SEISMIC PERFORMANCE OF TALL CONCRETE SPECIAL MOMENT FRAMES WITH HIGH-STRENGTH REINFORCEMENT

D.V. To¹, D. Sokoli², J.P. Moehle³, and W.M. Ghannoum⁴

ABSTRACT

Seismic performance of tall reinforced concrete special moment resisting frames with high-strength reinforcement is investigated through laboratory tests and nonlinear dynamic analysis. Four 20-story concrete moment frames, three reinforced with Grade 100 steel and one with conventional Grade 60, were designed in accordance with ASCE 7-16 and ACI 318-14 at a hypothetical site in San Francisco, California. All four frames had the same dimensions and concrete properties, resulting in identical design drifts. Frames with Grade 100 reinforcement were designed to have reduced amount of reinforcement providing equivalent nominal strength as the Grade 60 reinforcement model. Tests carried out as part of this study demonstrate that frames with higher-grade reinforcement had greater strain penetration, resulting in greater slip of reinforcement from connections. As a result of this, along with reduced reinforcement ratios, the frames with Grade 100 reinforcement were more flexible. In addition, many currently available types of grade 100 reinforcement have lower tensile-to-yield strength ratio and lower uniform elongation compared with Grade 60. Less strain-hardening with higher-strength reinforcement increases strain localization and P-Delta effects. Seismic response of these frame buildings with Grade 100 reinforcement is studied and compared against that of buildings with Grade 60 reinforcement.

¹Graduate Student Researcher, Dept. of Civil and Environmental Engineering, University of California, Berkeley, CA, USA (email: tovuduy@berkeley.edu)
²Ph.D. Candidate, Dept. of Civil and Environmental Engineering, University of Texas at Austin, Austin, TX 78712 (email: drit@utexas.edu)
³Ed &Diane Wilson Professor of Structural Engineering, Dept. of Civil and Environmental Engineering, University of California, Berkeley, CA, USA (email: moehle@berkeley.edu)
⁴Associate Professor, Dept. of Civil and Environmental Engineering, University of Texas at San Antonio, San Antonio, TX 78249 (email: wassim.ghannoum@utsa.edu)

Seismic Performance of Tall Concrete Special Moment Frames with High-Strength Reinforcement

D.V. To\textsuperscript{1}, D. Sokoli\textsuperscript{2}, J.P. Moehle\textsuperscript{3}, and W.M. Ghannoum\textsuperscript{4}

ABSTRACT

Seismic performance of tall reinforced concrete special moment resisting frames with high-strength reinforcement is investigated through laboratory tests and nonlinear dynamic analysis. Four 20-story concrete moment frames, three reinforced with Grade 100 steel and one with conventional Grade 60, were designed in accordance with ASCE 7-16 and ACI 318-14 at a hypothetical site in San Francisco, California. All four frames had the same dimensions and concrete properties, resulting in identical design drifts. Frames with Grade 100 reinforcement were designed to have reduced amount of reinforcement providing equivalent nominal strength as the Grade 60 reinforcement model. Tests carried out as part of this study demonstrate that frames with higher-grade reinforcement had greater strain penetration, resulting in greater slip of reinforcement from connections. As a result of this, along with reduced reinforcement ratios, the frames with Grade 100 reinforcement were more flexible. In addition, many currently available types of grade 100 reinforcement have lower tensile-to-yield strength ratio and lower uniform elongation compared with Grade 60. Less strain-hardening with higher-strength reinforcement increases strain localization and P-Delta effects. Seismic response of these frame buildings with Grade 100 reinforcement is studied and compared against that of buildings with Grade 60 reinforcement.

