Analysis on Nonlinear Seismic response of Multi-story Torsion-Irregular Structures

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ABSTRACT :

Previous earthquake disasters have shown that irregular structures tend to cause torsional damage. Although scholars all over the world have contributed much in torsion effect of structures, most code use results in linear analysis or nonlinear analysis of single storey to limit torsion effect. It is necessary to conduct nonlinear analysis on the torsion effect of multi-layer irregular structures. Moreover, centralized plasticity hinge model or segmented varying rigidity model were mostly applied in preceding studies, producing difficulty in dealing with special structure models, due to lack of test data. This problem is most significant for multidimensional hysteretic model of frame columns. The beam-column fiber element model with the method of flexibility promises a satisfactory method for multidimensional hysteretic relations. In this paper the fiber model is implemented to analyze the torsion effect of irregular structures. Three groups of 6-story irregular frame structure with different eccentricity ratios are analyzed. In line with Site Ⅱ, actual earthquake records are selected as the input while bidirectional earthquake excitation is taken into account. By checking the integral response (maximum drift and maximum base shear etc.) and local response (beam-column end plasticity hinge distribution and section angle ductility) of structures, the study try to evaluate the nonlinear seismic response of multi-story torsion irregular structures, and to provide guide opinion for the seismic design. The results show that the flexibility side are subject to more severe damage than the stiffness side, and the eccentricity ratio’s increase produces deteriorating torsion effect, which could be unobvious when using linear analysis model.

KEYWORDS : torsion, irregular structures, nonlinear analysis, fiber model, opensees

1. INTRODUCTION

Seismic damage in history have proven that irregular structure layout can easily lead to torsion damage (Chandler 1986, Wei Lian 2005), which are common seen in structural damage and pose great threat to human society. A persuasive example can be found in Guatemala earthquake, 1972. During the shake, the 15-level central bank tower collapsed due to asymmetric lateral force-resisting structural layout, while at the proximity the American Bank Building, an 18-level frame tube structure, survived with only minor damage. This is a typical case of damage from asymmetric lateral-force resisting structure layout. Actually, in 1985 Mexico earthquake about 50% structure are damaged due to structure torsion, directly or indirectly. The torsion problem in structure are even more prominent today due to diversified modern architecture styles. Although in many research paper and national codes the emphasis on the torsion problem has been demonstrated, and study on linear behavior or single level nonlinear structure proved fruitful, the nonlinear torsion behavior of irregular structure remains open for further examination. In this paper, three groups of 6-story irregular frame structure with different eccentricity ratios are analyzed. According to the analysis results, the study try to evaluate the nonlinear seismic response of multi-story torsion irregular structures, and to provide guide opinion for the seismic design.

2. NONLINEAR EARTHQUAKE RESPONSE ANALYSIS: METHOD AND PARAMETER

Studies (C.A.Zeris 1986, Fabio F. Taucer 1991, Chen Tao 2003) showed that fiber model provides acceptable simulation for the coupling effects of two-way bending moment and dynamical axial force in a frame column;

the flexibility-based FEM poses good simulation to highly nonlinear problem like element in soften stage, for the method uses the cross-section force to avoid construction of element nonlinearity. The cross-section force assumption is met in most beam column elements subject to axial force and bending moment. Combining two methods above with modern computational capability, nonlinear dynamical analysis using fiber FEA model built with flexibility approach becomes feasible. In this paper such model is built and analyzed on OpenSees (Open System for Earthquake Engineering Simulation) platform (S.Mazzoni 2006).

In the analysis the Scott-Kent-Parker single axis confined RC model (S.Mazzoni 2006) and Menegotto-Pinto model are applied. The RC cover layer and floor slab are applied with non-confined model, and the core is built with confined RC model. On nonlinear stage the material strength are set to average level. The fiber section and nonlinear beam column element by flexibility approach are used. On each element there are five integral nodes. The force and stiffness at control points are acquired by element level iteration. The element force and stiffness matrix are integrated along element axis using Gauss-Lobatto method.

