

## MODELING OF GRAVITY-LOAD-DESIGNED RC FRAME BUILDINGS FOR NONLINEAR DYNAMIC ANALYSIS

M. Suthasit<sup>1</sup> and P. Warnitchai<sup>2</sup>

<sup>1</sup> Doctoral Student, Dept. of Structural Engineering, Asian Institute of Technology, Thailand <sup>2</sup> Associate Professor, Dept. of Structural Engineering, Asian Institute of Technology, Thailand Email: msuthasit@hotmail.com, pennung@ait.ac.th

## **ABSTRACT:**

To evaluate seismic performance of Gravity-Load-Designed (GLD) RC buildings, accurate but yet efficient nonlinear models for GLD RC members are essential. The main objective of this study is to develop the nonlinear models for GLD RC members which are applicable to nonlinear dynamic analysis. Based on macro-modeling approach, nonlinear models for beams, columns, and beam-column joints were developed. These models are able to simulate hysteretic force-deformation relations which are governed by modes of failure experienced by RC members. In particular, the models can simulate lap-splice, shear, flexure, joint-shear, and bond-deterioration failures. To validate the models, experimental force-deformation relations of column and beam-column joint specimens were compared to those obtained from the models. A very good agreement was found. Finally, to illustrate the applicability of the proposed models, a mid-rise GLD RC frame building was analyzed. It was found that the seismic behavior of the building was controlled by interactions among different failure mechanisms experienced by critical members. The proposed nonlinear models can be used in future researches to conduct detailed seismic performance evaluation of GLD RC frame buildings.

#### **KEYWORDS:**

gravity load designed, nonlinear dynamic analysis, model, seismic performance evaluation

## **1. INTRODUCTION**

It is well recognized that gravity-load-designed (GLD) RC frame buildings are vulnerable to seismic hazard. These buildings contribute a major portion in existing building stocks around the world. Therefore, their seismic behavior has been a major research topic for decades. Non-ductile reinforcement detailing and configuration irregularities are primary factors responsible for poor seismic performance. Combinations of these factors result in a wide variety of seismic performance as evidenced in past earthquakes. To address the problem, many experiments have been conducted to investigate the seismic performance of isolated members and their sub-assemblages. Findings from these studies are very useful for understanding the behavior of GLD RC members is well as for developing their nonlinear models. Nowadays the seismic behavior of overall framing system as a result of interaction among members is not well understood. Both experimental and analytical studies on this topic are rather scarce. Regarding the experimental approach, the high cost of specimen construction usually prohibits the study. On the other hand, the analytical approach requires a lot of computational effort and, thus, simplified models are usually adopted. Assumptions used in such models may result in an oversimplification of analysis results which hinder the actual seismic behavior of GLD RC frame buildings.

To conduct an in-depth seismic evaluation on GLD RC buildings, nonlinear dynamic analysis is essential. The main objective of this study is to develop accurate and efficient nonlinear models applicable to such analysis. Specifically, nonlinear models for beams, columns, and beam-joint joints are developed and implemented in the computational platform named RUAUMOKO, which was developed by Carr (2005). Note that the models can also be applied in other software platforms with minor modifications due to the simplicity of adopted modeling tools. The models are validated through comparison between analytical and experimental force-deformation



relations. Finally, to illustrate the applicability of the models, nonlinear dynamic analysis of an example 6-story building is conducted.

#### 2. BEAM AND COLUMN MODELS

Because of similarity between the seismic behavior of beams and columns, only the column model development is discussed here. Nevertheless, the concept can also be applied to beams. Free-body, shear, and moment diagram of RC columns subjected to seismic load are shown in Fig 1. Lateral deformation of a column caused by these seismic demands consists primarily of 4 modes: flexural deformation, anchorage slip, lap-splice slip, and shear deformation (Fig. 2). Mode of failure and hysteretic relationship between lateral force and deformation of RC columns are determined by the contributions from these deformation modes. Additionally, the contribution from each deformation mode to the overall response depends on various factors such as member dimensions, axial force, shear span, and etc. Because some of these factors are varied during seismic excitation, all deformation modes should be explicitly taken into account in the model.





