STRUCTURAL APPLICATIONS OF A REINFORCED CONCRETE BEAM-COLUMN-SLAB CONNECTION MODEL FOR EARTHQUAKE LOADING

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ABSTRACT :

Analytical studies to evaluate the seismic response of reinforced concrete moment resisting frame structures, with and without joint deformations, demonstrated that the predicted inelastic behavior was not accurate when the joint region was assumed to be rigid. One of the main purposes of the research program was to develop a joint model that accounts for deterioration of shear strength and stiffness within the connection region and concentrated rotation due to rebar slip. The experimental data on joint distortion was used to develop and verify an analytical model that includes shear deformation and concentrated joint rotation components. The developed joint deformation model was then used in the nonlinear analysis of reinforced concrete buildings that provide different frame configurations and inelastic responses. The structures were subjected to inelastic dynamic time history analyses, using a variety of earthquake records that represent different levels of seismic risk. It was observed that if a deformable joint model was not included in the structural model, story drifts were underestimated significantly. Results of the nonlinear analyses for a selected building and brief information on the member modeling, including the joint model developed for this research program, are presented in this paper.

KEYWORDS: beam-to-column connections, joint deformation model, earthquake loading, shear deformation, nonlinear analysis

1. INTRODUCTION

While analyzing reinforced concrete frame buildings, the structure is defined as a set of various members. In current practice, although detailed inelastic beam and column members are used in structural analysis, the connection regions are generally modeled as rigid zones and the inelastic activities in the joint are not represented. In some cases, member models for beam and column elements may be adjusted to represent damage in the joints. However, when such a modeling procedure is used, there is no direct feedback to assess potential joint damage and to determine the effect of that damage on selecting the performance level for the frame.

Prior analytical studies of the seismic response of reinforced concrete moment resisting frame structures had indicated that the predicted inelastic behavior was not always accurate if the joint region was assumed to be rigid. When beam-to-column connections are modeled as rigid zones, the total story drift could be underestimated, and this may result in an improper evaluation of structural performance. Recent experimental results (Burak and Wight 2008) showed that the joint deformations could contribute up to 40% of the total story drift when a reinforced concrete beam-column-slab subassembly was at 2% story drift.

Therefore, to represent the structural behavior more realistically, either an independent joint model, or components that can be added to frame member models should be included in the nonlinear analysis of frame structures. This joint model should account for joint deformations resulting from rebar slip or pullout from the joint, and deterioration of shear strength and stiffness within the joint. Although these components could be modeled more precisely using a finite element model, such a procedure would not be practical for implementation of the push-over or dynamic analysis procedures to full frame structures.

2. MEMBER MODELING

To determine the seismic response of the reinforced concrete frame structures in a more realistic manner, member models were developed and calibrated using the experimental results. The member models were calibrated by applying the displacement history used in the experimental program to the top of the column in the nonlinear analysis. After some trial runs, the main parameters were established for each individual member model, the details of which are given below.

2.1. Beam Element

The beam element was modeled as an elastic segment with zero-length moment hinges at the column faces and rigid end zone elements within the column, as illustrated in Fig. 1 (a). The rigid end zone length was selected as half the column width. The beam moment vs. rotation relationship is shown in Fig. 1 (b).