The slab effect is considered by including the slab with reinforcement in the area of six slab thickness near a beam.

3. CASE STUDY DESIGN

In the paper three 3 by 2 six level frames are analyzed, all put in Chinese seismic code-GB 50011-2001's 8 degree 0.3g zone, with seismic engineering design performed with 0.35s characteristic period and site type II. The span length in two direction are 6m(3 spans) and 5m(2 spans) each. The storey height is 3.3m, except the ground storey with 3.9m. Table 1 gives beam/column sections. The stiffness eccentricity is induced by larger column section at the left. The dead and live load are 4.5kN/m2 and 2.0kN/m2 on floor, and 6.0kN/m2 and 2.0kN/m2 on roof. HRB335 bar are used for beam/column rebar, and HRB235 are used for stirrup and slab reinforcement. Strength level C30 is used for all RC members, with slab thickness of 100mm. When subject to frequent earthquake, the structures are designed to give results of ground storey column's axial load ratio and storey drift angle closed to the upper limit value. No rebar increase is applied in beam and column, except traditional structure consideration, and column rebar are integrated to meet the upper requirement. It is common-

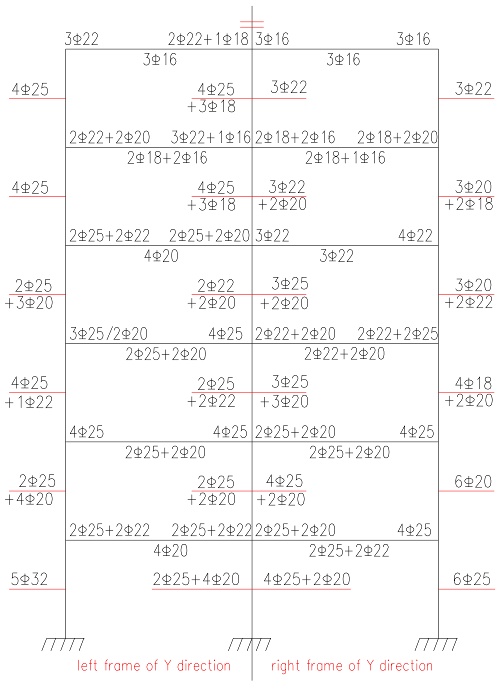


Fig. 1 Frame sectional reinforcement of model 3

Table 3.1 Cross-section of models

Model ID	Eccentric ratio e/r	member	Cross section mm×mm
1	0.13	beam	300×700
		left column(1st axis)	600×600/500×500
		right column(3rd axis)	550×550/450×450
2	0.44	beam	300×700
		left column(2nd axis)	650×650/550×550
		right column(2nd axis)	500×500/450×450
3	0.81	beam	300×700
		left column(2nd axis)	500×1400/500×1200
		right column(2nd axis)	500×500/450×450

Section of storey 4~6 is placed after "/"; stiffness eccentricity e: distance between stiffness centre and mass centre; mass revolving radius r: square root of the ratio between mass polar inertia moment relative to the mass centre and storey mass.

-ly accepted that frame by this consideration can meet the code requirement but at a marginal state. It is noteworthy that the PKPM, mostly wide used structure analysis package in China, output the beam rebar with single reinforcement assumption, leading to a pro-strong beam and strong-beam-weak-column situation. In the model this problem is considered by modifying beam's bottom rebar using T-shape cross section, and upper rebar using double-reinforcement model. The moment adjustment coefficient is canceled for columns with axial load ratio less than 0.15. Moreover, according a study (in publishing), for regular frames in 8 degree 0.3g zone, the weak storey (storey with column hinges) is usually produced even with the moment adjustment coefficient 1.2 is applied, and satisfactory coefficient value should be more than 1.8, which is taken in this paper.
In nonlinear analysis stage the average value of C30 concrete strength is 26.1MPa, and modular is 3.236× 104MPa.

