INELASTIC SEISMIC ANALYSIS OF RC FRAMED STRUCTURES CONSIDERING JOINT DISTORTION

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ABSTRACT

Seismic performance of reinforced concrete (RC) framed structure can be assessed with various analytical tools that may broadly be classified as linear elastic procedures and non-linear or inelastic analysis procedures. Since the reinforced concrete structures generally go in the inelastic range due to seismic loading, inelastic procedures predict the performance of the structures in a much better and realistic way than the linear elastic procedures. However, at the same time, the inelastic procedures are computationally much more demanding. Thus, a good balance between accuracy and computational effort is often sought for. Often, important structures are analyzed using inelastic procedures so that the actual performance of the same can be assessed under earthquakes, whereas less important structures are analyzed using linear procedures. To assess the seismic behavior of RC framed structures, various experimental procedures are used. Monotonic pushover tests give information about the load carrying and deformational capacity of the structure along with sequence of failure modes but only in one direction. Static cyclic tests, where inertia effects are not included give the above mentioned information for to and fro loading direction along with the information on energy consumption. Shake table tests, which are closest to the real life earthquake tests provide almost all the information required to understand the seismic behavior but the scale of such tests are usually limited by the capacity of the shaking table facility.

Corresponding to the experimental procedures, there are inelastic analysis procedures such as nonlinear static pushover analysis, static-cyclic analysis and inelastic dynamic analysis. Each analysis procedure can provide the corresponding information, as obtained from experiments, about the structure analytically. Each analysis type has its strengths and drawbacks and just one type of analysis may not be recommended for all types of structures. The choice of the analysis procedure can be made on the basis of the importance of the structure, how accurate the analysis needs to be, how detailed is the information sought for, how much safety margin is economically, technically and practically possible etc. One of the biggest challenges in order to obtain the realistic response of the structure analytically is the modeling of beam-column connections. It is well known that the response of beam-column joints highly influence the overall behavior of the structure under seismic loads, especially in case of non-seismically designed structure where the beam-column connections are often found to be the most vulnerable zone. Ignoring the inelastic behavior of these joints can lead to devastating failures in structures. Many different ways for modeling the joints are proposed by several researchers but the models are generally too complex to be implemented at the structural level, and therefore joints are still mostly not modeled in the analysis. Similarly, associating a hysteretic rule that is not too complex but is still realistic enough is very important. In this work, practically usable and accurate models are reported to realistically model the inelastic response of the structures considering joint distortion. A new model to consider the inelastic behavior of the joints of poorly detailed structures is developed and presented. A practical hysteretic rule based on the extension of "Pivot hysteretic model" is developed for members and beam-column joints and the same is also reported. The analytical models are validated against the experimental results using pushover analysis and static-cyclic analysis at structural level. The efficiency of the models developed in predicting the seismic response of structures considering joint distortions is shown.

INTRODUCTION

Majority of structures worldwide, including nuclear safety related structures fall under the category of reinforced concrete (RC) frame structures. Under seismic loads, invariably such structures undergo inelastic distortions. The old structures designed, detailed and constructed according to non-seismic provisions are more susceptible to large inelastic deformations and often display poor brittle failure mainly due to the failure of joints. 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 entering 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 [1, 2, 3]. 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 [4, 5, 6]. 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. However, a vast majority of the structures world wide consist of structures designed prior to the advent of seismic design codes.

While designing a structure, conventionally, the joint core is considered as rigid and the frame members are assumed to be connected forming a single node that symbolizes the joint. New codes [4,5,7] suggest an indirect approach to design the joint by limiting the joint shear stresses. However, again in older codes, no such provisions existed. Even in the case of nonlinear displacement based analysis (e.g. Pushover) all the plastic rotations are assumed to occur in the beams and/or columns forming the joint core, which is un-conservative. 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 nicely. 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. Since earthquake generates a cyclic loading on the structure, in order to completely understand the seismic behavior of structures, reliable but practical hysteretic models are required. The hysteretic model shall be applicable not only to the members but beam-column joints as well.