Introduction

The use of higher grade reinforcing steel has the potential benefit of reducing material quantities, thereby leading to reduced reinforcement congestion and reduced construction costs in reinforced concrete construction. Several steel mills in the United States can produce reinforcing steel of Grade 100 (nominal yield strength of 100 ksi) and higher. However, at the time of this writing, none of these higher grades can match the benchmark mechanical properties of Grade 60 A706 steel (Fig. 1). This raises questions about the performance characteristics of reinforced concrete construction that uses the higher-grade reinforcement. In this study, nonlinear dynamic

\textsuperscript{1}Graduate Student Researcher, Dept. of Civil and Environmental Engineering, University of California, Berkeley, CA, USA (email: tovuduy@berkeley.edu)
\textsuperscript{2}Ph.D. Candidate, Dept. of Civil and Environmental Engineering, University of Texas at Austin, Austin, TX 78712 (email: drit@utexas.edu)
\textsuperscript{3}Ed &Diane Wilson Professor of Structural Engineering, Dept. of Civil and Environmental Engineering, University of California, Berkeley, CA, USA (email: moehle@berkeley.edu)
\textsuperscript{4}Associate Professor, Dept. of Civil and Environmental Engineering, University of Texas at San Antonio, San Antonio, TX 78249 (email: wassim.ghannoum@utsa.edu)

analyses are carried out to investigate the seismic performance characteristics of tall buildings braced by reinforced concrete special moment resisting frames with Grade 60 and Grade 100 reinforcement.

![Figure 1: Mechanical Properties of Grade 60 and Grade 100 Steels](image)

### Description and Design of Buildings

Previous study completed at UC Berkeley investigated seismic response of 20-story tall reinforced concrete office buildings with special moment resisting frames and conventional Grade 60 reinforcement [12]. This archetype building shown in Fig. 2 is re-designed with Grade 100 reinforcement based on design requirements per ASCE-7 and detailing requirements per ACI 318-14. As a result, there are a total of four building models being studied including one building with conventional Grade 60 A706 (SBH60), one with Grade 100 having T/Y = 1.30 (SBH100), one with Grade 100 having T/Y = 1.18 (SBL100), and the last one with Grade 100 A1035 (SBM100). These buildings have two reinforced concrete special moment resisting frames (SMRFs) as seismic-force-resisting systems in each of the two principal directions on the perimeter of the buildings. They have four 21-ft long bays and twenty 12-ft tall stories to result in building height of 144 ft. The design floor live load is 60 psf.

The design with conventional Grade 60 reinforcement serves as the base model. From this base design, the dimensions of all structural members are kept the same and all reinforcement is replaced with Grade 100 steel. Thus, the amount of reinforcement in all structural members is reduced appropriately to provide equivalent nominal strengths. By code-based design with linear elastic analysis, the designs of all four archetype frame buildings with normal and higher-grade steel are similar except for the amount of reinforcement. All three buildings with Grade 100 are identical in design.

The design procedure follows the guidance provided in a technical brief NIST GCR 16-917-40 document [8] and [5]. The design model of the archetype frame is constructed in computer software ETABS 2016 (Computers and Structures, Inc.). The code-prescribed Modal Response Spectrum Analysis (MRSA) procedure was used for seismic design. The complete quadratic combination (CQC) was used as the modal combination rule for the first twelve (12) modes in the MRSA, which accounted for more than 98% of the modal mass. The applicable response modification factor was $R = 8$. Column moment strength is governed by design principle of strong columns and weak beams prescribed in ACI 318-14.

Table 1 presents the dimensions and longitudinal reinforcement. Note that beams and interior columns have constant cross section in the lower ten stories, with reduced sections in stories 11-20. Exterior columns were constant in stories 1-5, 6-10, and 11-20.
Table 1: Dimensions and Reinforcement of Design Frames

<table>
<thead>
<tr>
<th>Design</th>
<th>Grade 60</th>
<th>Grade 100</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zone 1</td>
<td>Zone 2</td>
</tr>
<tr>
<td>Beam</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b (in.)</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>h (in.)</td>
<td>40</td>
</tr>
<tr>
<td>Top &amp; Bottom Reinforcement</td>
<td>7 No. 10</td>
<td>7 No. 10</td>
</tr>
<tr>
<td>Ext. Col</td>
<td>b (in.)</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>h (in.)</td>
<td>42</td>
</tr>
<tr>
<td>Perimeter Reinforcement</td>
<td>28 No. 10</td>
<td>20 No. 9</td>
</tr>
<tr>
<td>Int. Col</td>
<td>b (in.)</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>h (in.)</td>
<td>42</td>
</tr>
<tr>
<td>Perimeter Reinforcement</td>
<td>20 No. 9</td>
<td>20 No. 9</td>
</tr>
</tbody>
</table>

