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A STUDY OF SEISMIC RESPONSE OF A BUILDING DESIGNED FOR INTERMEDIATE SEISMIC HAZARD

C. A. Arteta[1] and J. P. Moehle[2]

ABSTRACT

A ten-story reinforced concrete building with intermediate moment frames and ordinary structural walls is designed to satisfy the provisions of a set of building codes widely used in the United States and around the world. A series of 104 recorded earthquake ground motions consistent with the seismic environment of the building were selected to represent intermediate seismic hazard; the median spectrum of the set matched closely the design spectrum for the building. This allowed the assessment of the behavior of the structure as represented by a two-dimensional numerical nonlinear model. The results provide a comparison of the expected performance of a code-compliant structure and actual performance as represented by the numerical model. It was concluded that the prescriptive code-based approach does not always lead to adequate estimations of seismic demands. Of particular interest were the anticipated component shear forces, which for some key structural components were well beyond the design values. Coupled with structural detailing employed in these structures, the results indicate a high potential for relatively brittle failures in some key components during design-level shaking.

KEYWORDS: intermediate seismic zone, intermediate moment resistant concrete frames, ordinary reinforced concrete structural wall.

Introduction

Conventional seismic design practice for buildings uses prescriptive code provisions that specify required strengths and drift limits under the actions of code-specified forces. Seismic design forces are determined considering the seismic hazard at the site, the elastic dynamic properties of the structure, and seismic response modification coefficients related nominally to the ductility capacity and anticipated overstrength of the framing system. Structural elements are designed to have strength not less than the calculated (reduced) demand, with details that are consistent with the anticipated inelastic response. Given the scarcity of strong earthquake shaking to test and calibrate the design procedures, especially in regions of moderate seismicity, many of the prescriptive design procedures in today's codes are untested. A case study is presented whereby the prescriptive provisions for structural framing in a moderate seismic zone

[1]Auxiliary Professor, Dept. of Civil and Environmental Engineering, Universidad del Norte, Barranquilla, Colombia
[2]Professor, Dept. of Civil and Environmental Engineering, University of California, Berkeley, USA.

are tested through numerical analysis.

In planning the case study, it was of interest to consider structural systems that are commonly used in buildings located in regions of intermediate seismicity, and whose analysis would illustrate the effectiveness (or lack thereof) of current codes. One specific interest was the use of ordinary reinforced concrete walls as part of the seismic force-resisting system in intermediate seismic hazard zones, as permitted in US practice. Another interest was to examine procedures for estimating shear forces in reinforced concrete columns. Thus, the selected structure included both moment resisting frames and an ordinary reinforced concrete structural wall.

Two sets of design codes considered are: 1) ACI 318 [2008], ICC [2006], and ASCE/SEI 7 [2005] collectively, which are used widely in the United States and around the world, and 2) NSR-98 [1999] which is used in Colombia. A comparison of the design requirements of these codes identified differences with respect to the detailing of particular structural elements in intermediate design category buildings. By examining the simulated performance under demands imposed by real case scenarios of seismic hazard it should be possible to develop a perspective on the relative effectiveness of the different detailing provisions.

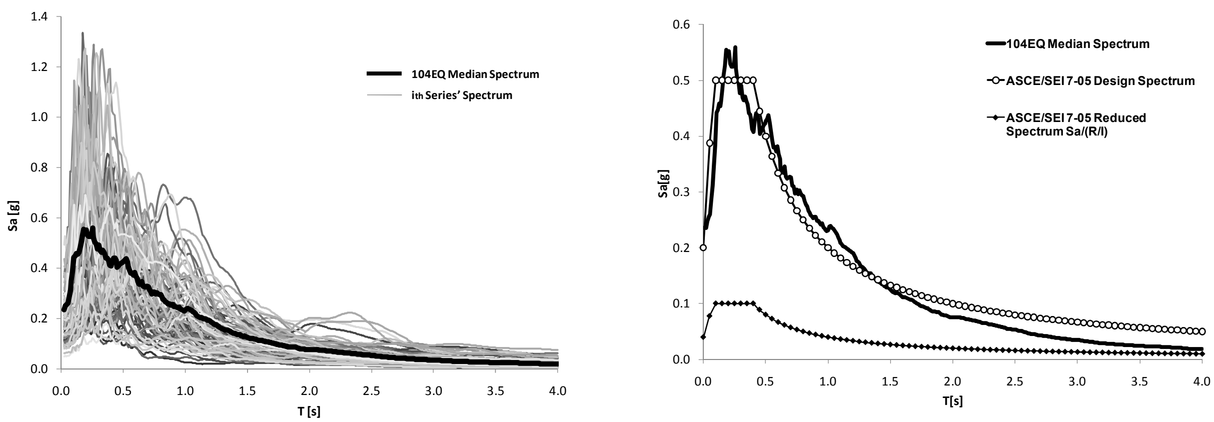
The case study is carried out by first desiging the idealized building according to building code requirements, and then constructing a planar analytical model of the building considering the nonlinear load-deformation properties, mass, and effective damping of the building. The model is then subjected to a selected bin of 104 recorded earthquake ground motions that represent the seismic hazard for the building. Results of the analysis are compared with expectations inferred from the building code.