In this work, a spring based model for predicting the inelastic shear behavior of beam-column joints is presented that can reasonably accurately capture the shear behavior of the joints and also is practical enough to be used at structural level with existing commercial software programs available. Also, a new hysteretic model based on the so called pivot hysteretic model is also proposed that can consider the hysteretic behavior of members as well as joints. The models are used to perform inelastic seismic analysis of non-seismically detailed frame structures. Comparison of experimental and analytical results of two structures under static loads and one under quasi-static-cyclic loads proves the efficacy of the models in capturing the seismic behavior and failure modes for the structures.

SPRING BASED JOINT MODEL

Model Formulation

The model proposed in this work uses limiting principal tensile stress in the joint as the failure criteria 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 (Fig 1). Based on the deformational behaviour of joint, the contribution of joint shear deformation to overall storey drift is modelled in a way that can consider the shear deformations in column and rotation in beam due to joint shear deformation. This is done as shown in Fig 2 (a) where shear springs in the column portion and a rotational spring in the beam region are assigned. Thus, in a frame analysis by software packages, in addition to springs assigned at the ends of the members (beams and columns), a joint core may be modelled by dividing the frames and springs are provided in the core region to consider the shear deformations of the joint as shown in Fig 2 (b).

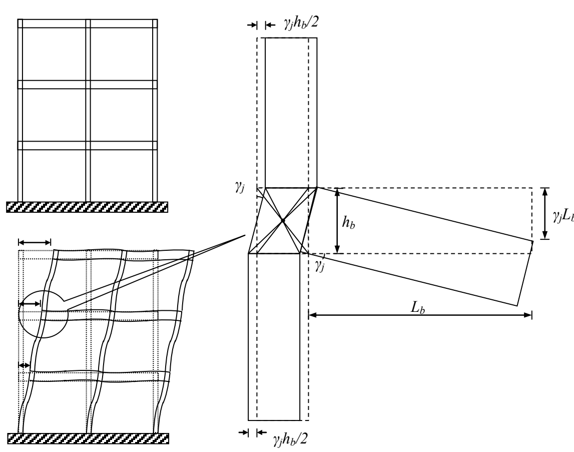
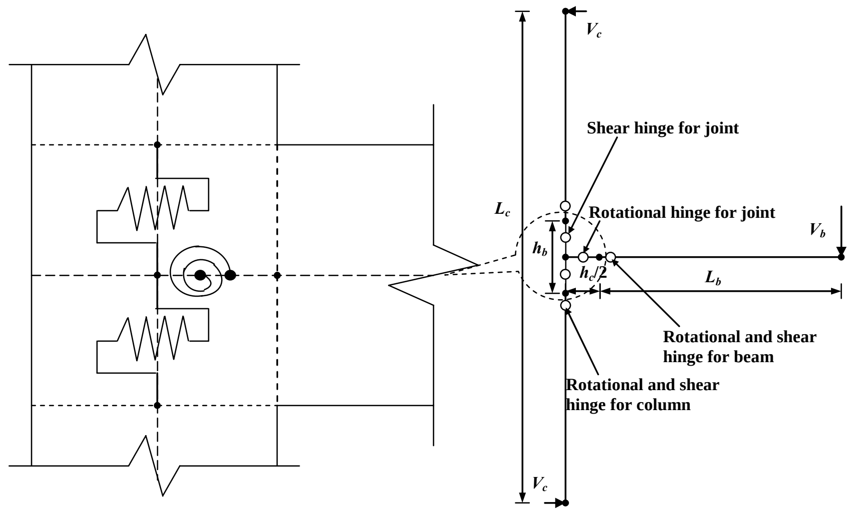


Fig 1 Contribution of joint deformation to storey drift

To determine the spring characteristics to model the joint core, certain failure criterion is required. In order to give due consideration to the axial loads acting on the column, critical principal tensile stress is selected as the failure criterion. For beam-column joints with the beam bars bent in the joint, the curve for principal tensile stress v/s shear strain that was validated and used in this work is shown in Fig 3. This curve is based on the recommendations by Priestley [8]. The rotational and shear spring characteristics are derived from equilibrium criteria for the joints.



