Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 652

ANALYTICAL VERIFICATION OF A SIMPLIFIED REINFORCED CONCRETE JOINT MODEL

B. Burak [1]

ABSTRACT

In the analysis of 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 on 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. In this research program a joint model was developed to account for deterioration of shear strength and stiffness within the connection region and concentrated rotation at the member interfaces. The parameters defining the joint model were validated using the data reported in prior experimental studies. The joint deformation model was then used in the analysis of a number of tested specimens that have different configurations and inelastic responses. In the analytical study, the specimen models were subjected to the displacement history used in each representative quasi-static test. The analytical results were then compared with the experimental data to verify the effect of the joint model on the total load-deformation response. Results of the analytical program and the details of the joint deformation model are presented in this paper.

Introduction

Reinforced concrete moment resisting frame structures are commonly used as the primary lateral load resisting system in low to moderate rise buildings in seismically active regions. If these structures are properly designed and detailed, they can resist strong earthquake ground motions. The behavior of beam-to-column connections significantly influences the earthquake resistance of these structures. Therefore, their performance is essential in ensuring the satisfactory seismic behavior of moment resisting frame structures.

---

[1] Assistant Professor, Dept. of Civil Engineering, Middle East Technical University, Ankara, Turkey 06531

For earthquake resistant design of moment resisting frame structures, the "strong column-weak beam" philosophy is used to ensure the development of beam plastic hinging at large displacements, rather than column plastic hinging. To enable full plastic hinging in the beam, the connection should be able to resist all the forces transferred by adjacent beams and columns without the degradation of joint strength and stiffness, and without the loss of anchorage for the beam and column bars.

Joint shear distortions contribute significantly to the total story drift. 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. 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 the implementation of the push-over or dynamic analysis procedures to frame structures.

In this analytical study, a simple joint model was developed that could be incorporated in a commercial software package available to practicing engineers. Previously obtained experimental data on joint distortions was used to develop this model, which accounts for deterioration of shear strength and stiffness within the beam-to-column connection region. This joint model was then calibrated by comparing the analytical results for some test specimens with the experimental data.

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.

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 Figure 1 (a). The rigid end zone length was selected as half the column width. The beam moment vs. rotation relationship is shown in Figure 1 (b).

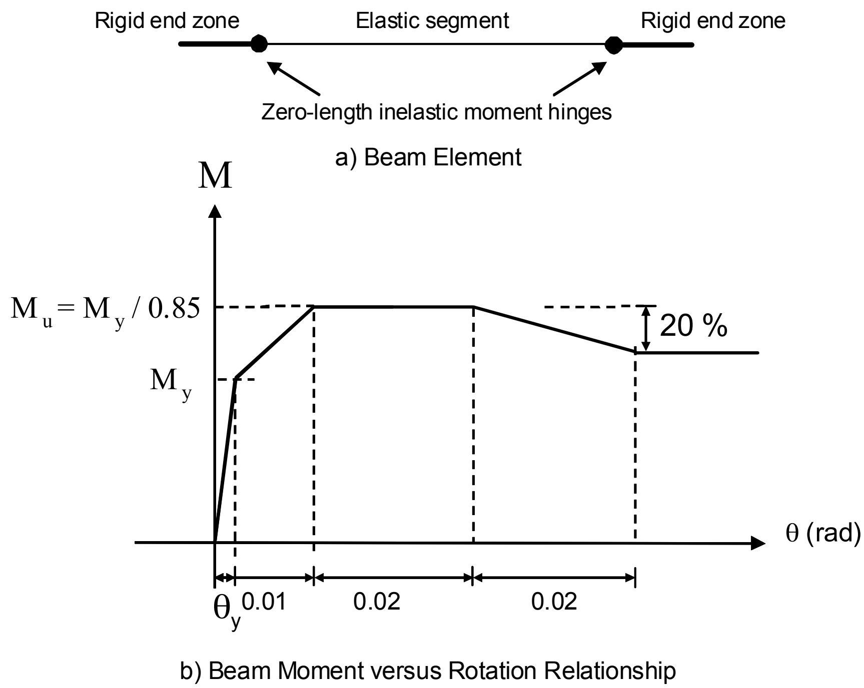


Figure 1. Beam Model

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 Figure 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. For beams, the energy dissipation coefficient was set equal to 0.4 up to the point where the beam reaches its ultimate moment strength, and 0.3 after the beam started to loose its strength.