Figure 2 Deformation modes of RC column: (a) flexure, (b) anchorage slip, (c) lap-splice slip, and (d) shear

In case of GLD RC columns, flexural moment capacity is normally uniform along the clear story height because equal amount of reinforcement is usually provided. Due to the uniformity of flexural moment capacity and the concentration of flexural moment demand at the column ends (Fig. 1c), the inelastic flexural deformation is confined within the column-end zones while leaving the middle portion responds in the elastic manner. This behavior can be idealized using a zero-length fiber element connected in series to each end of an elastic frame element (Fig. 3). Note that the 2D column model shown in Fig. 3 is drawn in 3D to facilitate the illustration only. Regarding the elastic frame element, the effective axial- and flexural-rigidity ( $EA_c$  and  $EI_c$ ) recommended by ASCE (2000) are used to define its elastic properties to account for inherent nonlinearities in RC members.

Based on macro modeling approach, there are mainly 2 alternatives for simulating response of plastic hinges: (1) using nonlinear rotational springs and (2) using fiber elements. The former requires less computational time. However, significant loss of accuracy has to be sacrificed because of its inability to simulate the axial-flexure interaction. In particular, the hysteretic moment-rotation response (strength and stiffness) of plastic hinges depends on the magnitude of the applied axial load which is varied during seismic excitation. As a consequence, the fiber elements approach is adopted in this study. The RC section of the plastic hinge being modeled is discretized into small fibers. Based on material characteristics, these fibers can be classified into 3 groups cover-concrete, core-concrete, and steel. The uniaxial material response of the individual fiber is represented by a nonlinear zero-length translational spring (cover-concrete, core-concrete, and steel springs shown in Fig. 3). To impose the plain-remains-plane assumption, these springs are connected to nodes A and B via rigid links shown by the thicken lines in Fig. 3. Note that, in Fig. 3, the steel springs are intentionally disconnected from node A to account for the effects of the anchorage slip (Fig. 2b). The anchorage slip is caused by bond-deterioration of reinforcement extended into an adjacent member. This results in an additional rotation to the plastic hinge. Additionally, when the strength of the anchorage zone is inadequate to fully mobilize the



reinforcement strength, the flexural moment capacity of the plastic hinge is reduced. Anchorage-slip springs shown in Fig. 3 are used to simulate the strength and the stiffness of the anchorage zones. By connecting these springs directly to the steel springs, the interaction between the anchorage zone and the plastic hinge is accounted for in the column model.

The lap-splice slip illustrated in Fig. 2c can significantly induce an additional deformation to RC columns. Subjected to a certain level of tension, tensile splitting-cracks are developed along the splice length. This causes both slippage and strength degradation of the spliced reinforcement. Because the lap-splice in GLD RC columns is usually located immediately above the floor level, the slippage along the splice length can be assumed to be located in the plastic hinge zone. This assumption allows the effects of lap-splice slip to be simulated by replacing the steel springs in Fig. 3 with lap-splice springs. To define the lap-splice springs, the lap-splice is idealized as shown in Fig. 4. The individual reinforcement in the lap-splice is assumed to behave as an anchored reinforcement. As a result, the lap-splice spring is identical to 2 anchorage-slip springs connected in series.

To simulate the effects of shear on the seismic performance of RC columns, it can be noticed from Fig. 1b that the seismic shear demand is uniform throughout the clear story height. This permits a nonlinear shear spring in Fig. 3 connected serially to the elastic frame element to simulate the overall shear behavior of the column.



Figure 3 Column model

Figure 4 Idealization of lap-splices

The mechanical model discussed previously is used to simulate the interaction among all deformation modes. In order to simulate the nonlinear response characteristics of these deformations, appropriate backbone curves and hysteretic rules have to be applied to each spring in the model. The adopted backbone curve for cover- and core-concrete springs is shown in Fig. 5a. The increase in compressive strength and ductility due to the confinement effect is calculated according to Mander et al. (1988) and Kent and Park (1971), respectively. The residual strength of core-concrete springs is assumed to be retained while that of cover-concrete springs is assumed to reach zero strength after spalling. The adopted hysteretic rule of concrete in the form of stress-strain relation is also shown in Fig. 5a.

For steel springs, a bilinear backbone curve which describes the elastic stiffness, post peak stiffness, and yield strength of reinforcement is used (Fig. 5b). A simplified hysteretic rule taking into account the Bauschinger, tension stiffening, and discrete cracks effects is shown in Fig. 5b. The backbone curve of anchorage-slip springs (Fig. 5c) is determined by assuming piece-wise constant bond stress distribution (Alsiwat and Saatcioglu, 1992). The Modified-Takeda hysteretic rule is used to simulate the response the anchorage-slip springs. The lap-splice spring is modeled as two anchorage-slip springs connected in series as shown in Fig. 4.