(a) Philosophy of joint core modelling (b) Modelling of joint in frame analysis
Fig 2 Principle behind proposed Joint Model

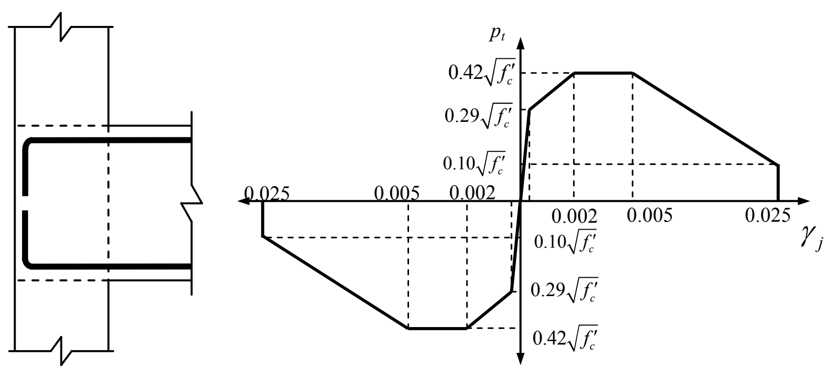
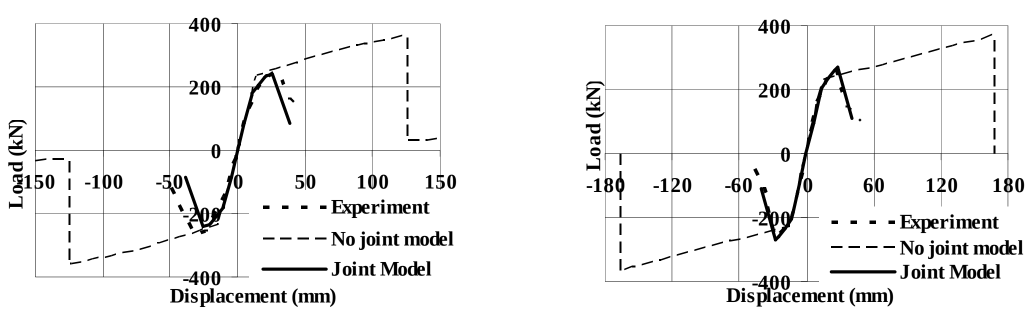


Fig 3 Principal tensile stress-shear deformation relation for joints with bars bent-in

Validation of Proposed Model with Experiments



(a) Joint with 10% axial load (b) Joint with 25% axial load
Fig 4 Validation of model with results of tests performed by clyde et al [9]

The proposed model was validated with experiments at joint level. Nonlinear static pushover analysis was performed to get the load-displacement behaviour of the joints that was compared with the envelope of the hysteretic loops obtained from the experiments. Fig 4 shows the result of two beam-column joints reported in literature by Clyde et al [9]. The results are shown for the joints tested with (a) 10% axial load on column and (b) 25% axial load on column. Two types of analysis were performed, one considering the joint springs as per the proposed joint model and another without considering nonlinearities in the joint (no joint model case). Fig 4 clearly shows that analysis without considering joint nonlinearity may lead to quite un-conservative results whereas the results obtained for the with joint model case are very close to experimental results.

HYSTERESIS MODEL

Model Formulation

Concrete is known to be a highly nonlinear material which tends to undergo severe hysteresis beyond yield. In case of reinforced concrete, the hysteretic behavior becomes more complex due to the effect of reinforcing steel and the behavior of bond between concrete and reinforcement under cyclic loads. Over the years, various researchers have proposed various models to predict the hysteretic behavior of concrete and reinforced concrete structural elements. In the past, elasto-plastic hysteretic rules that idealize the hysteretic loops in bilinear format were frequently used. Such an idealization, though reasonable for steel members, were found to be an over-simplification for concrete members. Some other models give more consideration to effects like stiffness degradation, pinching due to opening and closing of cracks, bond slip etc but they become computationally more demanding. In 1998, Dowell et al [10] proposed a so-called “Pivot hysteretic model” for reinforced concrete members. The model was basically developed for circular columns with the aim of serving the need of inelastic analysis of bridge pier columns. The model could consider the effect of axial load, lack of section symmetry and strength degradation. The model is quite simple and effective in modeling the force-deformation or moment-rotation response for the members. However, the model could not be directly applied to rectangular columns and beam-column joints.