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 Figure 2. For connections, the energy dissipation coefficient was set equal to 0.3 for all deformation levels.

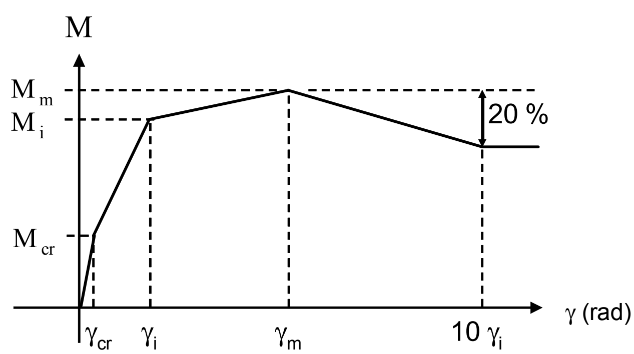


Figure 2. Joint Model

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 Figure 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 Figure 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. For the columns, an energy dissipation coefficient of 0.5 was taken at the yield point. This value was reduced to 0.3 at the ultimate point to account for stiffness deterioration and remained constant for larger rotations.

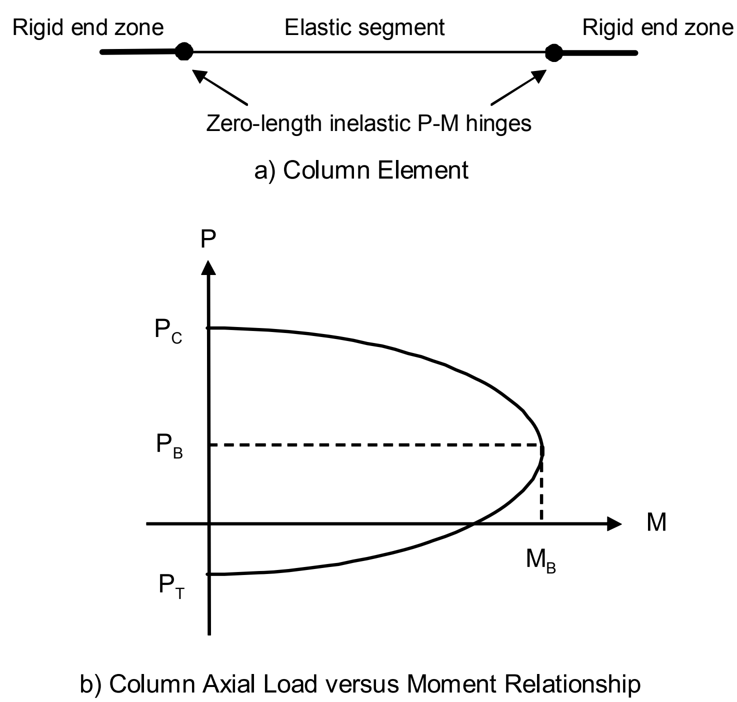


Figure 3. Column Model

Verification of the Model

When the analytical and the experimental results were compared, it was observed that the analytical results obtained using the member models defined here led to a good overall representation of the subassembly behavior. In this paper, the validity of the model is investigated by comparing the analytical results with the experimental data of selected interior and exterior beam-to-column subassemblies tested by Shiohara and Kusuhara (2006). Dimensions and detailing of Specimens A1 and A2 are given in Figure 4. Although these specimens are identical, due to two different loading schemes shown in Figure 5, Specimen A1 behaves as an interior beam-to-column connection, while Specimen A2 behaves as an exterior one.