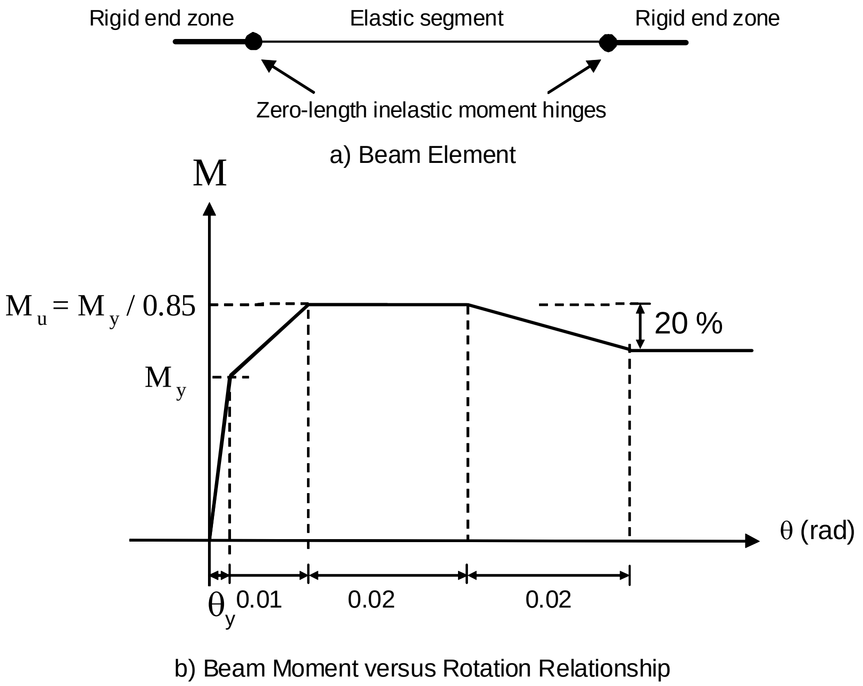


Figure 1 Beam Model

The main parameters that are required to define the elastic beam behavior are section dimensions, moment of inertia, I, modulus of elasticity, E, and Poisson's ratio, ν. The moment of inertia was taken as the cracked moment of inertia and set equal to the 35% of that for the gross section, which consisted of the beam and an effective slab width. The modulus of elasticity was computed from the actual material properties and Poisson's ratio was taken as 0.2. For the beam plastic hinge spring, initial stiffness is taken as a large value to prevent rotation before yielding. After the yield moments were obtained, a strain hardening ratio of 0.03 x 6 E_c I_b /L_b was used to compute ultimate moment strength. The rotation between the yield and ultimate moments was taken as 0.01 rad. Between 0.01 rad. and 0.03 rad. the moment remained constant. Then, 20% strength reduction was applied between 0.03 rad. and 0.05 rad. considering FEMA 356 recommendations. The yield curvature of the beam was found by using the actual material properties, and this was converted to the yield rotation by assuming an inelastic zone length of half the beam depth. Other rotation values corresponding to key moment values in Fig. 1 (b) were determined based on the test results. A 10% strength decrease was assumed to occur at large rotations.

Different energy dissipation coefficients were specified at different critical rotation values to account for stiffness deterioration. Based on the dissipation factors, the software reduces the area within the hysteresis curves proportional to the dissipation factor.

2.2. Connection Element

The inelastic connection panel zone in Perform-3D was used as the joint model. This element consists of four rigid links connected by hinges one of which has an embedded nonlinear rotational spring. The parameters required for this spring are the key joint shear deformation points and moments created due to shear stresses.

From a parametric study, the yield joint shear distortion which is defined as the joint shear deformation just before the yielding of stirrups in the connection region was obtained. This value depends on the parameters such as material properties f_c' and f_y, the reinforcement ratio of joint stirrups considering one layer of stirrups and their effective area, the confinement of the connection region provided by the framing beams and the column aspect ratio. Then, other key distortions were obtained as multiples of this value. Effective joint width was taken as the average of beam and column widths, $(b_b+b_c)/2$, as recommended by LaFave et.al. (2005). Joint shear stresses were computed considering the same parameters. To obtain the connection moment capacity, the horizontal joint shear strength was multiplied by a level arm equal to the distance between the top and bottom reinforcement of the beams framing into the column. The moment vs. shear deformation relationship for the connection region is shown in Fig. 2.

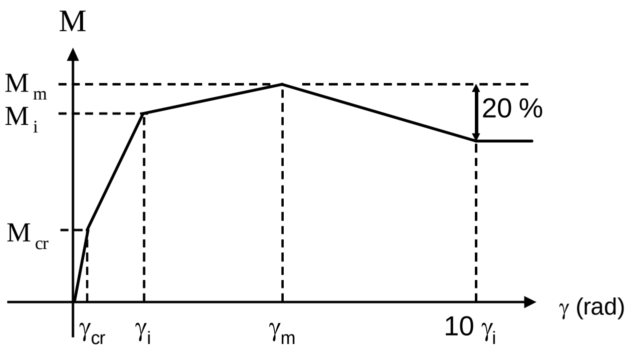
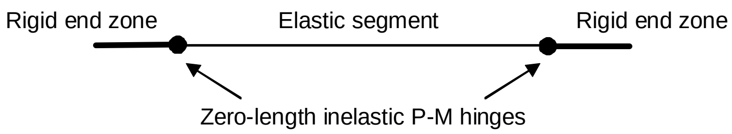