Seismic Demand Estimation

Two different representations of seismic demand were used as input to calculate structural responses. First, an elastic response spectrum consistent with the seismic hazard level of Sacramento, CA, an *intermediate seismic hazard zone*, was used to carry out a code compliant elastic analysis and design of the structure. Then, a nonlinear analytical model of the designed structure was subjected to non-scaled acceleration time series input as uniform excitation at the base.

Following conventional practice in elastic analysis and design, the design seismic demand was calculated using ASCE/SEI 7. This code defines a *Maximum Considered Earthquake (MCE) Ground Motion* as the basis for seismic demands for which a structure must be designed. Hazard level is given by contour maps of spectral accelerations at specific periods. Design values are taken as two-thirds of the MCE values. The resultant pseudo acceleration spectrum defines a seismic hazard level having a 10% probability of exceedance in 50 years (or PE10%/50yr). Seismic ground motion values used to construct the elastic design response spectrum (5% damped) were: S_s=0.65; S_1=0.18; Site Class: C; F_a=1.15; F_v=1.65; S_{DS}=0.49; S_{D1}=0.19; Occupancy Category: I; Importance Factor: I=1.0; Seismic Design Category Based on S_{DS}: C ($0.33 \leq S_{DS} < 0.50$ and Occupancy Category I); Seismic Design Category Based on S_{D1}: C ($0.133 \leq S_{D1} < 0.20$ and Occupancy Category I). The design coefficients and factors for seismic force resisting systems used were: Seismic Force-Resisting System: Intermediate Reinforced Concrete Moment Frames. Response Modification Coefficient: R=5.0. Deflection Amplification Factor: Cd=4.5. For design, this elastic design response spectrum was reduced to account for the expected inelastic behavior (Fig. 1b).

For the more realistic assessment of structural behavior, 52 pairs of recorded EQ ground motions representative of "Intermediate Seismic Hazard Zones" [ICC, 2006] were selected from the Pacific Earthquake Engineering Research (PEER) Center, NGA strong motion database, to perform 104 non linear analyses of the structure. The seismic hazard on the building's site was characterized by the following parameters: distance range to seismic source: 6.2≤ClsD≤18.5 [mi]; NEHRP soil class: C and D 1,000≤V_{S30}≤2,000 [ft/s]); magnitude range: 6.25 ≤ M ≤ 6.75. The selected ground motions were used without any scaling since the median value of their spectral accelerations matched very closely the elastic design response spectrum from ASCE/SEI 7 (Fig. 1b).



