# Effects of Modeling of RC Flat Slabs on Nonlinear Response of High Rise Building Systems



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#### SUMMARY:

In typical structural design practice, the performance of lateral-load-resisting systems is the main focus of (nonlinear) lateral analysis and flat plate systems are considered only as a source of mass and gravity loads. However, recent research on nonlinear dynamic analysis of high-rise core wall buildings with flat plate systems (e.g., Zekioglu et al., 2008; Yang et al., 2010) has shown that flat slabs can substantially affect the response of the buildings with flat plate systems. In this paper, nonlinear behavior of a 50 story RC tall building is discussed in terms of three lateral-load-resisting systems: (a) concrete core shear walls only, (b) core shear walls with flat slab elements; and (c) core shear walls with damped outrigger systems. The lateral drifts, story level accelerations and behavior of the flat plates are investigated.

Keywords: High Rise Buildings, RC Flat Slab, Nonlinear Response Analysis, Performance Based Design, Damped Outrigger

### **1. INTRODUCTION**

### 1.1. Objectives

There is currently resurgence in the construction of high rise buildings around the world, but there are major differences in the approach taken to seismic design in different countries. A performance-based philosophy has been explicitly required by regulatory authorities in Japan for buildings exceeding 60m since 1981. In China, height limits on tall buildings depend on the seismic zone, structural material and adopted structural system (GB50011-2001). In the United States, the building codes such as the California Building Code (CBC, 2010) and International Building Code (IBC, 2012) do not require a different methodology for the seismic design of high-rise buildings, but permit only a limited number of structural systems for buildings taller than 49m in height. This approach could lead to a structural design that is neither economical nor performing as intended when subjected to minor to severe earthquakes.

While the building codes permit performance-based design, little specific guidance has been provided. Recently published alternative design guidelines (CTBUH (2008), LATBSDC (2011), and PEER TBI (2011)) have provided much needed guidance for the seismic design of tall buildings. These guidelines facilitate the use of lateral-load-resisting systems that were not available for the design of tall buildings to the design community.

Reinforced concrete bearing walls with flat plates is one of the lateral-load-resisting systems that are permitted by these alternative design guidelines. In typical structural design practice, because the performance of lateral-load-resisting systems is the main focus of (nonlinear) lateral analysis, flat plate systems are considered only as a source of mass and gravity loads. As such, contribution of the flat plate system to the overall stiffness of the building is often neglected. However, recent research on

nonlinear dynamic analysis of high-rise core wall buildings with flat plate systems (e.g., Zekioglu et al., 2008; Yang et al., 2010) has shown that flat slabs can substantially affect the response of the building under seismic excitation.

Although the behavior of RC and PT flat plate systems under seismic excitation has been extensively studied analytically (e.g., Hwang and Moehle, 2000) and experimentally (e.g., Kang and Wallace, 2005; Klemencic et al., 2006), there have been relatively few studies on the nonlinear behavior of tall buildings with flat plate systems. This paper attempts to investigate, nonlinear behavior of a 50 story RC flat plate system in interaction with three lateral-load-resisting systems: (a) concrete core shear walls only, (b) concrete core shear walls with flat slab elements; and (c) concrete core shear walls with damped outrigger systems. The paper presents the results and discusses the influence of modeling RC Flat slab elements on the predicted performance of several benchmark tall building systems.

### **1.2. Description of Benchmark Buildings**

A 50 story tower with a floor plan of 29 m by 35 m was used as the reference building. The aspect ratio of the building in X and Y directions are 5.3 and 4.5, respectively. The gravity system of the building consists of 250 mm thick reinforced concrete slabs and square reinforced concrete columns. Three cases were used in the study: (a) RC bearing core walls only, (b) RC bearing core walls with flat slabs and; (c) RC core walls with damped outriggers at approximately mid-height and top of the building (Figure 1).



Figure 1. Mathematical models of the benchmark buildings; (a) concrete core shear walls only, (b) concrete core shear walls with flat slab elements, and (c) concrete core shear walls with damped outrigger system.