**Analytical Model**

Numerical models for each of the beams and columns have been developed to represent the reversed-cyclic behavior observed in companion laboratory tests by the same authors of this paper. Figures 3-6 compare measured and calculated cyclic responses.

A two-dimensional numerical model of a single special moment frame in the archetype building is constructed and nonlinear history analysis (NRHA) is performed using the Open System for Earthquake Engineering Simulation software platform [6].

Seismic mass is lumped and gravity load is applied at the joints. All columns at the base level are fixed to the “ground.” All beams and columns are modeled using force-based Euler-
Bernoulli nonlinear fiber-section frame elements with five Gauss-Lobatto integration points and $P - \Delta$ geometric transformation. Beam-column joints are modeled with rigid end zones in both columns and beams (Fig. 7). Slab effects on beam stiffness and strength are not considered in the numerical model. Strain penetration of beam longitudinal reinforcement into the beam-column joints and column longitudinal reinforcement into the foundation are modeled through nonlinear rotational springs using zero-length section elements. Shear behavior in the beams is only modeled by linear elastic property.

Figure 3: Response of Specimens with A706 Grade 60 Steel – Left: Beam – Right: Column

Figure 4: Response of Specimens with Grade 100 T/Y = 1.30 Steel – Left: Beam – Right: Column

Figure 5: Response of Specimens with Grade 100 T/Y = 1.18 Steel – Left: Beam – Right: Column

Figure 6: Response of Specimens with A1035 Grade 100 Steel – Left: Beam – Right: Column

Material properties: expected material properties are used in the frame model [11]. Yield strength of Grade 60 A706 is taken to be 65 ksi, the value measured in test of specimen SBH60. Expected yield strengths of Grade 100 with distinct yield plateau (SBH100 and SBL100) are both 105 ksi in frame model but frame SBH100 has Grade 100 steel with higher strain hardening ratio as the intent of dynamic analysis study is to explore this effect on the seismic performance of two archetype frames with different types of reinforcement. Concrete strength is 1.3 times
specified compressive strength of 8 ksi.

**Damping forces:** as studied by many researchers, initial stiffness Rayleigh damping has been recognized to cause spurious forces in the system and equilibrium is not maintained [3]. Therefore, tangent stiffness Rayleigh damping is implemented in the frame model such that equilibrium is satisfied everywhere in the system. The damping matrix is defined as a linear combination of mass matrix and tangent stiffness matrix Rayleigh damping with 2% damping ratio applied in modes 1 and 3. Damping coefficients calculated from 1st and 3rd modal properties of frame SBH60 are used to define the damping matrix in analyses of all frames studied here.

![Figure 7: Typical Model at the Joint](image)

**Ground Motions**

All four archetype buildings are hypothetically located in the financial district of downtown San Francisco, California. The soil condition at the selected location is categorized as stiff soil and site class D [2]. From the USGS seismic design map, the ordinates of the pseudo-acceleration spectrum at short and 1-s periods are $S_{DS} = 1.0g$ and $S_{D1} = 0.6$, respectively, where $g$ is gravitational acceleration, for a design earthquake level and 5% damping. For the maximum considered earthquake hazard, the corresponding spectral ordinates are $S_{DS} = 1.5g$ and $S_{M1} = 0.9g$. Based on these spectral ordinates, the design and maximum considered earthquake spectra are constructed according to ASCE 7. RotD50 spectrum is also computed by dividing MCE level spectrum by 1.1 for period less than 0.6 second and by 1.3 otherwise. Dynamic analyses are performed at two levels of shaking intensities: maximum considered earthquake (MCE) and the average RotD50. Ground motions are selected using a Matlab routine developed by the Baker Research Group [4].

