Identification of Reasons for High Inelastic Seismic demands in High Rise RC Core Wall Buildings

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Abstract

Recently, the issue of high inelastic seismic demands at severe ground shakings such as MCE level has been highlighted in the conventionally designed high rise reinforced concrete (RC) core wall buildings. Furthermore, it is also found that the MCE level demands obtained from more rigorous non-linear response history analysis (NLRHA) are much greater than the DBE level demands obtained from the response spectrum (RS) procedure. In this study, different factors responsible for the high inelastic seismic demands and large difference between the demands obtained from the RS method and NLRHA are identified. A case study building is analyzed by the RS and NLRHA procedures using DBE response spectrum and the seven MCE spectrum compatible ground motions respectively. The factors which could be apparently responsible for large difference between the demands by the RS and NLRHA method are identified as: overstrength factor, excitation intensities, and different damping ratios. The effect of these factors on seismic demands is quantitatively estimated by performing the NLRHAs at different combination of damping ratios and excitation levels. Apart from three obvious factors, some other factors, which cannot be explained by the NLRHA, are also found to be responsible for the high inelastic seismic demands and large difference between the DBE demands by the RS procedure and MCE demands by the NLRHA procedure. To identify those factors, modal decomposition of the inelastic responses of the case study building using uncoupled modal response history analysis (UMRHA) is in progress.

Keywords: High rise RC core wall buildings, inelastic seismic demands, inelastic modal decomposition, uncoupled modal response history analysis
1. INTRODUCTION

High rise reinforced concrete (RC) core wall buildings are gaining popularity because they offer advantages of lower costs, faster construction, and more open and flexible architecture compared to high rise buildings with other lateral-force-resisting systems (Maffei and Yuen 2007). They are also recently being constructed in high seismic areas (Mohle 2008). In this kind of structures, the lateral framing system consists of a central core wall, peripheral columns and, in some cases, outriggers connecting between the core wall and the columns. As the core wall is generally much stiffer than the peripheral columns, the lateral load in buildings, particularly those without outriggers, is mostly resisted by the core wall. For economic reasons these buildings are not designed to remain elastic under the DBE or the MCE levels. Flexural plastic hinge is normally allowed to form at the base of the core wall under such severe ground shakings, but the plastic rotation in the hinge zone must be within an acceptable limit and the wall above the hinge zone is expected to remain elastic.

Due to the long-period nature of the structures, a significant contribution to seismic responses from higher vibration modes is expected. Therefore, the response spectrum analysis (RSA) procedure, which accounts for multi-mode effects, is commonly used in the seismic design of high rise core wall buildings, while the traditional equivalent static design procedure, which assumes the first mode domination, is known to be inappropriate for such cases. In the RSA procedure, the elastic responses of each vibration mode are first determined from the DBE response spectrum at 5% damping ratio and then combined into the total elastic responses by either the SRSS or the CQC method (Chopra, 2001), and finally reduced to the seismic demands for structural design by a response modification factor “R” that accounts for the overstrength and inelastic effects of the structure (ICBO 1997).

Recently, however, new studies on a 60-story and a 40-story RC core wall building in high seismic areas show clearly that the RSA procedure greatly underestimates the seismic demands along the entire height of the core wall under both DBE and MCE (Klemencic 2008, Klemencic et.al. 2007, and Zekioglu et. al. 2007). In these studies, non-linear response history analysis (NLRHA) was carried out as part of the performance-based seismic design to verify the seismic demands estimated by the RSA procedure. The base shear demand and the mid-height bending moment of core wall under MCE ground motions computed by the NLRHA procedure are about 3 to 5 times of those obtained from the conventional RSA procedure; the base shear demand is as high as 15-20% of the total building weight. Such high shear and bending moment demands are likely to create several design problems and difficulties. If the designer follows the RSA procedure, he might end up with an unsafe design of the building. On the other hands, if the correct but high seismic demands from the NLRHA procedure are used, the design might lead to uneconomically thick wall with a very high amount of steel reinforcement and expensive foundation structures to withstand very large lateral loading, etc. It is therefore crucial to understand why the seismic demands are such high and why the RSA procedure fails to predict the demands. The improved understanding could lead to more effective design of measures to reduce the seismic demands to acceptable levels.