The same core wall thickness was used in all three cases. The aspect ratio of the core wall for all three analysis models is 17.0 and 10.6 in X and Y directions, respectively. Typical story height is 3.05 meter

with a 6.10 meter high lobby level. The lateral-load-carrying system comprises bearing core walls coupled with 0.40 to 0.76 meter deep spandrel beams. Concrete of grades of 55 MPa and 40 MPa were used for lower and upper half of the lateral-load-resisting system and the columns and slabs, respectively. Expected strength values were employed for the nonlinear response history analysis.

# 2. NONLINEAR MODELLING APPROACH

The nonlinear dynamic response history analysis of this building was carried out using Perform 3D (CSI, Version 5, 2011). The modelling assumptions that were used in this study are shown in Figure 2 and explained in the following sections.



Figure 2. Nonlinear modelling of components (Zekioglu et al., 2008)

## 2.1. Coupling Beams and Shear Walls

For the coupling beams and the gravity columns a lumped plasticity model was employed to represent the nonlinear behavior. This model permits plastic hinges to form and to develop plastic rotations at both ends of a column or beam. A plastic hinge is permitted to form at any node, with a yield surface having N-Mxx(major)-Myy(minor) interaction based on the fiber analysis of the reinforced sections at each level. Shear degradation in shear stiffness or in shear strength is not represented according to the capacity design and strong shear-weak bending strength hierarchy.

The reinforced concrete walls were modelled using elements defined by a number of fibers along the length of each wall. The fibers representing concrete and reinforcing steel are assigned for appropriate nonlinear hysteretic material models. By this modelling approach the non-linear interaction between axial effects (forces and strains) and bi-axial bending effects (moments-curvatures) is dealt with explicitly. Nonlinear fiber sections were used over the entire height of the building but the number of fibers in each section was reduced up the height of the building to decrease computation time.

#### 2.2. Flat Slabs

The floor slabs have substantial contribution to the overall lateral stiffness of the structure (Zekioglu et al., 2007; 2008). For cases (b) and (c), an equivalent beam model was used to represent the nonlinear behavior of the slabs. The model consists of an elastic equivalent beam and nonlinear plastic hinges at the slab-beam ends. Elastic effective width of the beam members was calculated using the equations provided by Hwang and Moehle (2000). The slabs which span from the columns to the core walls were represented by two elastic beams with each having half a span length. For these cases, the equivalent beam model was used to determine the width of the slab-beam framing into the column, whereas the entire span width was used for the slab-beams that frame into the core walls. Yield and ultimate moments for positive and negative bending of the slab-beams were determined from the slab flexural reinforcement that was expected to contribute, i.e., reinforcement within the column strip where the slab frames into the columns, and reinforcement within the entire width, where the slab frames into the core walls. ASCE/SEI 41 Supplement #1 (2007) was used to obtain the plastic rotation modelling parameters of the slabs.

### 2.3. Viscous Dampers

A total of thirty non-linear viscous dampers were used between the outrigger walls and the columns. Each damper had a maximum force capacity of 3MN. The non-linear characteristic that has been used to represent the damper is given as:

$$F = CV^{\alpha}$$
 (2.1)

where *F* is the restoring from the damping element, *C* is the damping constant, *V* is the end to end velocity across the element, and  $\alpha$  is the velocity exponent constant, which typically ranges from 0 to 1. The damper with  $\alpha$ =1 is called a linear viscous damper in which the damper force is proportional to the relative velocity. The damper with  $\alpha$  greater than 1, is not common for practical applications. The damper with  $\alpha$  smaller than 1 is called a nonlinear viscous damper which is effective in minimizing high velocity shocks. For a small relative force, the damper with a value less than 1 can give a damper force larger than those of the other two types of dampers.



Figure 3. Force-velocity relationships for viscous dampers

A 2% intrinsic damping has been assumed in the modelling of the structure. This has been represented using the "Damping-frequency-range" facility in Perform 3D. Note that the hysteretic damping associated with plastic hinging of coupling beams or shear walls and the energy dissipated in the viscous dampers are represented separately and explicitly.