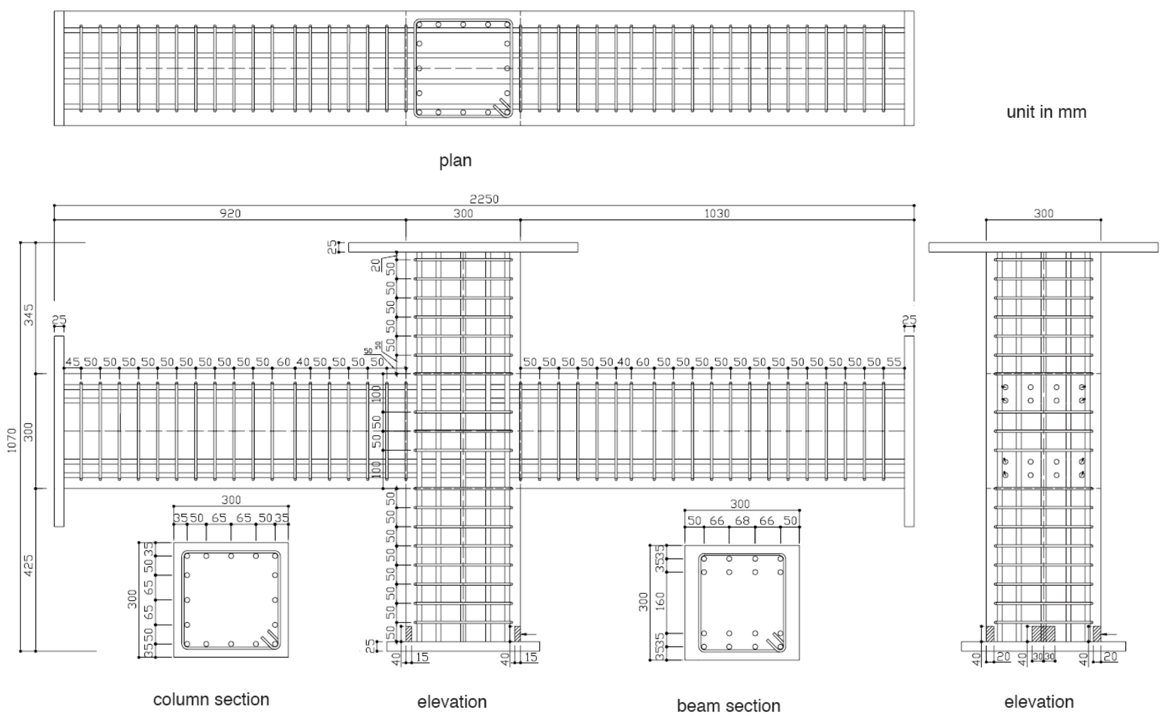


Figure 4. Dimensions and Detailing of Specimens A1 and A2

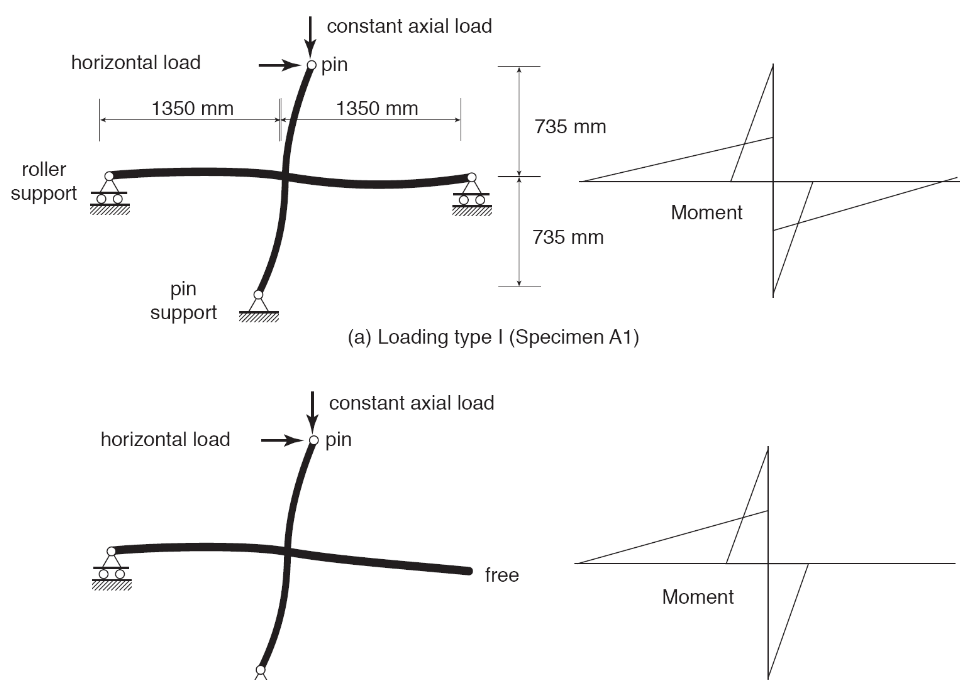
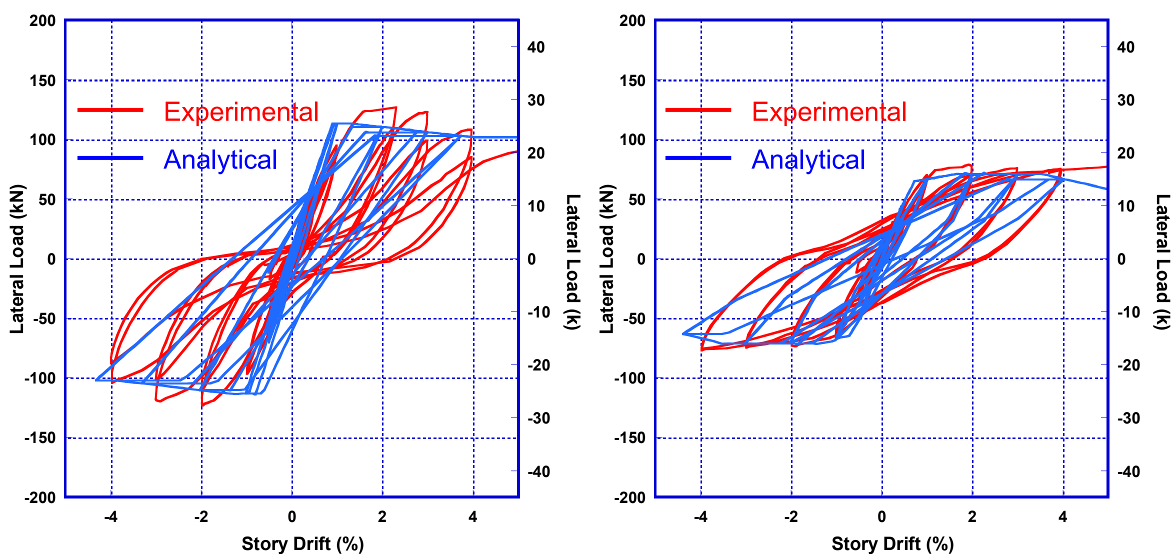


Figure 5. Loading and Boundary Conditions for Specimens A1 and A2

In Loading type I, the ends of the bottom column and both of the beams are pin supported and the top column moves in the horizontal direction under constant axial load. In Loading type II, the ends of the bottom column and one of the beams are pin supported and the end of the other beam is free. There are no internal stresses in the free beam, therefore, Specimen A2 behaves as an exterior beam-to-column subassembly. The top column of this specimen also moves in the horizontal direction under constant axial load as in Loading type I.

Figures 6 (a) and (b) show the comparison of the lateral load vs. story drift responses obtained from analytical and experimental results for interior and exterior beam-to-column connections, respectively. As can be observed from these figures, maximum load and drift values for each cycle were very close to the experimental results, which will help in the estimation of deformation demands, as well as strength demands. For the interior Specimen A1, the maximum observed experimental story shear was 126.6 kN, while the analytically obtained value was 113.4 kN, which gives a maximum prediction error of 10.4 %. For the exterior Specimen A2, the maximum experimental and analytical story shears were 77.9 kN and 71.8 kN, respectively, with an error of 7.8 %. For both specimens, the analytical results are conservative in predicting the maximum capacity and the overall behavior including strength and stiffness degradation is obtained with reasonable accuracy.