(a) Median spectrum for 104 time series (b) Response spectra comparison

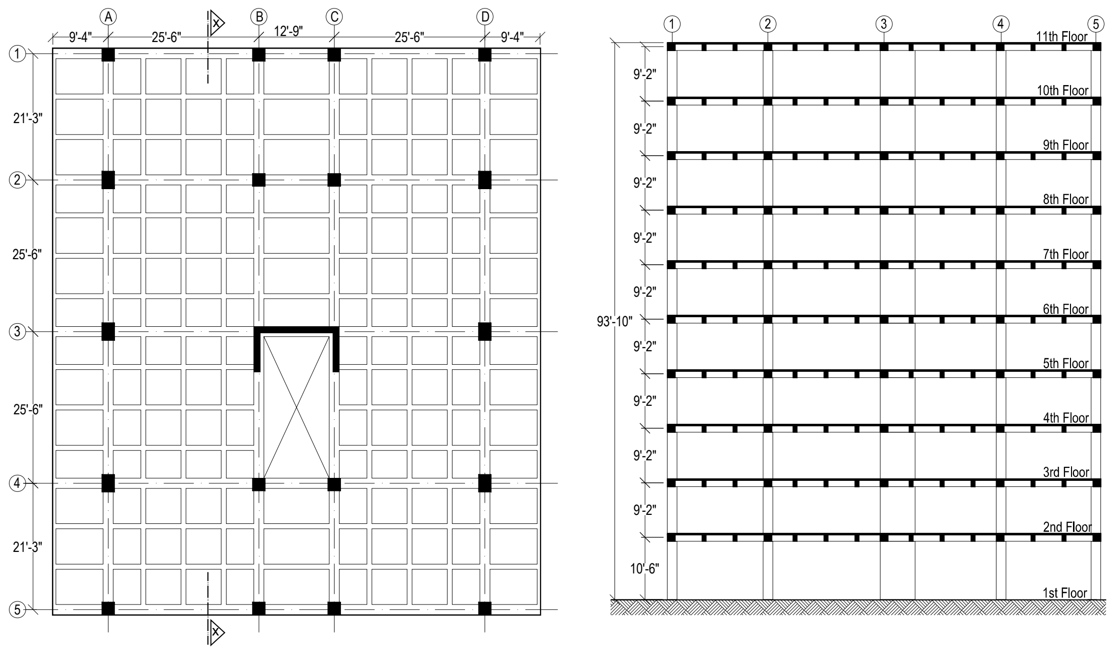
Figure 1. Response spectra (5% damped) comparison: (a) individual and median spectral values for the selected 104 ground motions; (b) code based design spectrum, reduced spectrum for design, and median spectrum of the selected 104 ground motions as representative of a realistic scenario of seismic hazard.

Structural Models

Structural Geometry

The structural system included moment resisting frames in the longitudinal and transverse directions. Frames were part of both the vertical and lateral load resisting systems and were connected through a rigid diaphragm composed of a two way joist slab with beams. A "C"-shaped reinforced concrete wall was added to the system to limit the maximum drift ratio to less than 1%. In US building codes, the drift limit is 0.02 regardless of whether ordinary, intermediate, or special framing systems are used. A drift limit less than 0.02 was selected based on concerns that the detailing of an ordinary reinforced concrete structural wall would be insufficient for drift ratio of 0.02.

Plan and elevation views of the building are shown on Figs. 2a and 2b, respectively. The building had a total of ten stories with a total building height of 93.8ft; the floor system was formed by a two way joist slab with beam span ranges (center to center) of 9.8ft to 19.7ft for a total structural area per floor of 4,680ft^2; the cross-sectional dimensions of the beams and joists were 16 x 16in and 8.0 x 16in, respectively; the two typical column cross-sectional dimensions were 20.0 x 20.0in and 20.0 x 28in; the wall thickness for the first three stories was 8in and 6in for the fourth story and above.



(a) Plan view of the floor system (b) Section x-x: elevation view

Figure 2. (a) Layout of structural elements of the floor system, columns, and shear wall; (b) vertical view of the building at section x-x.

Concrete was normal weight, with the following nominal properties for the different elements in the structure: the floor system had f'_c = 3,000 psi; columns and walls had f'_c = 4,000 psi; reinforcing bars had f_y = 60,000 psi.

Elastic Model for Code Based Analysis

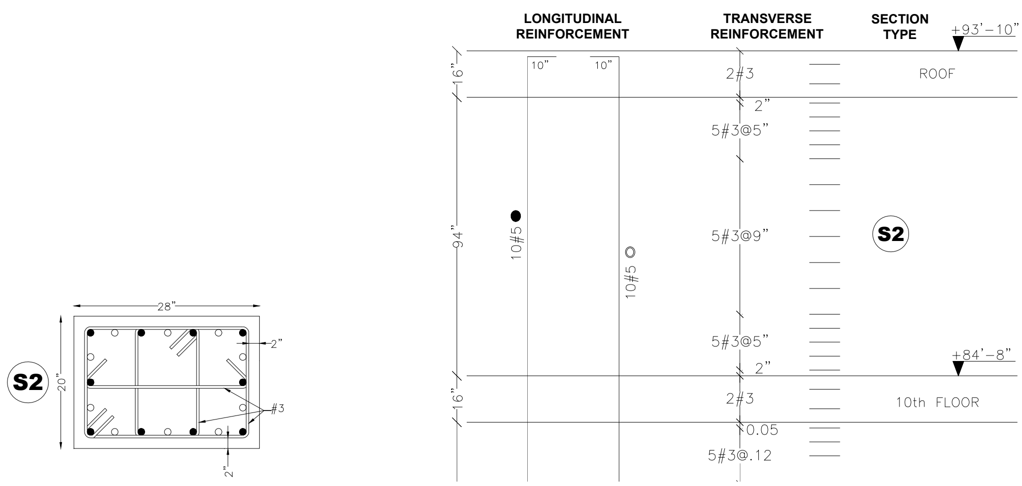
The elastic structural analysis was based on ASCE/SEI 7 Chapters 11 and 12 and ICC Chapter 6. The 3-dimensional computer model used for the analysis considered cracked section moment of inertia properties of the elements. The mass of the elements was lumped in each floor and structural elements at each story were attached by a rigid diaphragm constraint. A modal spectral analysis in two orthogonal directions was used to obtain design drift ratios limits and member forces.