The damped outrigger concept was developed by Arup to introduce high levels of dependable damping (energy dissipation) in high-rise buildings (Figure 4). Damping provided by such systems is an effective means of reducing dynamic response induced by wind or earthquake actions, and can usually lead to satisfactory performance at a cost lower than other methods (such as stiffening). The damped outrigger system uses outriggers to increase damping, which have traditionally been used to connect central core structures to the perimeter columns to enhance the stiffness and strength of high-rise buildings. This is achieved not by making the connections rigid between the outriggers and the perimeter columns, but by permitting relative vertical movement of partially restrained viscous dampers. Typically 5% to 10% of critical damping can be achieved.



Figure 4. Typical damped outrigger concept layout (Smith and Willford, 2007).

Compared to other supplementary the damping systems, damped outrigger system has the following advantages: (1) more damping can be achieved; (2) the damping effect does not depend on the frequency tuning which can be compromised by structural non-linearity during strong winds and earthquakes; (3) several modes can be damped out the same components; (4) the damping is provided by a significant number of individual components and thus a hypothetical failure of a proportion of these does not affect the overall damping generated.

### 2.4. Seismic Hazard

Seven time history pairs were used that spectrally match to the Maximum Considered Earthquake (MCE) hazard level (2475 yr) for a site in Los Angeles Downtown. The fault-parallel direction was assumed to be parallel to the X axis of the building model. Acceleration response spectrum plots derived from the records are shown in Figure 5. It can be observed that the spectral records are closely matched to periods smaller than 4 seconds but that there is greater dispersion for longer periods.



Figure 5. Acceleration Response Spectrum curves for Service Level and MCE scenario earthquake records.

### 3. NONLINEAR RESPONSE ANALYSIS RESULTS

Per PEER TBI Guidelines (2010), the preliminary design of the building was undertaken using Service Level seismic hazard (43 years return period, 2.5% damping). The seismic performance of the preliminary design was then evaluated using a non-linear analysis model under 7 pairs of MCE records. Figure 6 presents maximum and mean key demand parameters defining the global responses of the building subjected to the MCE level ground motion pairs: (i) core wall shear; (ii) overturning moment; and (iii) inter-story drift.



Figure 6. Comparisons of core shear, overturning moment and story drifts along the building height.

The seismic weight of the building was calculated to be around 500 MN. Nonlinear time history analysis yielded mean base shears of 69 MN (14%W) and 68 MN (14%W) in the X and Y directions, respectively for the core wall only building. Introducing flat slabs did not cause any difference in base shear. On the other hand, addition of the damped outriggers to the lateral-load-resisting system increased the base shear in the X direction to 90 MN (18%W). There was almost no change again in the Y direction (71 MN). There were no significant changes observed among the benchmark buildings in terms of core wall overturning moments. This lack of scatter of core wall bending moment profiles is partially due to redistribution of overturning moments between the core wall and the columns initiated by the outriggers.

The most significant effect of incorporating the flexural behavior of the flat slabs is observed in the maximum inter-story drift results (Figure 6), especially in the direction where coupling beam nonlinear behavior governs. The study showed that incorporating the flexural behaviour of the floor slabs into the nonlinear time history model leads to a reduction in story drift. For the core only model, the mean peak story drift predicted from the seven MCE nonlinear response history analyses was 2.69% (max. 3.66%) and 2.26% (max. 3.71%) for the X and Y directions, respectively. When the slab elements were incorporated into the building model, the drifts were halved to 2.07% (max. 3.84% and 3.38%) in both directions. The study also showed that introducing dampers into the nonlinear time history model led to a further-significant (and realistic) reduction in predicted story drift. For the outrigger system, the mean peak story drifts predicted from the seven MCE nonlinear response history analyses were 1.52% (max. 1.98%) and 1.88% (max. 2.50%) for the x and y direction, respectively.

Measured response of the non-linear damper elements was in agreement with the expected performance. Figure 7 presents the nonlinear hysteretic response curve obtained from one of these damper elements when subjected to Northridge event.



Figure 7. Nonlinear force-displacement response of a damper from Northridge event (F=3MN,  $\alpha$ =0.25).