Figure 2 Joint Model

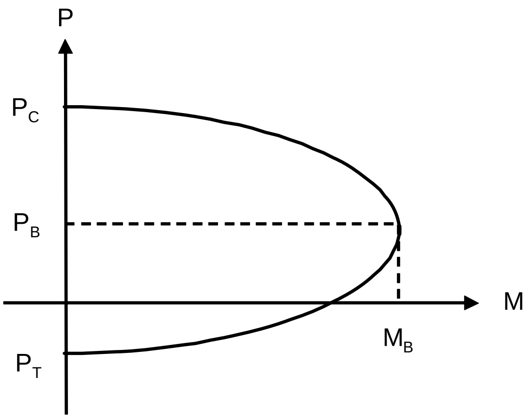
2.3. Column Element

The column element was modeled as an elastic segment with zero-length moment hinges at the beam faces and rigid end zone elements within the beam, as illustrated in Fig. 3 (a). The inelastic activity observed in the columns was not as significant as in the beams and they remained elastic for most of the test. So, the zero-length moment vs. axial load rotation element in Perform-3D was an appropriate element for modeling the column behavior. The rigid end zone was taken as half the beam height for the column members.

The main parameters that are required to define the elastic column are section dimensions, moment of inertia, I, modulus of elasticity, E, and Poisson's ratio, ν. The moment of inertia was taken as the cracked moment of inertia, which was assumed to be equal to 70% of that for the gross section. The modulus of elasticity was computed from the actual material properties and Poisson's ratio was taken as 0.2. The column section yield surface is given in Fig. 3 (b). The only moment value required is the balanced moment capacity of the column, computed using a linear strain distribution with a maximum compression strain of 0.003 at the compression edge of the concrete section and a yield strain at the level of the outermost tension reinforcement. The axial loads corresponding to pure axial compression and concentric axial tension failure were also required. These values and two other parameters, which were used in defining the shape of the relationship between moment and axial load, were used to define the yield surface. A bilinear relationship was assumed for moment vs. rotation and an elastic one for axial load vs. displacement, with the ultimate values of balanced moment and pure axial compression, respectively.