4. NONLINEAR DYNAMICAL ANALYSIS RESULTS

4.1. Wave Selection

4 pieces of earthquake wave record are selected for nonlinear time history analysis. Among them three are picked by dual band wave selection method (Yang Pu 2000) from database and the other is artificial wave generated using ARMA model to match the national code spectrum. According to national code the characteristic period T_g is increased by 0.05s, and the T_g=0.4s in the record used. The wave record can be seen in Table 2. In Figure 2 a comparison is made between record spectrum and Chinese national code GB50011-2001 spectrum, the two are close before t=1.7s.

Table 4.1 wave record used in the analysis

ID	Earthquake	Time	Record station	Dire.
SF	San Fernando	1971.02.09	Los Angeles, CA	South
ML	Mammoth Lakes Aftershock	1980.05.27	Mammoth Elementary School, CA	Nort
CL	Coyote Lake	1979.08.06	Gilroy Array	140
AF	Artificial wave matching site II			

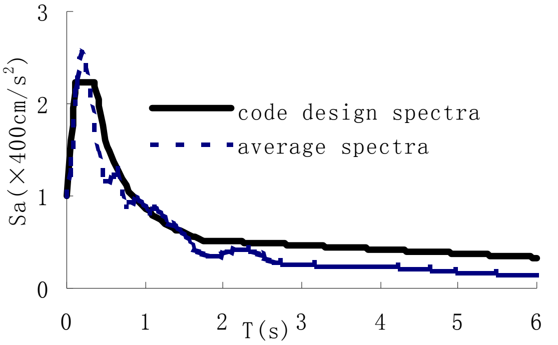


Figure 2 major quake spectrum against code spectrum

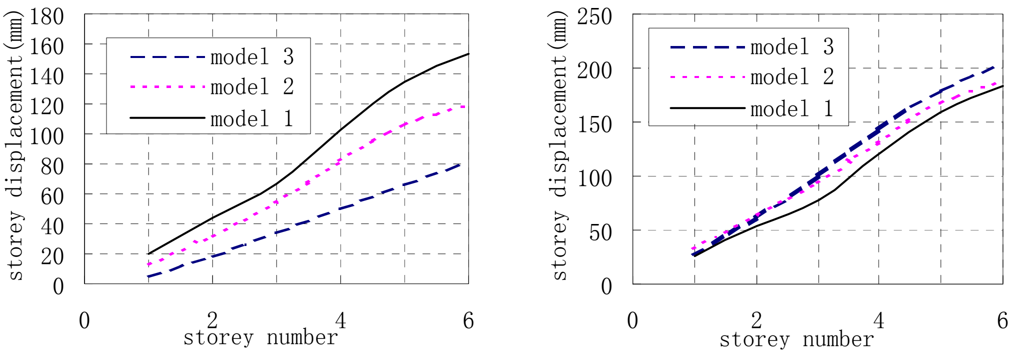
4.2. Displacement Response

The output of time history analysis using major earthquake multi-wave input includes storey displacement, storey drift angle, torsion angle, etc. The maximum value of Y-direction storey displacement, storey drift angle, torsion angle, scaled by storey number, can be seen in Figure 3~5. The results are separated by rigid/flexible side, each indicating the side frame with small/large distance from the stiffness centre. By Figure 3, the maximum value of rigid side storey displacement decreases when eccentric ratio increases, while at the flexible side the displacement increases, with a lower step. By Figure 4, the storey drift angle at rigid side changes in a same pattern with storey displacement, increasing with eccentric ratio, and the increasing step is large, compared with flexible side's trend, in which model 1 and model 2's results are slightly larger than model 3. In the first models, some storey drift angles of the flexible side are close or even over the 1/50 limit for the major earthquake set by code, and maximum value of model 3 is 1/55, slightly lower than the limit. Generally, the flexible side is in a more disadvantageous state, and the changing rate of displacement response is larger in rigid side.
Figure 5 shows that the relative torsion angle exhibit significant increase when eccentric ratio increases, which is the evidence that the displacement difference between rigid side and flexible side goes larger as eccentric ratio increases. Figure 6 shows the storey torsional displacement ratio increases as well. The storey torsional

displacement ratio equals to maximum storey displacement divided by average displacement, so it shows extent of the storey torsion. Generally, the torsion effect increases as the eccentric ratio increases.