To validate the proposed column model, two cantilever column specimens subjected to quasi-static cyclic loading (Worakanchana, 2002) are considered. These specimens represent typical GLD RC columns in Thailand. Comparison between lateral force-deformation relations obtained from the experiment and the analytical model are shown in Figs. 6 and 7. The force-deformation relations as well as the modes of failure predicted by the proposed model match well with those found in the experiments.





(c) backbone curve and hysteretic rule for anchorage-slip and lap-splice springs

(d) backbone curve and hysteretic rule for shear-springs

Figure 5 Backbone curves and hysteretic rules adopted for the column model



Figure 6 Lateral force-displacement relations obtained from the experiment and the analysis - specimen R1





Figure 7 Lateral force-displacement relations obtained from the experiment and the analysis – specimen R2

## 4. BEAM-COLUMN JOINT MODEL

Beam-column joints (BC-joints) are one of the most critical components in RC frame structures. The lateral deformation of the overall framing system is significantly contributed by deformation in BC-joints. Fig. 8a shows a free-body diagram of an interior BC-joint subjected to seismic load. It can be seen that the force acting at the ends of reinforcement in the BC-joint are in the same direction. This usually causes bond-slip (Fig. 8b). It can also be noticed in Fig. 8a that shear stress is induced by simultaneous action of forces transferred to the BC-joint. This shear stress results in joint-shear deformation (Fig. 8c) and, in some cases, crushing of the concrete inside the BC-joint. In summary, there are 2 deformation modes associated with BC-joints: (1) bond-slip and (2) joint-shear deformation. The mechanical model shown in Fig. 9a is proposed.



Figure 8 A beam-column joint subjected to seismic load

Unlike the anchorage of column reinforcement in a foundation, the bond stress distribution along a reinforcement passing through a BC-joint can not be determined in advance. As a result, the reinforcement is discretized into small segments. Each segment is represented by steel springs (see Fig. 9a). At a connection between two consecutive steel springs, a bond-slip spring is added to simulate the force transferring between the steel and the surrounding concrete. Note that severe bond deterioration is very common in GLD BC-joints. In some case, tensile force acting on one side of the reinforcement is totally transferred to the other side. This causes two major consequences: (1) a reduction in join-shear stress and (2) an additional deformation in the plastic hinge of the adjacent beams and columns. The proposed mechanical model can simulate these effects.

Refer to Fig. 8a, joint-shear,  $V_j$ , can be determined by writing equilibrium equation of horizontal force acting on the BC-joint. This results in Eqn. 3.1 where  $C'_c$ ,  $C'_s$ , and  $T_s$  denote resultant force of concrete compression, steel compression, and steel tension, respectively. Assuming lever-arm depth, jd, is known (see Fig. 8a), Eqn. 3.1 can



be rewritten as shown in Eqn. 3.2 where  $M_C$  denotes moment acting on the BC-joint due to concrete compression and  $M_j$  denotes joint-shear force in term of moment. By connecting a nonlinear rotational spring to other elements in fig. 8a, the moment acting on the nonlinear rotational spring is identical to  $M_j$  in Eqn. 3.2.

$$V_{j} = C_{c}' + C_{s}' + T_{s} - V_{c}$$
(3.1)

$$M_{i} = jd \cdot V_{i} = M_{c} + jd \cdot C_{s} + jd \cdot T_{s} - jd \cdot V_{c}$$

$$(3.2)$$

The mechanical model discussed previously can be used to simulate the interaction between of the bond slip and the joint-shear deformation. In order to simulate the nonlinear response of the individual deformation mode, appropriate backbone curves and hysteretic rules have to be applied to each spring in the model. To simulate the discretized reinforcement, the backbone curve and the hysteretic rule shown in Fig. 5b are applied to the steel springs. To simulate the nonlinear bond-slip response, a backbone curve proposed by Eligehausen et al. (1983) and the Modified Takeda-hysteretic model is applied to the bond-slip springs (see Fig. 9b). For the nonlinear rotational spring, a backbone curve is determined using the Modified Compression Field Theory (Vecchio and Collins, 1988). A hysteretic rule proposed by Saiidi and Sozen (1979) is adopted. Fig. 8c shows the backbone curve and the hysteretic rule applied to the nonlinear rotational spring.

To validate the proposed BC-joint model, two beam-column joint specimens tested by Cheejaroen (2004) are considered. These specimens represent typical GLD RC beam-column subassemblages in Thailand. Quasi-static cyclic loading were applied to these specimens. Comparison between lateral force-deformation relations obtained from the experiment and the analytical model are shown in Fig. 10. Note that only one comparison is shown here. The force-deformation relations as well as the modes of failure predicted by the proposed model match well with those found in the experiments.