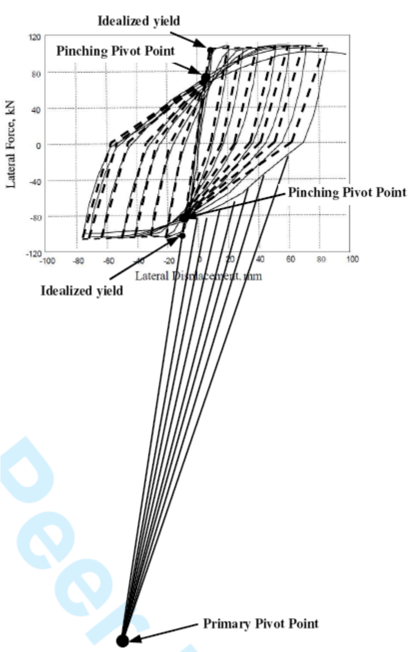


Fig 5 Experimental hysteretic response and idealization

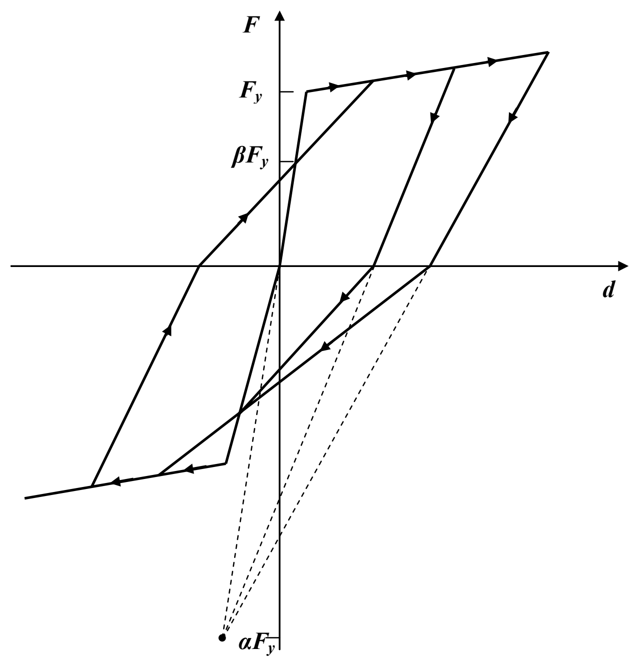


Fig 6 Basic pivot hysteretic model

Given the simplicity and ease of applicability of the model, in this work, the pivot hysteretic model is extended to appropriately model the hysteretic response of rectangular reinforced concrete members (beams and columns) and beam-column joints. The assumptions of the model are based on observations from a large set of experimental data on RC members. Fig 5 shows an experimental hysteretic response for a RC rectangular column testes under cyclic loads by Ohno and Nishioka [11] and its corresponding idealization. Fig 6 shows the basic pivot hysteretic model where the hystertic response is controlled by two parameters namely ‘α’ and ‘β’. It is considered that while unloading, the load deflection path is guided towards a common point known as primary pivot point (αF_y). Also, it is observed from the experimental results that the force-displacement paths tend to cross the elastic loading line at approximately the same point called pinching pivot point (βF_y). Thus, if these two parameters, namely 'α' and 'β' can be assigned to the nonlinear springs for members and joints, the hysteretic rules can be set.

In the original model [10], contours for 'α' and 'β' were provided for circular columns as a function of axial load ratio and longitudinal reinforcement ratio. The contours for 'α' and 'β' were developed using the fiber element analysis of various circular column RC columns and were validated with the experimental results. Since, the model was originally developed for circular bridge columns, the axial load ratio was considered only up to 20% of the axial load capacity of the column, as in bridge columns, the axial loads seldom go higher than 20% of its capacity. However, in case of columns of framed structures, the axial load level can be much higher and therefore, the charts for 'α' and 'β' are required for the complete range of axial load levels. Also, both 'α' and 'β' are considered to be varying only with respect to longitudinal reinforcement and axial load ratio, neglecting the transverse reinforcement ratio, while it is well-known that confining reinforcement governs the energy dissipation characteristics of RC members. In this extension of pivot model, the variables to control the value of 'α' are considered same, i.e. axial load ratio and longitudinal reinforcement ratio, whereas in addition to the axial load ratio, transverse reinforcement ratio instead of longitudinal reinforcement ratio was considered as a better variable to control the value of β since pinching is known to be significantly influenced by transverse reinforcement. Based on a large database of experimental results on rectangular RC columns and beam-column joints, the following equations were suggested to evaluate the 'α' and 'β' parameters.

