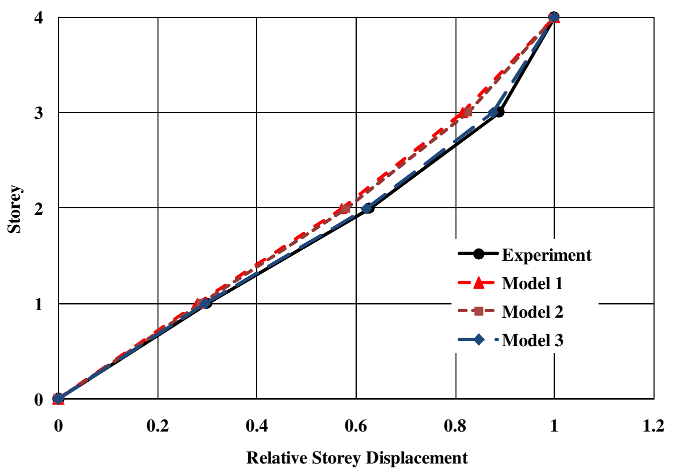


Figure 14. Comparison of deflected shape of the structure

Fig. 15 depicts the various hinges formed in the structure in the computational model with flexural, shear, torsional and joint hinges. An enlarged view of the first floor of the structural model is shown in Fig. 15 where each hinge and its corresponding physical significance in real life case are shown. The consistency between the hinges obtained in the analysis and the failures in the experiment is remarkable. Thus, it has been shown that in order to capture the overall behavior of RC structures, neglecting the inelasticity in the joints can lead to inaccurate results. The first two models over-predicted the base shear resistance of the structure and inaccurate deflected shapes were also derived. In contrast, it was found that via Model 3, not only the pushover curves, but also the deflected shape of the structure as well as the failure modes and locations could be satisfactorily simulated. Therefore, it can be concluded that the joint model works well at structural level and joint modeling is essential for correct prediction of the seismic behavior of RC structures detailed as per non-seismic guidelines.

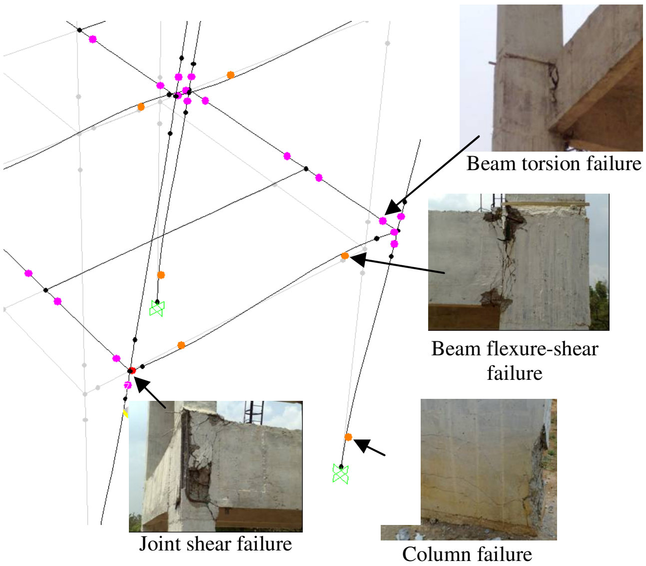


Figure 15. Comparison of failure modes as experimentally and numerically derived

CONCLUSIONS

In this paper, the importance of modelling the inelastic joint behaviour to obtain realistic seismic response of NSD structures is highlighted. The beam-column joints often govern the response of NSD structures subjected to seismic loads. Therefore, it is essential to consider their inelastic behaviour in numerical models. A practical and realistic model that is capable of considering joint inelastic behaviour within the framework of frame analysis approach is discussed. It is shown through different examples on beam-column joint sub-assemblies that considering the joint as rigid may lead to unrealistic and unsafe predictions of load-displacement behaviour as well as failure modes. The importance of joint modelling is also emphasized through an example of a full-scale NSD structure tested under pushover loads. It is shown that only the model where the joint inelastic behaviour is suitably considered can lead to realistic predictions on load-displacement behaviour, displacement profile and failure modes.

REFERENCES

Baker ALL and Amarakone AMN (1964) "Inelastic Hyperstatic Frames Analysis", *Proceedings of the International Symposium on the Flexural Mechanics of Reinforced Concrete*, ASCE-ACI, Miami, 85-142

Dhakal RP, Pan TC, Irawan P, Tsai KC, Lin KC and Chen CH (2005) "Experimental study on the dynamic response of gravity-designed reinforced concrete connections," *Engineering Structures*, 27, 75–87

Genesio G and Sharma A (2010) "Seismic retrofit solution for reinforced concrete exterior beam-column joints using a fully fastened haunch - Part 2-1: As-built joints" *Test Report No. WS 221/07 - 10/01*, IWB, University of Stuttgart 2010

Kupfer H and Bulicek H (1991), "A consistent model for the design of shear reinforcement in slender beams with I- or Box-shaped cross section", *International workshop on concrete shear in earthquake*, Houston, Tex., 256-265

Park R and Paulay T (1975) Reinforced Concrete Structures, John Wiley & Sons, New York

Park R, Priestley MJN and Gill WD (1982) "Ductility of square-confined concrete columns," *Journal of Structural Engineering*, ASCE, V. 108, ST4, April 1982, pp 929-950

Paulay T and Priestley MJN (1992) Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley and Sons, New York

Priestley MJN (1997), “Displacement based seismic assessment of reinforced concrete buildings” *Journal of earthquake engineering*, 1(1): 157-192

Sharma A, Eligehausen R and Reddy GR (2011) “A new model to simulate joint shear behavior of poorly detailed beam–column connections in RC structures under seismic loads, part I: exterior joints,” *Engineering Structures*, 33:1034-1051

Sharma A, Reddy GR, Vaze KK and Eligehausen R (2013) “Pushover experiment and analysis of a full scale non-seismically detailed RC structure,” *Engineering Structures*, 46: 218-233

Sharma A (2013) Seismic Behavior and Retrofitting of RC Frame Structures with Emphasis on Beam-Column Joints – Experiments and Numerical Modeling, PhD Thesis, IWB, University of Stuttgart.

Watanabe F and Lee JY (1998) “Theoretical prediction of shear strength and failure mode of reinforced concrete beams,” *ACI Structural Journal*, 95(6): 749-757

Wong HF and Kuang JS (2008), “Effects of beam-column depth ratio on joint seismic behavior,” *Structures and Buildings*, 161(2): 91-101