In this study, the reason(s) for high seismic demands in high-rise RC core wall buildings and the failure of the RSA procedure will be investigated using a case study building. Its inelastic dynamic responses to MCE ground motions will be computed by the NLRHA procedure and then compared with those obtained from the RSA procedure. The study will enable us to gain a greater insight into this important problem, and may lead to an improved seismic design as well as a more effective control of high seismic demands for this type of structures.
2. SEISMIC DEMANDS OF A CASE STUDY BUILDING

In this section a high rise RC core wall building is selected as a case study building. Two levels of earthquake ground motions—DBE and MCE—are considered. MCE or the maximum considered earthquake is defined as the ground shaking level at the building site with a 2% probability of exceedance in 50 years, while DBE or the design basis earthquake is the level with a 10% probability of exceedance in 50 years and is assumed to be two-thirds of MCE.

2.1. The Case Study Building

The case study building is a 415-ft tall, 40-story residential tower with a typical story height of 10 ft and a lobby-level height of 20 ft. It has three levels of below-grade parking with 10 ft story height and a thick foundation slab resting on a firm stratum. The surrounding soil condition can be classified as ‘stiff soil’ equivalent to the soil type S\text{D} in the UBC 97. The lateral framing system consists of a central RC core wall and 14 peripheral columns, whereas the gravity load carrying system consists of 8-in-thick post-tensioned concrete flat slabs resting on the peripheral columns and the central core. A typical floor plan is shown in Figure 1. As shown in Figure 1, at each floor level the core wall has many openings—four in Y-direction and one in X-direction. The presence of these openings makes the core wall act like a group of walls connected to each other by coupling beams.

![Figure 1: Plan of the building](image)

2.2. Seismic Demands by the RSA Procedure

The RSA procedure in the UBC 97 (ICBO 1997) is adopted here in this study. This is because the UBC 97 has been widely used as a model code for seismic design of buildings in many countries. The design spectrum in this RSA procedure is the elastic response spectrum at 5% damping ratio of DBE in seismic zone 4 on soil type S\text{D} (Figure 2). The response modification factor ‘R’ is selected as 5.5 as the
case study building can be classified as a ‘building frame system with concrete shear walls’. It is also required that the base shear demand (or design base shear) must not be less than 90 percent of the design base shear determined by the static force procedure. To satisfy this requirement, it is necessary to replace the R factor of 5.5 by an effective response modification factor $R_{eff}$ of 4.0. With this new $R_{eff}$ factor, the seismic demands are computed for the first six vibration modes, and are combined by either the SRSS or the CQC method into the total seismic demands for design (hereinafter called ‘design demands’). The calculation of the shear demands is limited to the X-direction, which is considered to be sufficient for the purpose of this study. The design shear and moment demands of the core wall over its entire height are illustrated in Figure 3.

The obtained design demands are then utilized in the capacity design of the building structure. A kinematically admissible plastic mechanism suitable for a tall building with coupled walls is first selected. In this mechanism, ductile plastic hinges are allowed to form only at the base region of the coupled walls and at two ends of every coupling beam. The amount and distribution of longitudinal steel reinforcement of the core wall in the base region (levels 0-5) and flexural reinforcement at the two ends of the coupling beams are determined such that its nominal flexural strength times the strength-reduction factor ($\phi$) of 0.9 is approximately equal to the design base moment. The calculation of this nominal flexural strength also takes into account of the effects of gravity loads using the UBC-97 load combination rules. All other portions of the core wall and coupling beams and all other structural elements (columns, slabs, etc.) are assumed to have sufficiently high strengths to remain elastic during when ductile plastic hinges are formed in the predetermined locations. No attempt at this stage is made to determine the required strengths in all these elements, but they will be determined later by the NLRHA procedure in the following sub section.