a) Column Element



b) Column Axial Load versus Moment Relationship

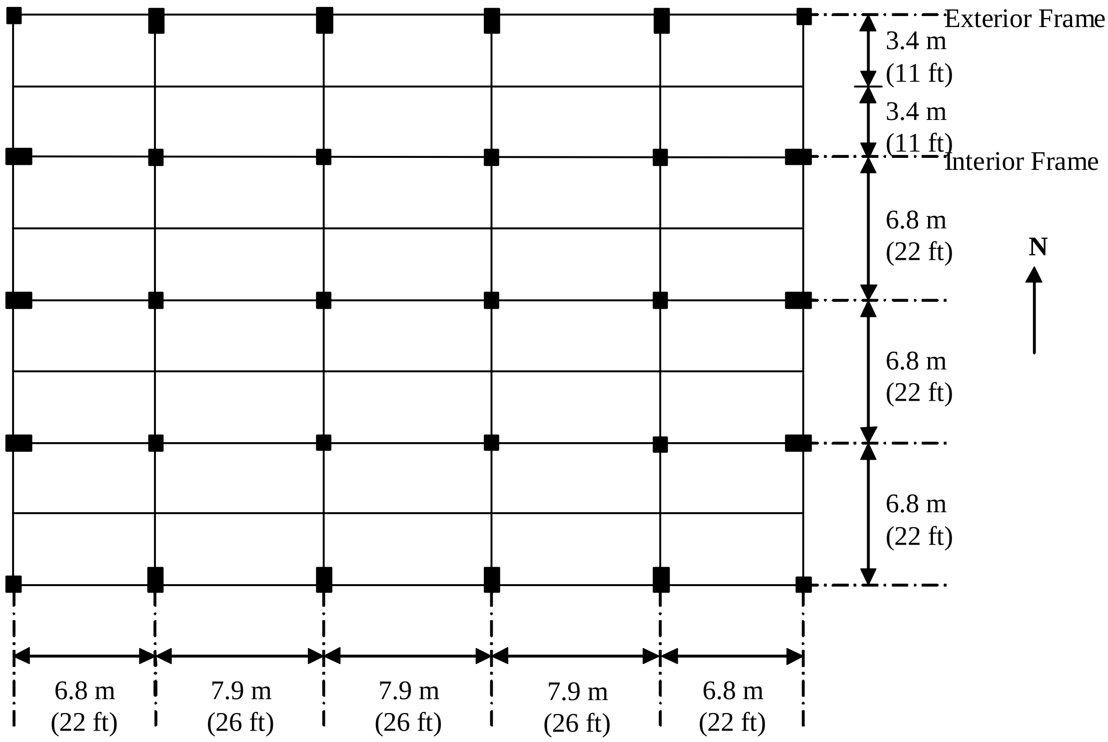
Figure 3 Column Model

3. DYNAMIC TIME HISTORY ANALYSES OF A REINFORCED CONCRETE BUILDING

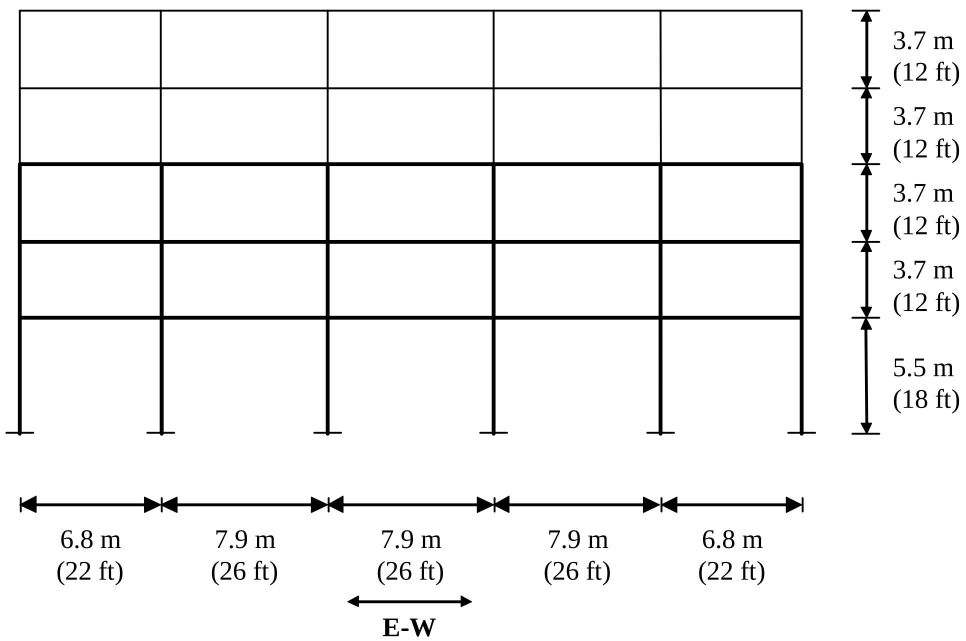
A five-story reinforced concrete moment resisting building with eccentric spandrel beams in the exterior frames and concentric beams in the interior frames that was designed by Burak (2005) was used in the dynamic analysis. The frames were designed using the specifications of the ACI 318 Building Code (2002), and ACI-ASCE Committee 352 (2002) design recommendations for beam-to-column joints. The seismic design forces were computed using the International Building Code (IBC 2000). The building represented an office building located in Los Angeles, and plan and elevation views of the building are shown in Fig. 4. The beam and column sizes were reduced for the top two stories where the strength demand was lower.

