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Assessment of floor accelerations in special steel moment frames

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ABSTRACT

The floor acceleration response of Special Steel Moment Frames subjected to earthquakes is evaluated considering different modeling options, i.e., elastic and inelastic behavior, with and without gravity frames, and considering different strength levels of the partially restrained connections in the gravity frames. The characteristics of the Special Steel Moment Frames considered in this study are based on those considered in the ATC 76-1 project. The influence of each modeling option on peak floor accelerations and on floor response spectra is then investigated. Results are also compared with the design acceleration demands on nonstructural components indicated in ASCE 7, which leads to relevant observations.

1. Introduction

Structural design of buildings against seismic loads has changed over the last several years. The reasons for the changes are advances in research and experiences of real events that have occurred. Current building codes provide design rules that have, as the main goal, to limit the probability of collapse of the building during an earthquake. Recent events like the 1994 Northridge (USA), 1995 Kobe (Japan) and 2010 Maule (Chile) earthquakes have shown that the majority of buildings performed as expected against collapse. However, there were cases where the building system did not suffer any structural damage but the nonstructural damage inside the building made it impossible to occupy the facility after the event [1,2]. In hospitals this represents a major risk against life; moreover the direct and indirect economic losses can be greater than the cost of the structure [3].

Nonstructural Components (NSCs) in buildings are divided into two main categories: those that are sensitive to story drifts and those that are sensitive to accelerations [4]. Examples of NSCs sensitive to story drifts are: masonry walls, windows, interior doors, partitions, etc. Acceleration sensitive NSCs include parapets, suspended ceilings, ducts, boilers, chiller tanks, etc. [5]. The acceleration-sensitive NSCs receive the forces that arise from the motion of the structure to which they are anchored or attached, from now on denoted as the "supporting structure". The method usually used to estimate the accelerations that affect the NSCs is the Floor Response Spectrum (FRS) method. This approach computes the elastic response spectrum of a selected floor using as input the total acceleration response of the same floor. It can be applied when the interaction between the NSC and the supporting structure is not significant, i.e., when the response of the structure is essentially the same regardless of whether the NSC is present or not. In general, the interaction between the NSC and the supporting structure is not significant when the mass of the NSC is less than approximately 1.0 percent the mass of the supporting structure [6]. If the NSC mass is larger, dynamic interaction will occur between NSCs and the supporting structure, and the FRS method might in some cases produce overly conservative results [7].

Research has been performed to understand the acceleration demands on NSCs. Most of the investigations were executed considering the supporting structure to behave elastically [3,8,9]. However, several authors have studied the effect of the building inelastic behavior on the accelerations that are imposed on NSCs. The general trend is that inelastic deformations in the supporting structure reduce the acceleration demands with respect to the demands based on the elastic response of the supporting structure. However, in some cases, particularly for short-period NSCs (say, NSC period less than 0.5 s), demands based on inelastic structural response might actually be greater. Some of these studies were performed using simple SDOF structures [4,10], and others studied the effect of plasticity in MDOF structures [11–17].

Currently, nonlinear dynamic analysis is performed using models that present different levels of detail depending on the required outcome. For example, the FEMA P-695 methodology [18] requires very detailed nonlinear models that are able to capture collapse of structures when they are subjected to ground motions scaled to different levels of intensity. In the case of acceleration demands on NSCs, the mathematical models that have been studied so far usually do not have the level of detail as required by other analyses such as the FEMA P-695. For instance, Sewell et al. [12] studied a 5-story lumped-mass shear

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beam structure, therefore inelastic deformations occurred only at the springs representing the story force-deformation relationship. Rodriguez et al. [15] investigated the floor accelerations of 3-, 6- and 12-story buildings with cantilever walls. The inelastic deformations in the walls occurred only at the bottom of the first story. Medina et al. [11] studied the floor accelerations of stiff and flexible one-bay frames of 3, 6, 9 and 18 stories. The one-bay frames used in the study were designed to satisfy the strong-column weak-beam requirement. Therefore, inelastic deformations were allowed to occur only at the beam ends and at the bottom of the first story columns. Chaudhuri and Villaverde [16] studied the effect of inelastic structure response on the seismic demands on NSCs. In the investigation 4-, 8-, 12- and 16-story flexible and stiff moment steel frames were analyzed. The frames were modeled using fiber sections with a bilinear material with post-yield stiffness equal to 3% of the initial stiffness. More recently, Wieser et al. [17] studied the floor accelerations using three dimensional models. A total of four buildings were analyzed. The numerical models of the buildings used elastic beam-column elements for the gravity system and fiber sections for the lateral force resisting system. The material assigned to the fiber sections is a nonlinear steel material with 2% post-yield stiffness ratio.

These studies demonstrate the analytical sophistication of research that has been performed to characterize the floor accelerations when MDOF structures are subjected to ground motions and behave inelastically. However, it can be seen that the models that have been used did not have all the details that a complete nonlinear mathematical model could include. Fiber models can capture plastification well but since strength degradation was not included, the buildings cannot collapse under larger ground motion intensities. Additionally, models with the capability of plastification only at the beam ends and at the bottom of the first story columns cannot present story mechanisms because plastic hinges in the columns are not allowed to occur.