2.3. Seismic Demands by the NLRHA Procedure

For the verification of seismic demands by the NLRHA procedure, it is necessary to have a set of ground motion records that can represent MCE. Here, the MCE response spectrum is assumed to be 1.5 times the DBE response spectrum as shown in Figure 2. Seven free-field horizontal ground motion records whose spectra resemble the target MCE spectrum are selected from the PEER NGA (http://peer.berkeley.edu/nga/) and COSMOS databases (http://www.cosmos-eq.org/scripts/default.plx). To make all these records fully represent MCE, each record is at first scaled by a constant such that its spectrum roughly match with that of MCE as shown, and then modified by adding small wavelets to the time series to finally obtain accurate spectral matching with MCE as shown in Figure 2. A spectral matching software RSPMATCH 2005 developed by Hancock et al. (2006) is used in this study. RSPMATCH 2005 employs a time-domain spectral matching technique which was originally proposed by Lilhanand and Tseng (1987) and later on improved by Hancock et al. (2006).

A nonlinear model of the case study building for NLRHA is created in Perform 3D version 4 (CSI 2006). The portion of core wall expected to remain elastic (level 5 up to the roof) is modeled by elastic shear wall elements. The core wall is divided into many horizontal layers of 10 ft thickness each, and each layer is made of 22 shear wall elements. For the plastic hinge region (levels 0 to 5), each shear wall element is a nonlinear element comprising of 8 concrete and 8 steel vertical fiber segments, resulting in 352 fiber segments per layer. A bilinear hysteretic model of non-degrading type is used for the steel fiber. The post-yield stiffness is set to 1.2 percent of the elastic stiffness. The yield strength of grade 60 steel bars is assumed to be 70.2 psi, which is 1.17 times its nominal yield strength. This is to account for the fact that actual material strengths are generally greater than nominal strengths specified by the design codes. For the same reason, the cylinder compressive strength of concrete ($f_{c'}$) is set to 1.3 times the specified strength described in Sec. 2.1. The tensile strength of concrete is set equal to $\sqrt{7.5f_{c'}}$ (ICBO
In making concrete fiber, the Mander’s stress-strain model for confined and unconfined concrete is approximated by a tri-linear envelope, and hence the confinement effects on ultimate compressive strength and post-peak strain ductility capacity by the transverse reinforcement in core wall is accounted for. The hysteretic model for concrete in compression in Perform 3D always set the unloading stiffness equal to the initial elastic stiffness. The reloading stiffness, however, can be adjusted, and it is adjusted such that stiffness degradation is increased with the increase in the plastic strain.

Coupling beams are modeled by elastic beam elements with plastic moment hinges at both ends. The moment-rotation relationship of each plastic hinge is determined from the beam’s cross-sectional properties and assumed plastic zone length of 0.5 times the beam depth. Columns and slabs are modeled by elastic column and slab/shell elements, respectively. The foundation slab is assumed to be firmly fixed to the rigid ground where the input ground motions are introduced. The geometric nonlinearity (P-Δ) effects are also accounted for.

According to recent recommendations for the seismic design of high-rise buildings, the traditionally assumed modal damping ratio of 5% is too high and not realistic for high-rise buildings (CTBUH 2008). A damping ratio of between 1% and 2% for fundamental translational modes appears reasonable for buildings more than 160 ft and less than 820 ft in height. Moreover, an analysis of damping in RC buildings measured at significant elastic response levels suggests the following relationship: \( \xi_i = 1.4 \xi_{i-1} \), where \( \xi_i \) is the damping ratio of the ith mode (Satake et. al. 2003). With all these premises, the modal damping ratios of the first six translational modes are set to 1.0% (first mode), 1.4%, 2.0%, 2.7%, 3.8%, and 5.3% (6th mode). In addition, a small amount of Rayleigh damping is added to the model to stabilize other higher frequency (and less important) modes.

With this nonlinear building model, nonlinear response analyses are performed for seven input MCE ground motions applied in the X direction. The results show, as expected, that plastic hinges are formed at the base region of the core wall and at two ends of every coupling beams in the X direction. The maximum plastic rotation at the base region of core wall for every input motion is less than 0.02 radian but constitutes more than 80% of the total inelastic energy dissipation in the building. The maximum plastic rotations at two ends of coupling beams vary approximately from 0.005 to 0.012 radian, and their combined inelastic energy dissipation is less than 20% of the total.