Twenty ground motions are selected such that the average spectrum of fault-normal (FN) component spectra of all ground motions approximates the MCE response spectrum. From these selected motions, the fault-parallel (FP) component spectrum each is scaled to agree with the RotD50 response spectrum. The set of 20 selected ground motions contains about 10 near-fault pulse-like motions that have distinct velocity pulses due to directivity effects. Pseudo-acceleration spectra and their average are shown in Fig. 9 along with target spectra.

![Figure 8: Pseudo-Acceleration Spectra – Left: FN Components – Right: FP Components](image)
Overall Results

Figure 10 (left side) plots calculated roof displacement histories of the four buildings under the FN component of ground motion recorded at station El Centro Imp. Co. Cent from the Superstition Hills-02 earthquake. Figure 10 (right side) plots calculated stress-strain behavior of a representative beam element for the four buildings. In response under this motion, the beam element in frame SBH60 sustained the least calculated strain demand, mainly because the roof deflection is the least among the four buildings. Grade 100 A1035 steel used in frame SBM100 sustains the largest strain. Frames SBH100 and SBL100 have very similar calculated strains that are between strains calculated for Frames SBH60 and SBM100.

Figure 11 plots the mean of the maximum lateral drifts over height, normalized by building height. It is apparent that frame SBH60 with conventional Grade 60 A706 reinforcement achieves the least roof drift of 1.15% while frame SBM100 with Grade 100 A1035 produces the largest roof drift of all frames at about 1.45%. Buildings SBH100 and SBL100 both obtain equivalent roof drift of 1.3% that is between those of frames SBH60 and SBM100. Figure 12 plots the mean of the maximum story drift ratios, for which similar observations can be made. The spectral ordinates of FP-component motions were intended to be about 80% of the FN-component ordinates. Observing the results in Fig. 11, average drifts of all frames subjected to FP-component motions is approximately 80% of those under FN-component motions as expected since the average spectra of these components matched their corresponding target spectra reasonably well.

Figure 13 plots the mean of the maximum absolute values of story shears. Frame SBL100 attracts the least amount of story shear (approximately 10% of seismic weight); this observation is consistent with the lower strain-hardening ratio for the reinforcement used in this frame (T/Y = 1.18). SBH100 with Grade 100 having higher strain-hardening (T/Y = 1.30) develops slightly more shear force. Frame SBH60 with larger amount of reinforcement attains much larger story shear force, probably because of its higher stiffness and strain hardening as compared with SBH100 and SBL100. Frame SBM100 developed the largest story shear, possibly associated with its higher strain-hardening. Nonlinear-static analyses showed that Frame SBM100 was stronger than the other three frames considered in this study.
Shear forces on individual columns were calculated for each input ground motion. Results are organized by interior and exterior columns and by fault normal and fault parallel ground motions. Figures 14 and 15 present shears for individual ground motions and average values for each suite of motions, all normalized by $A_g \sqrt{f'_c}$, where $f'_c$ is in psi units.

Both interior and exterior column normalized shears show a sudden reduction occurring in story 10 (Figures 14 and 15). This reduction is because the column cross-sectional area (used to normalize the results) increases going from story 11 to story 10. Exterior column shears also show a notable increase in the lowest story (Figure 15), which is not apparent for interior columns (Figure 14). The increase is associated with beam growth, which appreciably increases the column displacements in one direction and, hence, increases the maximum column shear.
Figure 14: (a) Interior Column Shear in All Frames – FN Component