Figure 9 BC-Joint model: (a) mechanical model, (b) backbone curve and hysteretic rule for the bond-slip springs, and (c) backbone curve and hysteretic rule for the nonlinear rotational spring.





Figure 10 Lateral force-displacement relations obtained from the experiment and the analysis – specimen M2

## 5. AN ANALYSIS EXAMPLE

To illustrate the applicability of the proposed models, a nonlinear model of a 6-story 3-bay GLD RC building is constructed and analyzed using nonlinear dynamic analysis. Typical span length and story height are 7.0 m. and 3.7 m., respectively. Cross sectional dimensions of beams and columns are 0.30 m. by 0.65 m. and 0.60 m. by 0.45m., respectively. Reinforcement detailing of all structural members in the building are of non-seismic type. Therefore, various brittle modes of failure are expected during ground motion. Strong motion obtained during December 1979, Imperial Valley Earthquake is selected for analysis. PGA of the original record is scaled to 0.30g. Subjecting the model to ground motion, the roof drift time history is plotted in Fig. 11. At point A, the building was sustained no damage. However, during cycle back to the positive direction, plastic hinges were developed in some of the 2<sup>nd</sup> floor beams. At point C, where roof drift was peak at 1.6%, the failures including reinforcement yielding, lap-splice failure, and joint-shear failure were found. These damages were concentrated in the lower stories. This analysis example illustrated that the proposed models are practically applicable to nonlinear dynamic analysis and can be used to conduct seismic performance evaluation in a very detailed manner.



Figure 11 Nonlinear dynamic analysis results: solid dots, white dots, and rectangles indicates respectively, reinforcement yielding, lap-splice failure, and joint-shear failure.



## 6. CONCLUSION

The nonlinear models for GLD RC members including beam, column and BC-joint model were proposed. By comparing the hysteretic force-displacement relations obtained from the analysis to those from the experiments, it was found that the models give acceptable accuracy in predicting the strength, stiffness, and failure mode of GLD RC members. However, only a few numbers of experiments were used in the validation. To make a further improvement, it is suggested that an additional experiments should be used to validate the models. Applicability of the models to nonlinear dynamic analysis was confirmed by conducting the analysis example. The proposed models can be used in future researches to investigate seismic performance of GLD-RC buildings.

## REFERENCES

ACI (2005), Building Code Requirements for Structural Concrete and Commentary (ACI 318M-05), American Concrete Institute.

Alsiwat, J. M. and Saatcioglu, M. (1992), Reinforcement Anchorage Slip under Monotonic Loading, *Journal of Structural Engineering*, **118:9**, 2421-2438.

ASCE (2000), Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA-356), American Society of Civil Engineers.

Carr, A. J. (2005), RUAUMOKO Manual, The University of Canterbury, Department of Civil Engineering, Christ Church, New Zealand.

Cheejaroen, C. (2004), Effects of Bond Deterioration on Seismic Behavior of Interior Beam-Column Joints, M. Eng. Thesis, Asian Institute of Technology, Thailand.

Eligehausen, R., Popov, E. P., and Bertero, V. V. (1983), Local Bond Stress Slip Relationships of Deformed Bars under Generalized Excitations, EERC Report 83/23, Earthquake Engineering Research Center, University of California, Berkeley.

Elwood, J.K. and Moehle, J.P. (2003), Shake-Table Tests and Analytical Studies on the Gravity Load Collapse of Reinforced Concrete Frames, PEER Report 2003/01, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley.

Kent, D.C. and Park, R. (1971), Flexural Members with Confined Concrete, *Journal of Structural Division*, **97:ST7**, 1969-1990.

Mander, J.B., Priestley, M.J.N., and Park, R. (1988), Theoretical Stress-Strain Model for Confined Concrete, *Journal of Structural Engineering*, **114:8**, 1804-1826.

Paulay, T. and Priestley, M.J.N. (1992), Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley and Sons, Inc.

Saiidi, A. and Sozen, M. (1979), Simple and Complex Models for Nonlinear Seismic Response of Reinforced Concrete Structures, Report UILU-ENG-79-2031, Department of Civil Engineering, University of Illinois, Urbana, Illinois.

Vecchio, F. J. and Collins, M. P. (1986), The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear, *Journal of American Concrete Institute*, **83:2**, 219-231.

Worakanchana, K. (2002), Quasi-static cyclic loading test of reinforced concrete columns with lap splice in typical mid-rise buildings in Thailand, M. Eng Thesis ST-02-3, Asian Institute of Technology, Thailand.