Code Based Design and Detailing

The design procedure was based on the ACI 318 code Chapters 1 through 17. For detailing of the *intermediate moment resistant concrete frames*, recommendations from Chapter 21 were followed and references from ASCE/SEI 7 Chapter 14 and ICC Chapter 19 were used.

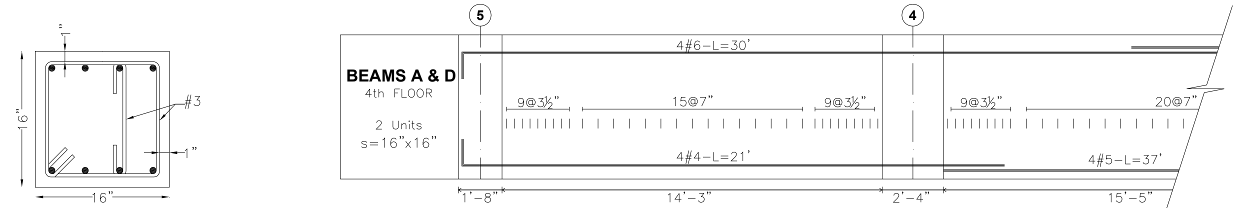
The detailing requisites for *intermediate moment resistant concrete frames* are given by ACI 318 Chapter 21.3 "Intermediate moment frames." It was found that the design was governed by minimum requirements with respect to the shear detailing of the beams and for the longitudinal and shear reinforcement of the columns. The shear walls were detailed as Ordinary Structural Walls as permitted by ICC Chapter 1908.1.4.

Figs. 3 and 4 depict typical detailing used in the column and beam elements, respectively; Figs. 5a and 5b show detailing of the wall. For columns, the longitudinal reinforcement ratio (total steel area divided by gross column area) ranged from ρ=1% to ρ=1.7%. For beams, the steel ratios (area of tension reinforcement divided by web width and effective depth) ranged from the minimum allowed quantity of ρ=0.35% to ρ=0.80%. Since the demand on the walls in the first three stories of the structure was considerable, boundary elements and a higher quantity of longitudinal (vertical) as well as transverse (horizontal) reinforcement steel were required. The vertical steel quantity ranged from ρl_{min}=0.25% to ρl=0.90% for walls in the first three stories. The horizontal still quantity was ρl_{min}=0.20%. In the upper walls (4th story and up), longitudinal steel quantities were in the range ρl_{min}=0.20% to ρl=0.33% and transverse steel quantities was ρl_{min}=0.20%.



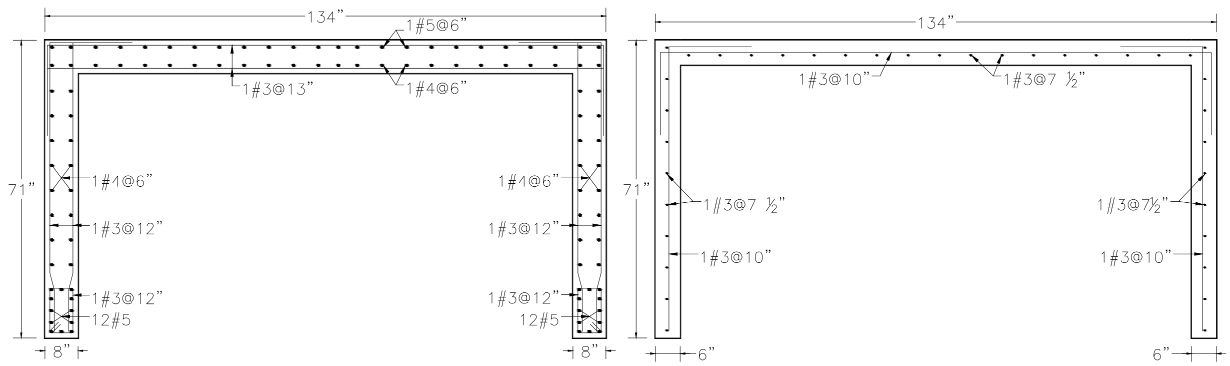
(a) Column cross section (b) Typical layout of column reinforcement