For rectangular beams and columns:

$$\alpha = 0.170k_\alpha + 0.415 \tag{1}$$

$$\beta = 0.485k_\beta + 0.115 \tag{2}$$

Where,

$$k_\alpha = p_t \,/\, \mathrm{ALR} \tag{3}$$

$$k_\alpha = (\mathrm{ALR})^{0.25}(\rho_{sh})^{0.2} \tag{4}$$

$$\mathrm{ALR} = \text{Axial Load Ratio} = \frac{\text{Axial load on the beam or column}}{\text{Axial load capacity of the beam or column}} \tag{5}$$

p_t = percentage of longitudinal reinforcement in the beam or column

ρ_{sh} = volumetric transverse reinforcement ratio in the beam or column

Similarly, for beam-column joints with non-seismic detailing and beam bars bent in (Fig 3):

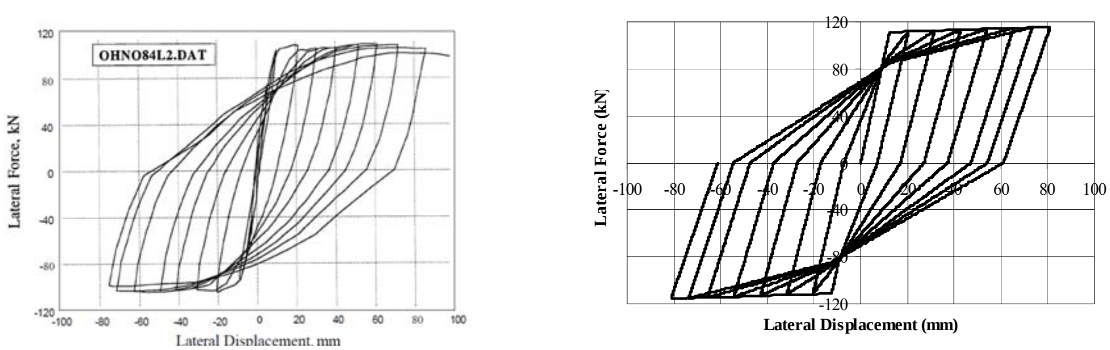
$$\alpha = 4.4 - 25.38(\mathrm{ALR}),\ \text{for } 0 \le \mathrm{ALR} \le 0.13\text{, and} \tag{6}$$

$$\alpha = 1.0,\ \text{for ALR>0.13} \tag{7}$$

$$\beta = 0.125 + 0.44(\mathrm{ALR}) \tag{8}$$

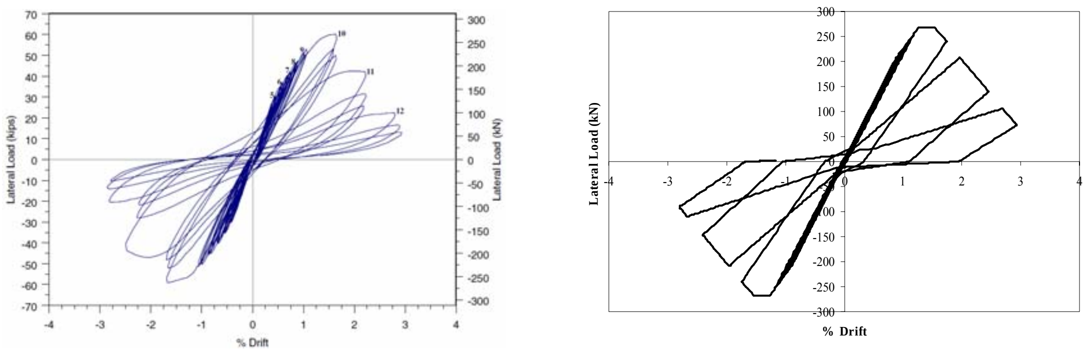
Validation of Proposed Model with Experiments

Fig 7 shows the comparison of the experimental and analytical results for a column tested by Ohno and Nishioka [11] and Fig 8 shows the same for a joint with 25% axial load tested by Clyde et al [9]. Based on the excellent agreement between the experimental and analytical results, it can be said that the hysteresis model can be successfully employed to predict the hysteretic response of RC structural members as well as joints.