Figure 14: (b) Interior Column Shear in All Frames – FP Component

Figure 15: (a) Exterior Column Shear in All Frames – FN Component

Figure 15: (b) Exterior Column Shear in All Frames – FP Component
ACI 318-14 contains requirements for calculation of the column design shear force. As a minimum, the column design shear shall be at least equal to the shear determined from the applicable load combination including effect of earthquake loading; here we refer to this shear as $V_{MRSA}$. Additionally, column design shear shall not be less than shear corresponding to the development of the column probable moment strength $M_{pr, col}$ at both ends of column, resulting in $V_{u,i} = \sum M_{pr, col,i} / l_{u,i}$. For tall building frames with large columns, this requirement can result in very large design shears. As an alternative, ACI 318-14 allows that the column shear need not exceed the shear associated with occurrence of the probable moment strengths $M_{pr, beam}$ of beams framing into the beam-column joints above and below the column. A common practice is to assume that the probable moments from the beams are distributed equally to the columns above and below the joints, resulting in column shear of approximately $V_{u,i} = \frac{\sum M_{pr, beam,i}}{2l_{u,i}}$. (In the first story, base fixity results in $\sum M_{pr, beam,i} = \infty$, so it is more appropriate to substitute $M_{pr, col,i}$ at base of the building if the columns are fixed against rotation).

Visnjic et al. (2014) [13] and Moehle (2014) [7] proposed an alternative approach whereby the shears from linear analysis are amplified to account for effects of anticipated nonlinear dynamic response of the building frame. Based on this procedure, column design shears can be calculated by:

$$V_u = \omega \Omega_0 V_{MRSA}$$

where $V_{MRSA}$ = column shear obtained from modal response spectrum analysis
\[ \omega = 1.3 \text{ as a dynamic amplification factor} \]
\[ \Omega_0 = \text{overstrength of structural system, which can be approximated as} \]

$$\Omega_0 = \frac{\sum M_{pr}}{\sum M_{u,MRSA}}$$

\[ \sum M_{pr} = \text{sum of probable moment strengths of all beam and column plastic hinges in a beam-yielding mechanism} \]
\[ \sum M_{u,MRSA} = \text{sum of the moments calculated from modal response spectrum analysis at all beam and column plastic hinge locations of the same beam-yielding mechanism in absence of gravity loads.} \]

Column shear forces calculated by the various approaches are plotted and compared against the average of those from nonlinear dynamic analyses in Figs. 14 and 15. As expected, $V_{u,i} = \sum M_{pr, col,i} / l_{u,i}$ results in large overestimation of column shears in all cases. $V_{u,i} = \frac{\sum M_{pr, beam,i}}{2l_{u,i}}$ provides reasonable approximation of shear in exterior columns but overestimates that in interior and middle columns in the first story of buildings. The last approach using $V_{MRSA}$ and adjustment factors produces the best estimate of shear in all exterior, interior, and middle columns. However, it is worth noting that the shear in the exterior columns of the first story is underestimated by this method as it does not account for the effects of beam elongation.

It can also be observed that the last method slightly overestimates shear in exterior columns for frames SBH100 and SBL100 as these frames are reinforced with higher-grade steel that has lower strain-hardening ratio than that of conventional Grade 60 A706. Hence, the overstrength factor of the structural system is overestimated for these two frames.
Conclusions

1. Building frames SBH100, SBL100, and SBM100 with Grade 100 were less stiff than SBH60 with conventional Grade 60 A706.
2. Frames with higher-grade reinforcement sustained more drift than that of a nominally identical frame with Grade 60 steel. SBM100 with Grade 100 A1035 that had round-house-shaped stress-strain relationship had the largest drift. SBH100 and SBL100 had similar drift in spite of slight difference in strain-hardening in reinforcement properties.
3. Frames with high-strength steel and higher strain-hardening attracted larger base shear than other frames.
4. ACI 318-14 procedures for determining the column design shears can significantly underestimate shears calculated by nonlinear dynamic analysis. An alternative procedure presented here produces improved estimates.

Acknowledgments

This study is supported by Charles Pankow Foundation, the CRSI Foundation, and ACI Foundation’s Concrete Research Council.

References

1. ACI 318 (2014). Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, American Concrete Institute, Farmington Hills, MI.