(a) (b)

Figure 6. Lateral Load vs. Story Drift Responses for Specimens A1 and A2

Some features could not be reproduced by the analytical model due to limitations in the software. Pinching in the hysteresis loops can not be defined, so the analytical results have wider loops for load vs. story drift relationships. However, the envelope curves that could be obtained from the analytical modeling successfully represent the overall behavior within acceptable error limits. Furthermore, Perform-3D response was stiffer than the experimental response for the early cycles due to the higher initial flexural stiffness values obtained for the beams. After cracking, the stiffness values in the analytical model showed better agreement with the experimental results.

When the analytical model was applied to an exterior beam-to-column subassembly from the same test series, Specimen B2, which does not have a free beam, larger strength degradation observed in this specimen after the beam yielding due to insufficient anchorage capacity of compressive longitudinal bar at the column face was successfully captured by the model. The detailing, loading scheme and lateral load vs. story drift response for this specimen are given in Figures 7 to 9, respectively. For this exterior Specimen B2, the maximum observed experimental story shear was 92.2 kN, while the analytically obtained value was 83.7 kN, which gives a maximum prediction error of 9.2 %.

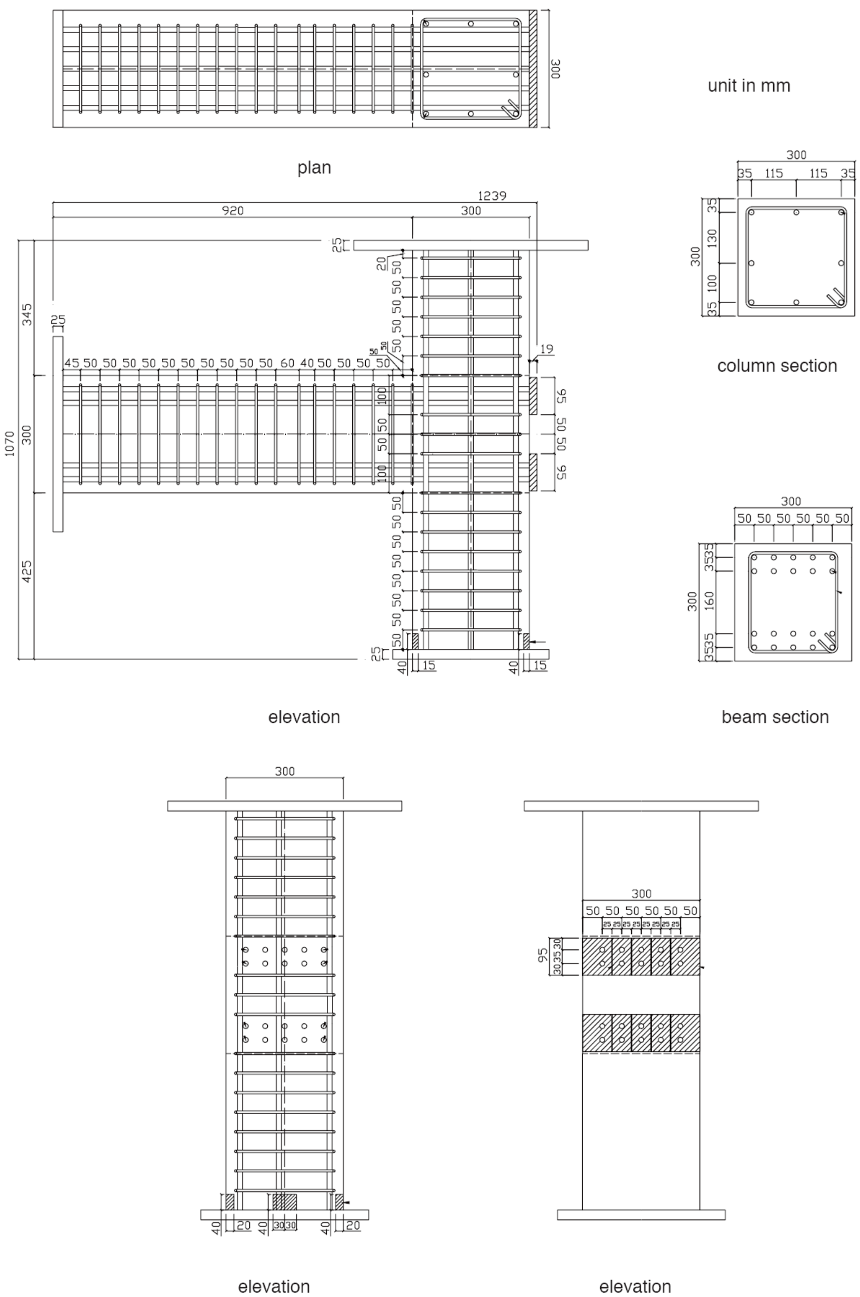


Figure 7. Dimensions and Detailing of Specimen B2

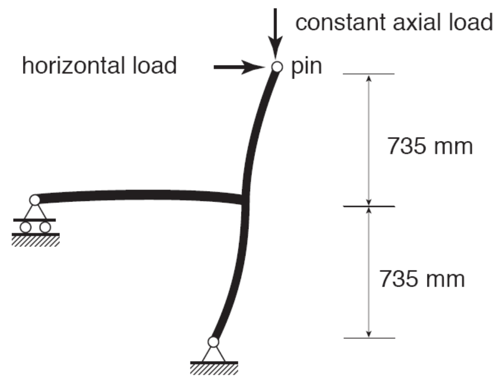