(a) Experiment (b) Analysis

Fig 7 Comparison of experimental and analytical hysteretic response for RC column



(a) Experiment (b) Analysis

Fig 8 Comparison of experimental and analytical hysteretic response for RC beam-column joint

INELASTIC SEISMIC ANALYSIS OF STRUCTURES

Monotonic pushover analysis

Two structures tested under pushover loads were analyzed. Structure 1 was a three storey structure with a column height of 1.8 meters. The plan dimensions were 3m x 3m, with bay width of 1.5 meters in both directions. Fig 9 shows the general geometric arrangement of the structure. The typical beam and column size was 150mm x 200mm and the slab thickness was 100mm. Structure 2 was a full scale replica of an existing office building. The typical beam size was 230mm x 1000mm and the column size varied from 400mm x 900mm to 300mm x 700mm. The slab thickness was 130mm. Fig 10 shows the general geometric arrangement of the structure 2.

Fig 9 Structure 1 being tested

Fig 10 Structure 2 being tested

In the analytical model, flexural hinges were generated in the form of moment-rotation characteristics using the Modified Kent and Park model [12]. The shear hinge properties for the members were generated using the Watanabe and Lee model [13] whereas the torsion hinge characteristics were obtained following the procedure recommended by Park and Pauley [14]. The joint hinges were modeled following the joint model as explained earlier. Three different analyses were performed for both the cases. In the first case, only the flexural and shear hinges were considered for the members. In the second case, additionally torsion hinges were considered and in the third case, joint hinges in addition to flexural, shear and torsional hinges were considered. Fig 11 shows the pushover curves obtained for structure 1 and Fig 12 shows the same for structure 2. It can be observed that in both the cases, the model with the joint hinges modeled along with other hinges (flexural, shear and torsion) yields the best results and that not modeling the joint hinges can lead to significantly un-conservative results, thus highlighting the importance of considering joint distortion.

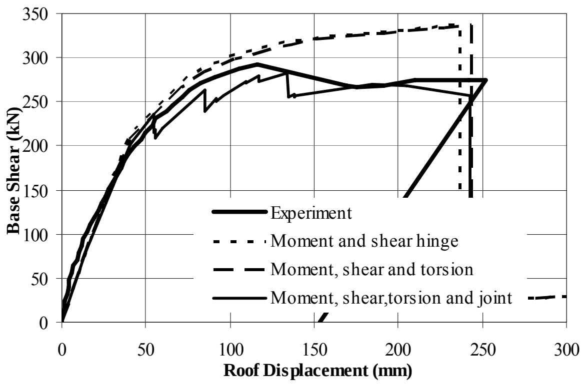


Fig 11 Results for structure 1

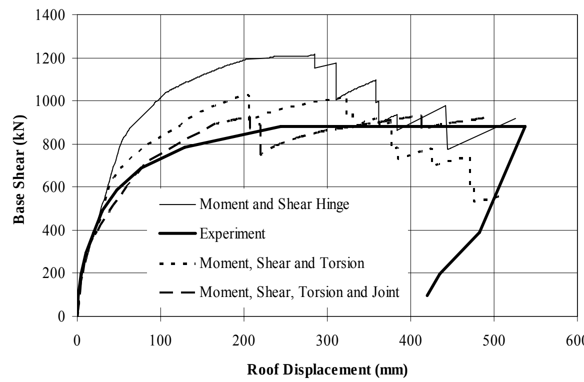


Fig 12 Results for structure 2

Cyclic Analysis

In this work, a RC frame structure tested by Calvi et al [15] that was a 2/3rd scale replica of a three-storey structure, was analyzed. The geometrical and reinforcement characteristics of the test can be obtained from ref [15]. To simulate a pre-1970's construction type, no transverse reinforcement was placed in the joint region and plain round bars were adopted for both longitudinal and transverse reinforcement. Beam bars in exterior joints were anchored with end-hooks. Fig 13 shows the experimental hysteretic loops as obtained for the structure [15] and Fig 14 shows the failure patterns as observed during the test. It was reported that the damage was mostly concentrated in the exterior tee-joints or at the beam/column interfaces.