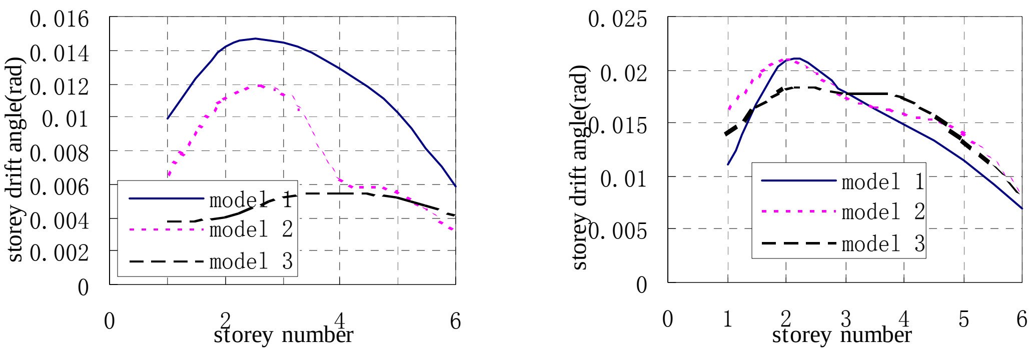
4.3. Storey Shear Force Response

Figure 7 provides the distribution of storey shear force in three models. Model 3 outputs largest force for it's the vertical members' stiffness at rigid side is significantly larger than that of model 1 and 2, and in turn model 3 has larger total stiffness.



（a） rigid side （b） flexible side

Figure 3 Model 1~3's maximum storey displacement in major earthquake, Y-direction