Figure 8. Loading and Boundary Conditions for Specimen B2

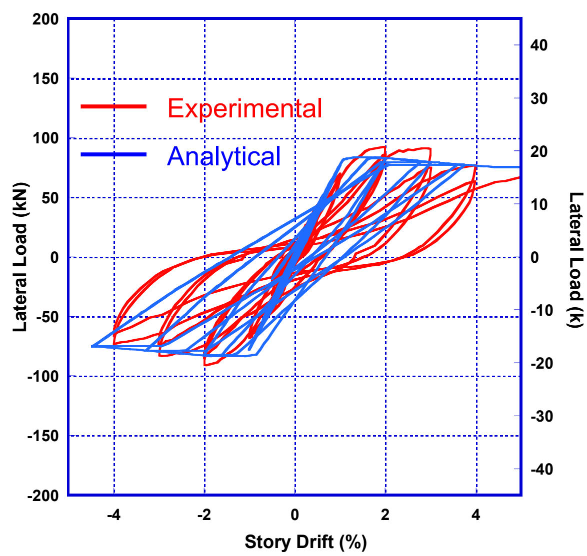


Figure 9. Lateral Load vs. Story Drift Response for Specimen B2

Conclusions

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 strength and drift demands. Although finite element modeling could be used to predict the behavior of these components, this would not be practical for the implementation of the push-over or dynamic analysis procedures to frame structures. Therefore, a simple joint model was developed in this analytical study that could be incorporated in a commercial software package available to practicing engineers.

From the comparison of the analytical results with experimental data, it was observed that the utilized analytical model successfully represents the overall seismic behavior within acceptable error limits. Furthermore, with this model, potential joint damage can be easily assessed and the effect of that damage on selecting the performance level for the frame can be determined.

References

ACI Committee 318, 2008. Building Code Requirements for Structural Concrete (ACI-318-08), 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*

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

FEMA 356, 2000, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Fecereal Emergency Management Agency, Washington, D.C., 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*, 58-62

Shiohara, H. and Kusuhara, F., 2006. Benchmark Test for Validation of Mathematical Models for Non-linear and Cyclic Behavior of R/C Beam-column Joints, *Report to Japan Society for the Promotion of Science*