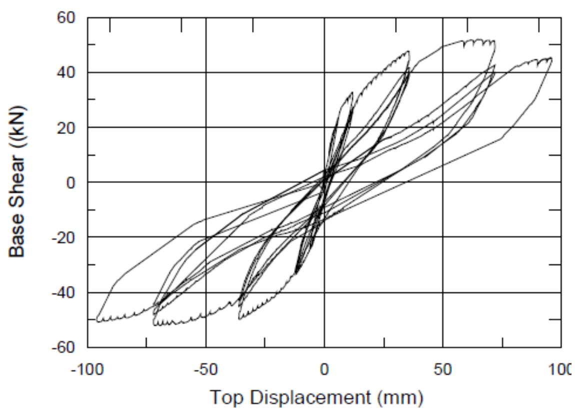


Fig 13 Experimental hysteretic plots [15]

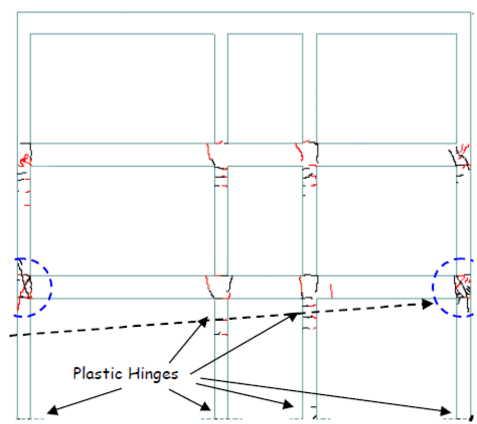


Fig 14 Experimental failure patterns [15]

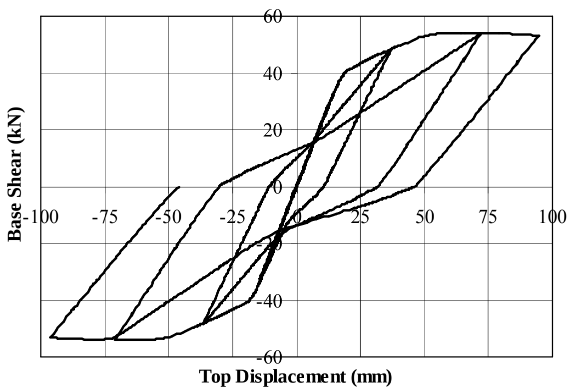


Fig 15 Analytical hysteretic plots

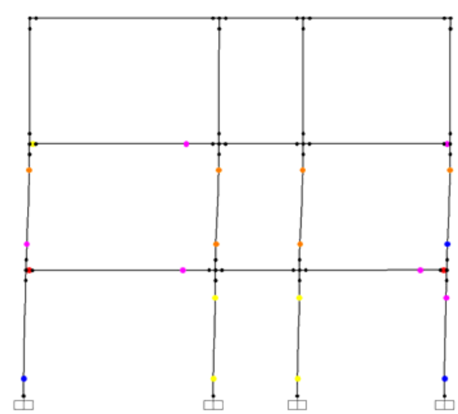


Fig 16 Analytical failure patterns

Fig 15 shows the hysteretic plots obtained for the structure from inelastic quasi-static-cyclic analysis and Fig 16 shows the analytically predicted damage pattern for the structure. Since, in this test, the reinforcement was made up of plain round bars with the beam bars anchored via end hooks in the joint region, principal tensile stress v/s shear deformation curve as recommended by Pampanin et al [16] was followed. Also, based on test data and as recommended by Dowell et al (1998), the α and β parameter were taken as $2/3^{rd}$ of the values obtained by eqs (1) through (8). The comparison of the hysteretic loops as well as failure patterns clearly demonstrates the capability of the models to predict the seismic behaviour of RC structures.