![Figure 2: Comparison of the matched spectra of the ground motions with DBE and MCE spectra](image)
For each input motion, the maximum shear and bending moment at every floor level of the core wall are determined. The results from all seven input motions are then compared, and the upper-bound, mean, and lower-bound values of these responses are computed and plotted against the height of the building as shown in Figure 4. These responses are seismic demands of the inelastic structure when subjected to MCE ground motions.

3. COMPARISON OF SEISMIC DEMANDS AND DISCUSSIONS

The design demands of the core wall determined by the RSA procedure are compared with the ‘true’ seismic demands evaluated by the NLRHA procedure in Figure 4. It is clear that the seismic demands are much higher than the design demands over the entire height of the core wall, and their distribution patterns are markedly different. The mean base shear demand is as high as 5 times the design base shear, and is approximately 25% of the total seismic dead load of the building. Similarly, the mean bending moment demand at the wall’s mid height is as high as 5 times the corresponding design moment. In the base region of the core wall where flexural yielding occurs, the mean moment demand is about 1.5 times the design moment. This ratio of 1.5 is essentially the overstrength factor resulting from the higher material strengths than nominal design values and the strain hardening effect of steel reinforcement. All these results are in general agreement with the study by Klemencic et. al. (2007).

There are many possible reasons contributing to this striking difference between ‘true’ seismic demands and design demands. Firstly, the ‘true’ demands are computed from MCE ground seismic motions, which are 1.5 times of DBE motions used for determining the design demands. Secondly, the realistic modal damping ratios of 1% to 5% in the nonlinear building model are generally lower than the traditional 5% damping ratio for every mode in the RSA procedure. Thirdly, the overstrength in plastic hinge zones leads to an overall increase in both moment and shear demands by a factor of about 1.5.

Apart from these three, there could be other reasons.

Nevertheless, it seems reasonable to first investigate the effects of these three obvious reasons. One MCE ground motion modified from the Superstition Hills, 1987-PR-360 record is chosen for this investigation. A DBE spectrum matched motion is created by multiplying the MCE motion by two-thirds. Another nonlinear building model where the damping ratio of every significant vibration mode is set to 5% is also created. Several NLRHAs are then carried out for different combinations of input motion (MCE & DBE) and building model (low damping & 5% damping), and the resulting seismic demands are

Figure 3: Comparison between design demands in the core wall from the RSA procedure and seismic demands from the NLRHA procedure; Figure 4: Peak core wall shear and moment demands with 1-5% damping and 5% damping at DBE and MCE level excitations.
compared in Figure 5. As expected, the seismic demands by the DBE motion are noticeably lower than those by the MCE motion; their ratio varies approximately between 1:1.1 to 1:1.75. Similarly, the seismic demands of the building with 5% damping ratio are also lower than those with lower (but more realistic) damping; their ratio varies approximately between 1:1.1 to 1:1.45. Despite all these reductions, the seismic demands computed by the building model with 5% damping ratio for the DBE motion are considerably greater than the design demands. To account for the effect of overstrength, the design demands are increased by a factor of 1.5 (shown as design demands x OSF in the Figure 4) and are compared with the above seismic demands. A considerable difference still exists, which cannot be explained by the reasons mentioned above. A more in-depth analysis is therefore required to gain further insight into this issue.

4. CONCLUSIONS

In this study, the factors responsible for high inelastic seismic demands and the large difference between demands obtained from the RS and NLRHA procedures in the high rise RC core wall buildings were identified. A case study building was examined in detail. The effect of some obvious factors such as overstrength, excitation level, and different damping ratios is quantitatively estimated using NLRHA procedure. It is observed that there is still considerable difference between the RS demands and NLRHA demands which cannot be explained by performing only NLRHA. To determine the reasons for this considerable difference, modal decomposition of the inelastic demands using UMRHA method is required, which is currently in progress.

The study is of practical importance because the detail understanding will lead to better design of the high rise RC core wall buildings as well as to design and identify control measure for reduction of the high inelastic seismic demands.

REFERENCES