（a） rigid side （b） flexible side

Fig. 4 Model 1~3's maximum storey drift angle in major earthquake, Y-direction

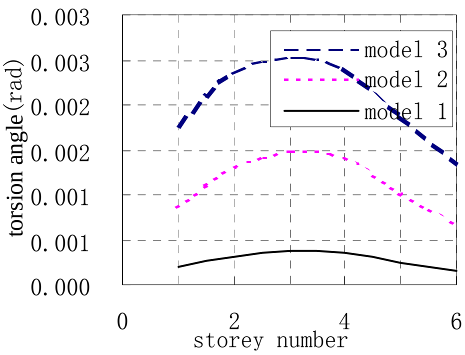


Fig. 5 maximum torsion angle

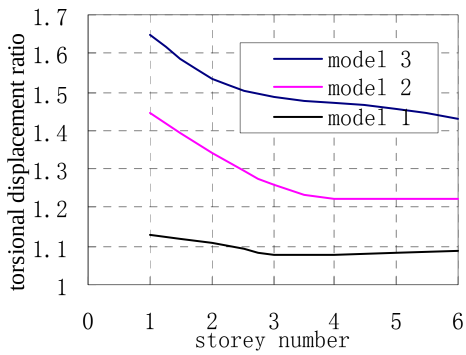


Fig. 6 maximum torsional displacement ratio

4.4. Pattern of Plastic Hinges

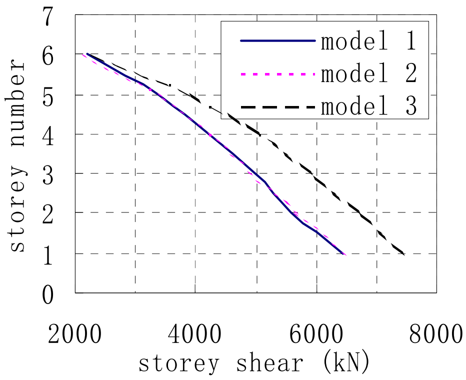


Figure 7 Storey shear force

In Figure 7~9 the plastic hinges pattern are given for three models. The hinges shown are produced with major earthquake artificial wave input, and when the maximum value of storey drift angle is reached. The circled node in the figures indicates the yielding member. Hinges produced in other wave input, although with some difference, generally exhibit same pattern. According to the results, the column hinges at rigid side decreases as eccentric ratio increases, especially in model 3, whose hinges are mostly on beam. However, different eccentric ratio does not produce significant pattern change for column hinges in flexible side. On this side, although some columns' both ends yielded, the structure stays away from large scale storey drifting for the confinement by other frames lower plastic movement. But the trend toward large scale storey drifting increases as the eccentric ratio increases. Generally, the flexible side produce more column hinges, some on both ends of a column, and this side is in a more disadvantageous state.

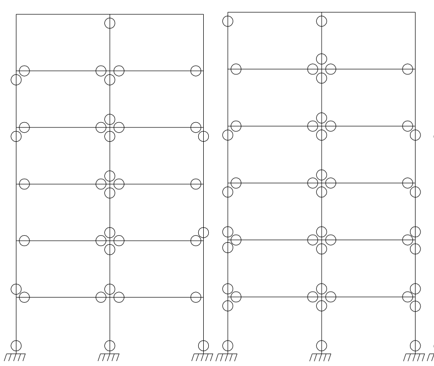

(a) rigid side (b) flexible side

Figure 8 model 1's plastic hinge pattern

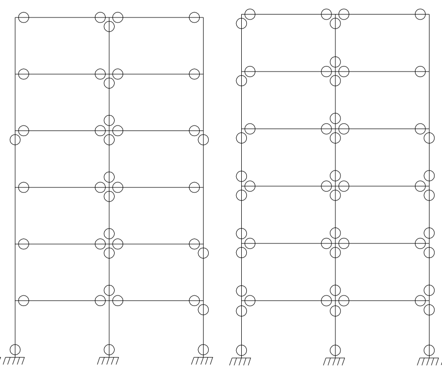

(a) rigid side (b) flexible side

Figure 9 model 2's plastic hinge pattern

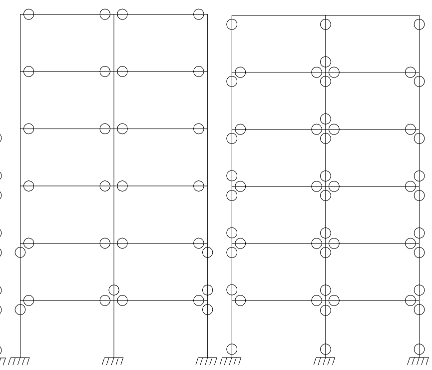

(a) rigid side (b) flexible side

Figure10 model 3's plastic hinge pattern

5. CONCLUTION

The multi-wave nonlinear dynamical time history analysis for the frames with different eccentric ratio, emphasizing the nonlinear torsion effect, shows following propositions:

1)As eccentric ratio increases, the storey displacement and drift angle decreases at rigid side, the storey displacement at flexible side increases, with a lower step than the rigid side. The storey drift angle exhibit no obvious change, mostly fluctuate around the code limit value.

2)As eccentric ratio increases, storey relative torsion angle and storey torsional displacement ratio increases, indicating increasing torsion effect.

3)As eccentric ratio increases, the difference of storey drift angle between two sides increases.

4)For frames designed by the code, the increase of eccentric ratio does not produce much change in planar distortion, but significant increase in torsion angle, and in turn the torsion moment in structural members, especially the vertical ones. This torsion moment, together with its coupling effect with bending and shearing strength, is not widely considered by engineers, while more attention on this kind of effect is justifiable.

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