The main purpose of this study is to contribute to a more detailed and more realistic understanding of floor accelerations by quantifying the level of detail required in a structural model when floor accelerations are computed. This is why the structures chosen to be investigated are the Special Steel Moment Frames (SMFs) analyzed by Zareian et al. [19] for the ATC 76-1 project [20]. These models have the amount of detail required to evaluate their collapse performance using the FEMA P-695 methodology. At the moment, this methodology can be considered the most stringent in terms of requirements of nonlinear mathematical models. In order to define even further the importance of the level of detail of the model, the gravity system is also considered in this investigation. In a recent study [21], it was found that the gravity system has a significant influence on the collapse performance of the same SMFs considered in this investigation. Finally, this study also intends to obtain an accurate quantitative assessment of the level of acceleration demands that can be realistically expected in NSCs anchored to multi-story SMFs located in areas where the level of seismic activity is high, such as the western United States, Japan, and Chile.

The FRS method is used in this study to characterize the seismic demands on NSCs anchored at different floor levels in multi-story SMFs. Therefore, the interaction between the NSCs and the primary structure is neglected due to the (assumed) small mass of the NSCs (most generally the actual scenario in office and residential multi-story buildings).

2. Methodology

The numerical models of the SMFs to be analyzed were created using OpenSees [22]. The first step was to verify the accuracy of the models by performing a FEMA P-695 analysis and by comparing the results with the ones presented by Zareian et al. in the ATC 76-1 project. As specified by the FEMA P-695 methodology, the strength and stiffness of the gravity system was not included in the analysis performed by Zareian et al. In this study, however, once the models without the gravity system were verified, the gravity system was explicitly modeled in order to evaluate its possible influence on floor accelerations. The gravity system is modeled using partially-restrained (PR) connections that have different strength levels, and the gravity columns were modeled using fiber sections with a bilinear material with 10% kinematic hardening.

Once the models were validated and the gravity system incorporated, the structures were subjected to the 44 Far-Field ground motions as specified by the FEMA P-695 methodology. The level of intensity to which the ground motions are scaled is the Design Earthquake (from now on denoted simply as DE). This criterion was adopted because the acceleration demands on NSCs indicated in ASCE 7-05 [23] and in ASCE 7-10 [24] are consistent with the DE. The total acceleration response is obtained for each floor, and is then used as input to compute the floor response spectrum. In order to compute the influence of inelastic deformations in the models, the floor accelerations were also computed but with the models behaving elastically.

3. Overview of buildings analyzed

As already mentioned, the buildings analyzed were taken from the ATC 76-1 project. Complete information about the design and the nonlinear models can be found in the referenced document [20]. However, a summary of the main information is provided herein. The study performed by Zareian et al. considered buildings of 1, 2, 4, 8, 12 and 20 stories. They were designed following the ASCE 7-05 [23] requirements with the exception that the deflection amplification factor C_d was taken equal to the response modification factor, R, as specified in FEMA P-695. The gravity system was not included in the analysis as the P-695 procedure specifies.

Two different analysis methods were used to design the buildings: the Equivalent Lateral Force (ELF) method and the Modal Response Spectrum Analysis (RSA) method. All the SMFs were designed using reduced beam section connections (RBS) for a seismic design category D_{max} ($S_s = 1.5$ g, $S_1 = 0.6$ g), for a typical gravity load, and considering Site Class D.

This investigation analyzes a subset of the buildings from the ATC 76-1 project. These are the 2-, 4- and 8-story models designed using the RSA method. The base of the columns of the buildings were fixed for the 4- and 8-story models, and pinned for the 2-story model. The nomenclature that identifies these structures in ATC 76-1 is 2RSA (2 story), 3RSA (4 story) and 4RSA (8 story). Fig. 1 shows the plan view for all the buildings. The bay width (center line dimensions) between columns of each SMF is 20 ft. The height of the first story is 4.6 m (15 ft) (to top of steel beam), and the height of all other stories is 4 m (13 ft). According to ATC 76-1, this configuration is deemed representative of actual single- and multi-story SMF buildings. The design dead load (D) is 4.31 kN/m^2 (90 psf) uniformly distributed over each floor, and the cladding load is applied as a perimeter load of 1.2 kN/m^2 (25 psf). The unreduced design live load (L) is 2.4 kN/m^2 (50 psf) on all floors and 0.96 kN/m^2 (20 psf) on the roof. These loads were considered in the analysis in a combination of 1.0 D + 0.25 L.

Fig. 1 illustrates the configuration of the gravity system only in the direction to be analyzed (East-West). As already mentioned, gravity connections are considered to be PR connections. The columns at the ends of the gravity frame along Line 3 (columns A3 and F3) are part of the perimeter SMFs oriented along the N-S direction. These columns can be expected to provide some (albeit small) strength in the E-W direction even though in this direction they are part of a gravity frame and are oriented on their weak axis. This strength, however, was not considered because the sole influence of the gravity columns is intended to be evaluated.

4. Modeling approach

The numerical two-dimensional models idealized to perform the nonlinear analysis were created using OpenSees. Material and geometric nonlinearities were included in every model. In the case where