CONCLUSION

In this work, inelastic seismic analysis of three structures considering joint inelastic behavior was presented. The significance of joint modeling in order to assess the overall seismic behavior of reinforced concrete structures is highlighted. A joint model was described and was validated against the experimental results of beam column joints. The model was then applied at the structural level for two different structures tested under pushover loads. It has been shown that the model is easy to implement at the structural level and can be used for correct prediction of inelastic behavior of reinforced concrete structures. It is also shown that non-modeling of the inelastic behavior of beam-column joints can lead to quite un-conservative results.

Further, an extension of pivot hysteretic model was presented that could be applied for rectangular beams and columns as well as non-seismically detailed joints. The model was validated against experimental results at the member level as well as joint level. The model along with the joint model was applied for the inelastic cyclic analysis of an RC framed structure tested by Calvi et al [15]. It was shown by the comparison of the experimental and analytical hysteretic loops as well as failure patterns that the models are capable of predicting the seismic behaviour of RC structures very well.

REFERENCES

[1] Paulay, T., Priestley, M.J.N., “Seismic design of reinforced concrete and masonry buildings” *John Wiley publications*, New York, 1992.
[2] Paulay, T., Park, R., “Joints of reinforced concrete frames designed for earthquake resistance”, *Research report 84-9*, department of civil engineering, University of Canterbury, Christchurch, 1984.
[3] Paulay, T., Park, R., Priestley, M.J.N., “Reinforced concrete beam-column joints under seismic actions”, *ACI Journal* 1978; 75-60: 585-593.
[4] ACI 318M-08, “Building Code Requirements for Reinforced Concrete”, *American Concrete Institute*, Detroit, Michigan, 2008
[5] NZS 3101, “Design of Concrete Structures”, *Standards Association of New Zealand*, Wellington, 1995
[6] IS 13920:1993, “Indian Standard ductile detailing of reinforced concrete structures subjected to seismic forces - Code of Practice”, *Bureau of Indian Standards*, New Delhi, 2002
[7] CEN Technical Committee 250/SC8, “Eurocode 8: Earthquake resistant design of structures – part 1: General rules and rules for buildings” (ENV 1998-1-1/2/3), *CEN, Berlin*, Germany.
[8] Priestley, M.J.N., “Displacement based seismic assessment of reinforced concrete buildings” Journal *of earthquake engineering*, 1997; 1(1): 157-192.
[9] Clyde, C., Pantelides, C.P., Reaveley, L.D., “Performance-based evaluation of exterior reinforced concrete building joints for seismic excitation”, *Report PEER 2000/05*, July 2000
[10] Dowell, R.K., Seible, F., Wilson, E.W., “Pivot hysteresis model for reinforced concrete members”, ACI Structural Journal, V. 95, No. 5, September-October, 1998, pp 607-617.
[11] Ohno, T. Nishioka, T. “An Experimental Study on Energy Absorption Capacity of columns in Reinforced Concrete Structures”, *JSCE, Structural Engineering/Earthquake Engineering*, V.1, N.2., pp. 137-147.
[12] Park, R., Priestley, M.J.N., and Gill, W.D., “Ductility of square-confined concrete columns”, *Journal of Structural Engineering*, ASCE, Vol. 108, ST4, April 1982, pp 929-950.
[13] Watanabe, F., Lee, J.Y., “Theoretical prediction of shear strength and failure mode of reinforced concrete beams”, *ACI Structural Journal*, V.95, No.6, November-December 1998, pp. 749-757.
[14] Park, R., Paulay, T., “Reinforced Concrete Structures”, *John Wiley Publications*, New York, 1975.
[15] Calvi, G.M., Magenes, G. and Pampanin, S., “Experimental Test on a Three Storey R.C. Frame Designed for Gravity Only”, *12th European Conference on Earthquake Engineering*, London, paper n. 727, 2002.
[16] Pampanin, S., Magenes, G., Carr, A., “Modelling of shear hinge mechanism in poorly detailed RC beam-column joints”, *Proceedings of the FIB 2003 Symposium*, May 6-8, 2003 Athens, Greece.