Rectangular columns with the same size and reinforcement detailing were used for the interior columns of the exterior frame and the exterior columns of the interior frame, in different orientations. Each column had a reinforcement ratio more than 1%. The exterior frame had corner columns of 51 x 51 cm (20 x 20 in.) and interior columns of 46 x 61 cm (18 x 24 in.) oriented in the weak axis. For the top two stories, each column dimension was reduced by 5 cm (2 in.). The exterior columns of the interior frame are 46 x 61 cm (18 x 24 in.) oriented in the strong axis, while the interior columns are the same as the corner columns of the exterior frame (51 x 51 cm (20 x 20 in.)) for the bottom stories. The top stories had column dimensions 5 cm (2 in.) less than the bottom stories.

The eccentric spandrel beams of the exterior frame for the bottom stories were 30 x 53 cm (12 x 21 in.). For the top floors, the dimensions were reduced to 25 x 46 cm (10 x 18 in.). For the interior frame, the concentric regular beams were 36 x 53 cm (14 x 21 in.). The regular beams for the top floors were 30 x 46 cm (12 x 18 in.).



a) Plan View



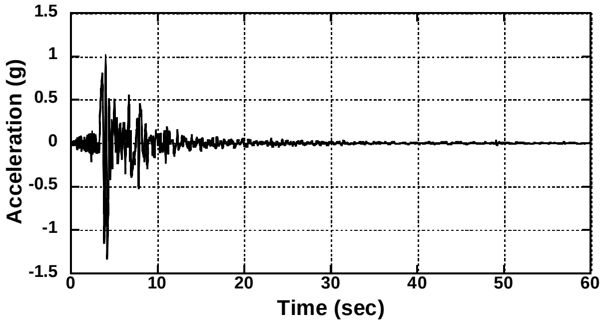
b) Elevation View

Figure 4 Plan and Elevation View of the Building used in the Analytical Study

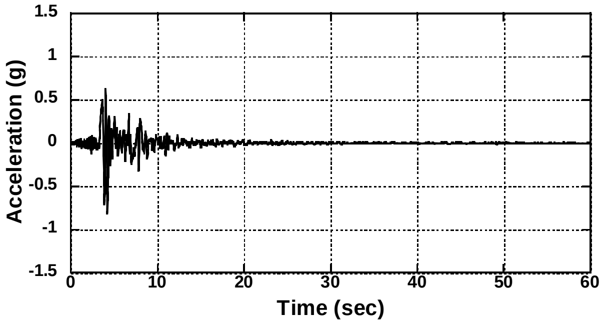
In half of the dynamic nonlinear analyses, the developed joint model was used, while in the other half, the joint regions were assumed to be rigid to evaluate the effect of joint shear distortions on the story drift demands and overall structural response.

In each analysis, total design dead load and 25% of the reduced live load was included when computing the seismic mass of a story, which was lumped at one of the middle nodes. The floor diaphragm was assumed to be rigid at each story level. Before the models were subjected to earthquake ground motions, gravity load was applied to the structure as a uniformly distributed load including full dead load and 25% of the live load.

The effect of earthquake intensity on the building behavior was studied by using two records with different scaling factors. The Sylmar records were selected from the ground motions developed for the SAC project (Somerville *et al*. 1997) to represent two different probabilities of exceedance, 10% in 50 years (10/50) and 2% in 50 years (2/50), respectively (Fig. 5).



a) Sylmar 2/50 Record

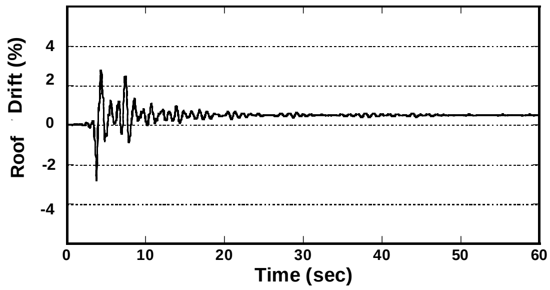


b) Sylmar 10/50 Record

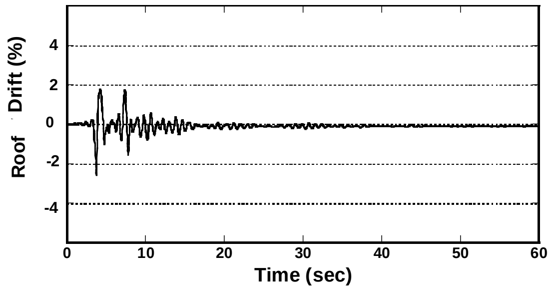
Figure 5 Acceleration Time-History of the Selected Ground Motions