Figure 3. Example of column reinforcement (tenth story).



(a) Beam cross section (b) Typical layout of beam longitudinal and transverse reinforcement

Figure 4. Example of beam reinforcement (fourth story).



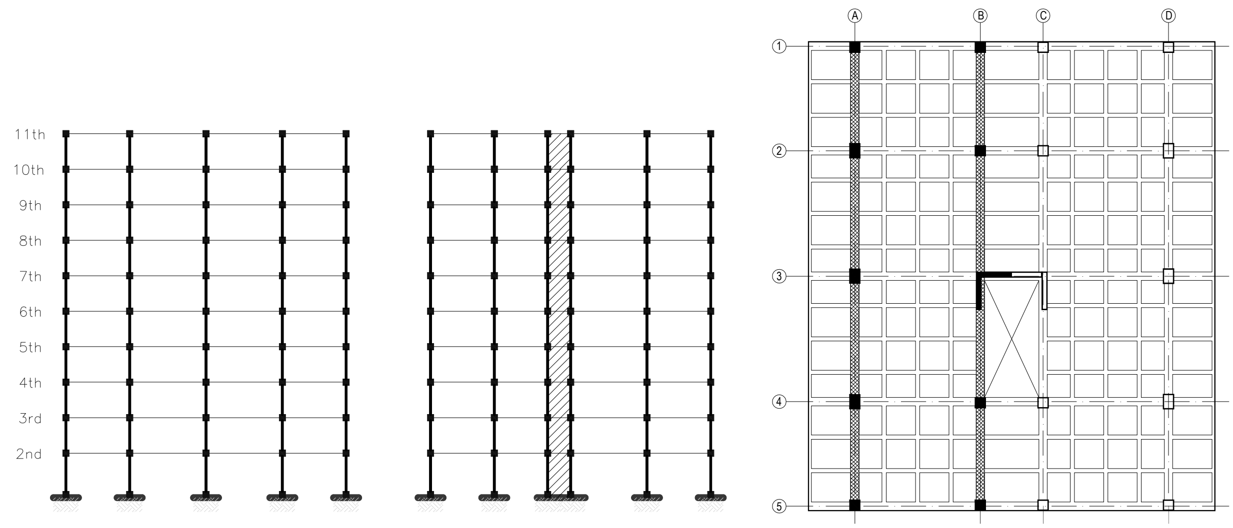
(a) Wall cross section (stories 1 – 3) (b) Wall cross section (stories 4-10)

Figure 5. Wall reinforcement.

NonLinear Model for Seismic Performance Assessment

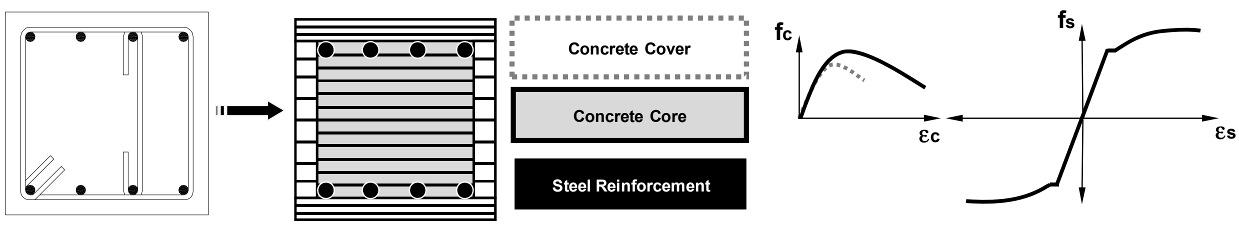
Nonlinear dynamic analysis was used to model and assess the actual behavior of the idealized building under representative earthquake ground motions. The software package Open System for Earthquake Engineering Simulation, *OpenSees* (Mazzoni et al., 2006) was selected because its nonlinear analyses capabilities have been validated by many researchers around the world and because it facilitated conducting a large number of simulations. As shown in Fig. 6a, a 2-dimensional model representation of the structure was selected for the dynamic nonlinear analyses. The symmetric geometry of the building made it possible to select half the vertical and lateral load resistant system in the longitudinal direction (Fig. 6b). To simulate the large in plane stiffness of the floors, a rigid diaphragm constraint was imposed on the joints of each level. To represent second-order effects, P-Delta type of geometric transformation was used for the columns while the small target drift ratio ($\Delta_{target} \leq 1\%$) enforced in the elastic design allowed for the use of linear geometric transformation in the beams.