The contribution of the components of the lateral-load-resisting system to the overall dissipated energy was investigated to determine the major contributors. As shown in Figure 8, for all three cases, the energy dissipation of the lateral system was heavily reliant on the coupling beams (27.1% to 37.4%). In all three cases, the energy dissipated by the flexural yielding of the core walls was limited to 2.5% to 9.9%. For the building with damped outriggers, only 2.5% was dissipated by the core walls meaning essentially elastic behaviour. Interestingly, when the floor slabs were considered in the analysis models, they were able to dissipate a major part of the total energy (32.5%).

From past experience, 5 to 10% energy dissipation was expected before the analysis was done. The nonlinear analysis results confirmed this expectation. The damper elements had dissipated 8.2% and 5.6% of the total energy in the X and Y directions, respectively.



Figure 8. Energy dissipation percentage of lateral system components at benchmark building systems.

A sensitivity study was undertaken to investigate the effects of damper capacity (*F*) and velocity exponent ( $\alpha$ ) on the energy dissipation. The analysis results had shown that the amount of energy dissipated by the dampers were sensitive to the capacity of the dampers. This implied that the strategy to optimize the number of dampers by using higher capacity dampers, would not give the intended performance if dampers with capacities higher than 3 MN are used as was the case. The common practice for wind applications is to use smaller velocity exponents to be effective at very small velocity magnitudes. Sensitivity analysis under the MCE level hazard was undertaken using velocity exponents of 0.10, 0.25, 0.50 and 1.00. The non-linear analysis model with velocity exponent of 0.10 failed to converge. The sensitivity study results had shown that velocity exponent optimal range for a seismic event was between 0.25 and 0.50 with minimal scatter.



Figure 9. Energy dissipated by different capacity dampers.



Figure 10. Fragility relations of the damage states for reinforced concrete slab-column connections without shear reinforcement,  $0.2 \le \text{GSR} \le 0.4$  (Gogus, 2010)

Contribution of the slab stiffness and damped outriggers to the behavior of the models were assessed based on the fragility relations derived by Gogus (2010) (Figure 10). Three fragility relations represent the stiffness degradation (DS1), yielding of the flexural reinforcement (DS2), and punching (DS3) of reinforced concrete slab-column connections which have gravity shear ratios ranging from 0.2 to 0.4. Average drift values obtained in this study were plotted on the same figure to allow comparison between the three Perform 3D models. Based on the fragility relations shown in Figure 10, all the models fall into a state where yielding of the flexural reinforcement has already occurred, but punching has not yet been experienced. Results indicate that addition of the slab stiffness to the model led to a 22% decrease in the lateral inter-story drift of the structure, whereas addition of the damped outriggers resulted in a further drift reduction of 20%, reducing the possible damage the structure could experience.

### 4. CONCLUSIONS

Recently published alternative seismic design guidelines permit high-rise core wall buildings with flat plate systems in high seismic areas as an effective lateral-load-resisting system. Past research on the dynamic response of this type of buildings (e.g., Zekioglu et al., 2008; Yang et al., 2010) has shown that flat slabs can substantially affect the seismic response. For this study three analysis models were compared: (a) core wall only; (b) core wall with flat slabs; and (c) core wall with flat slabs and damped outriggers. The comparison of these three case studies indicated that when explicitly modelled, flat slabs could have substantial contribution to the energy dissipation of the structural system. According to the analysis, results inter-story drifts decrease up to 22%. This reduction in drift demands do not translate to a substantial performance improvement in the fragility function for the flat slabs. Addition of the damper elements were able to dissipate 8.2% and 5.6% of the total energy in the X and Y directions, respectively. Sensitivity studies on damper properties indicated that the amount of energy dissipated by the dampers was sensitive to the capacity of the dampers and that the velocity exponent optimal range for a seismic event was between 0.25 and 0.50.

Although all three cases satisfied the requirements of PEER TBI guidelines, expected structural and non-structural damages in all three buildings are not the same. The reduction of the mean inter-story

drifts to a range of 1.5 to 2.0% using damped outriggers could substantially reduce the damage in the flat slabs and other structural and non-structural elements.

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