4. ANALYTICAL RESULTS

When connections were modeled as rigid zones, the maximum roof drift was significantly lower when compared to the analysis carried out by including the joint panel zones in the model. The difference in the roof drifts due to connection modeling goes upto 25% for different earthquake records depending on the joint shear force demands they produced. This difference is important in predicting the maximum story drift of a reinforced concrete frame structure. The story drifts could be underestimated significantly if the connections are modeled as rigid zones, and this may result in an improper evaluation of structural performance.



a) Frame with Joint Model



b) Frame with Rigid Connections

Figure 6 Maximum Roof Drift

The interstory drifts were also affected significantly by including the joint panel zones in the frame modeling. For all the frames and the applied earthquake records, the maximum interstory drift was observed between the ground and the first floor and it got lower at each successive story level. The distributions of interstory drift through each story for the exterior frames under the 2/50 Sylmar record are shown in Fig. 6 for the frame with the joint panel zone and that with rigid connections. The response for 2/50 Sylmar record was shown in this figure, because the effect of joint model was more noticeable for this demanding ground motion when compared with the 10/50 record. As can be observed from this figure, the interstory drift could be significantly underestimated if the connections are modeled as rigid zones.

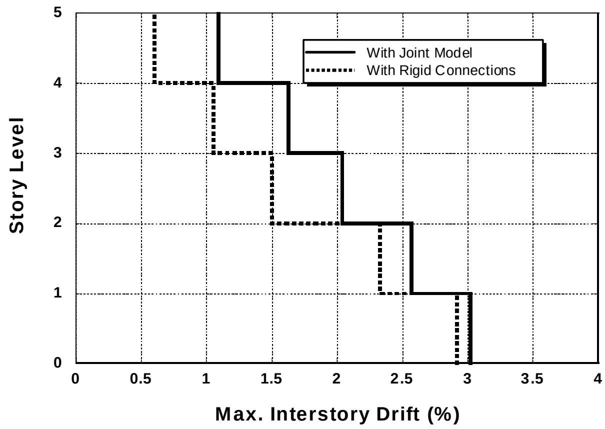


Figure 7 Distribution of Interstory Drift through Each Story

The use of the developed joint model in the time history analyses also changed the structural response. When the connections were considered as rigid zones, the beam plastic rotations increased between 1.5 and 2.0 times of that for the frame with the deformable joint panel zones. The reason for lower beam rotations when the joint model was utilized is the participation of the softer connection region in developing the total deformation due to earthquake loading.

5. CONCLUSION

When the connection regions are considered as rigid zones, the story drifts are significantly underestimated and this could lead to an underestimation of the required stiffness of a reinforced concrete building. Therefore, a joint model that accounts for the inelastic deformations in the beam-to-column connections is required in the dynamic analyses of frame structures to accurately predict the drift demands. This way the inelastic activity in each member including beam-to-column connections could be identified adequately. This is important in identifying the performance of each member, and therefore the structure, under earthquakes with different intensities.

REFERENCES

ACI Committee 318 (2005). Building Code Requirements for Structural Concrete (ACI-318-02), American Concrete Institute, Farmington Hills, MI, USA.

ACI-ASCE Committee 352 (2002). Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures (ACI 352-R02), American Concrete Institute, Farmington Hills, MI, USA.

Burak, B. and Wight, J.K. (2008). Experimental investigation on the seismic behavior of eccentric RC beam-column-slab connections. *ACI Structural Journal* **105:2**, 154-162

Burak, B. (2005). Seismic behavior of eccentric reinforced concrete beam-column-slab connections, Ph.D. Dissertation, University of Michigan, Ann Arbor, MI, USA.

IBC (2000). International Building Code, International Code Council, Falls Church, Virginia, USA.

LaFave, J. M., Bonacci, J. F., Burak, B. and Myoungsu, S. (2005). Eccentric beam-column connections – performance and design of joints subjected to seismic lateral load reversals. *Concrete International* **27:09**, 58-62