Nonlinear elements with distributed plasticity and fiber sections at the integration points were used to model all structural elements. The fiber sections allowed for the use of actual uniaxial stress-strain relationships for the different materials in every section. With this approach, the variation of curvature along member lengths as well as the effects of axial load on moment-curvature relations are correctly approximated (Fig. 7).



(a) Elevation of the selected 2-dimensional model (b) Plan view with selected frames

Figure 6. (a) Configuration of the frames and the corresponding "L-shaped" portion of the shear wall in the mathematical model; (b) plan view of the selected frame and portion of the shear wall for the nonlinear analyses



(a) Beam's Section (b) Fiber Model Representation (c) Material's stress-strain curves

Figure 7. Representation of an actual structural element through a nonlinear fiber section model

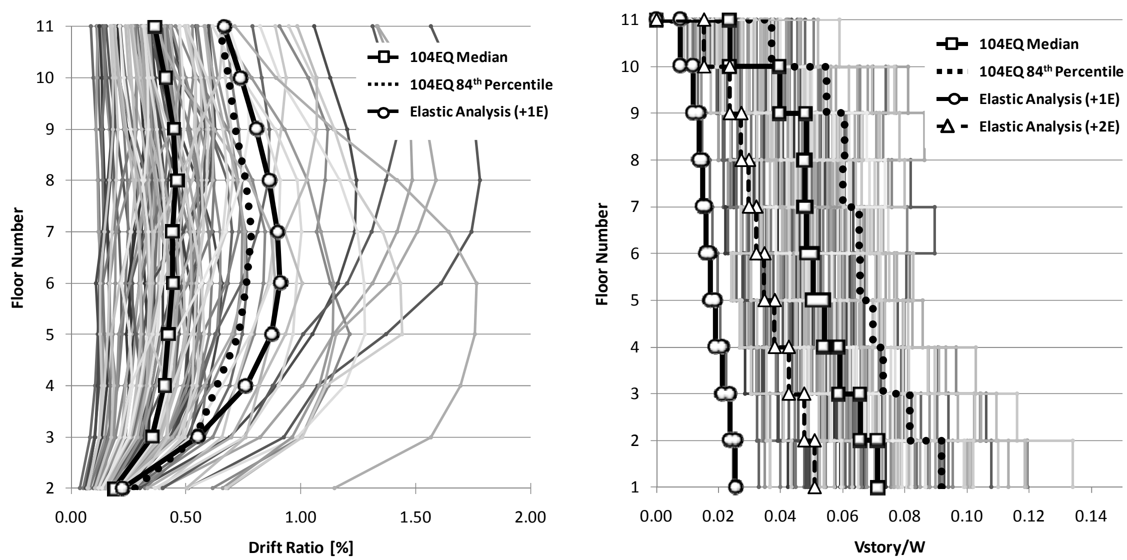
The structure was assumed to be classically damped. Mass and stiffness-proportional Rayleigh damping was used to simulate the energy dissipation characteristics of the building. A damping ratio of ζ=5% was assumed for the first and third elastic modes.

Comparison of Results from the Linear and Nonlinear Dynamic Analyses

To compare the expected performance of the code compliant design and the expected performance of the structure as represented by the nonlinear model, results are presented at two different levels of structural response: 1) global response of the structure (e.g., drift ratio per story and story shear demand) and 2) local response of key structural elements (e.g., shear wall and column shear demand).

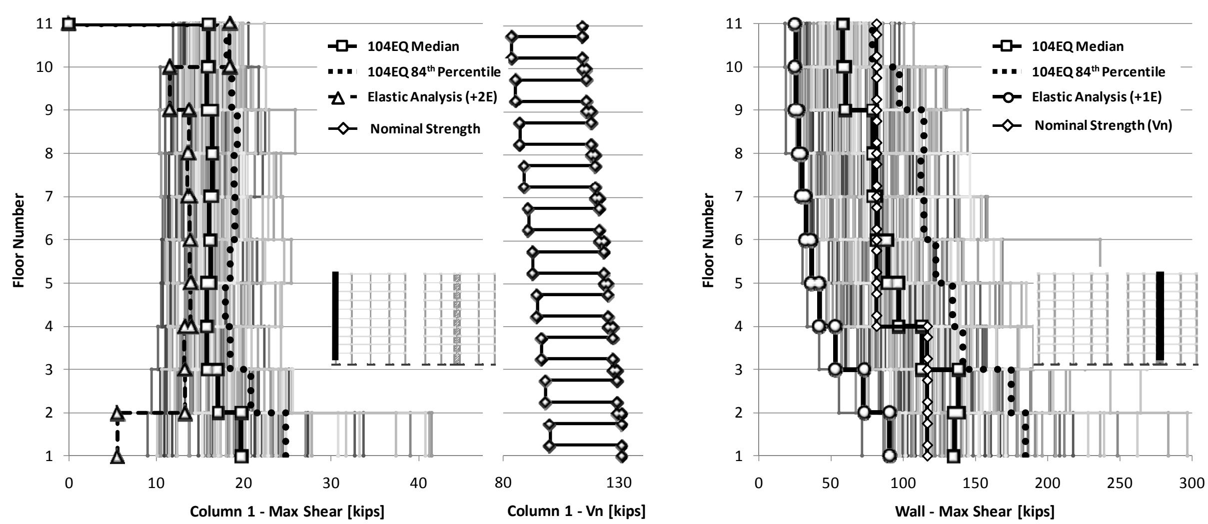
Figures 8a and 8b present two selected structural responses gathered from the different linear and nonlinear analyses performed: 1) interstory drift ratios and 2) story shears. The data shown contain values of elastic response (used for design) obtained from load combinations including appropriately factored gravity loads along with seismic load at +1E and +2E levels.

From the 104 nonlinear dynamic analyses, median values of response are presented along with values at the 84th percentile level (~ +1 standard deviation). Story drifts are conservatively estimated by the code-level forces because the cracked-section properties underestimate dominant initial stiffness properties of the structure. However, story shears are underestimated.



(a) Story drift ratio (b) Normalized story shears

Figure 8. (a) Maximum drift ratio per floor; (b) normalized maximum story shears.



(a) Extreme column (b) Shear wall

Figure 9. (a) Comparison of elastic shear design forces, nonlinear shear demand, and nominal shear strength on an extreme column; (b) comparison of elastic shear design forces, nonlinear shear demand, and nominal shear strength on the shear wall.

Figs. 9a and 9b present a comparison of the column and wall shear forces as obtained from the code compliant elastic analysis and the nonlinear dynamic analysis. As permitted by the building code, the elastic demand shown for the column is obtained from load combinations of vertical load and seismic forces at +2E level (i.e. αDL+βLL+2E). For the shear wall, the elastic demand per the building code is obtained from load combinations containing +1E level of seismic load (i.e. αDL+βLL+1E). The nominal shear strength of these structural components is also shown. For the examples shown, the calculated median shear demands exceed the code design values. These results suggest that procedures for determining design shears in intermediate systems may be unconservative.

Conclusions

A single case study considers the design and expected seismic performance of a multi-story concrete building in a zone of intermediate seismicity. It is found that the column and wall design shears are less than the median level of demand imposed by earthquake ground shaking representative of the design level. Thus, it seems likely that the case study building would fail to achieve the performance objectives of the building code.

These results suggest that building code procedures for estimation of member shears should be reconsidered. For intermediate moment frames, ACI 318 permits the column and beam design shears to be based on load combinations using 2E, in which E represents the calculated effect of earthquake loads. For shear walls, it is permitted to use the shear E calculated directly from effect of earthquake loads. Either the factors on E should be increased or (preferably) alternative procedures based on capacity design concepts should be used.

The use of ordinary reinforced concrete walls for structures in regions of intermediate seismicity also should be reviewed. The details provided in ordinary walls may be insufficient to achieve target performance objectives given the high shears to which they might be subjected. Although not shown in this paper because of space limitations, such walls also may have inadequate protection for flexural demands imposed by expected earthquakes (Arteta and Moehle, 2007).

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