CONTRIBUTION of GRAVITY FRAMES TO SEISMIC PERFORMANCE OF STEEL MOMENT RESISTING FRAMES

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ABSTRACT:

Traditionally, lateral stiffness and strength of the gravity frames in steel buildings are neglected in structural analysis. During the past earthquakes, such as Northridge, USA, 1994 and Kobe, Japan, 1995, unexpected failures at beam-to-column connections were observed at steel moment resisting frames (MRFs). In the aftermath of these earthquakes, extensive research has been carried out to reveal the causes of these failures. Based on the detailed observations, it is likely that the reserve capacity provided by the gravity frames prevented the highly damaged steel buildings from collapsing, since majority of the moment-resisting connections failed prematurely during the 1994 Northridge earthquake. Even though the influence of gravity frames on structural behavior can be substantial, little attention is paid to evaluate its impact on structural response. This paper evaluates the contribution of interior gravity frames (GFs) in seismic performance of special moment resisting steel frames (SMRFs). For this purpose, 4- and 9-story SMRFs were designed in accordance with the requirements of Draft Turkish Seismic Code (2016). The frames are, then, subjected to incremental dynamic analysis. In order to evaluate the contribution of the interior GFs to the overall seismic performance of the structural system, inelastic behavior of shear tab (simple) connections at beam-to-gravity-only columns were idealized as semi-rigid joints. A general purpose structural analysis software, ETABS, is utilized for the analyses. The results of the study are presented in terms of story drifts, base shear vs. roof displacement.

KEY WORDS: Gravity-Only Frames, Steel Moment Frames, Seismic Design

1. INTRODUCTION

In the current design practice, it is typical to separate the gravity load-carrying and Lateral Force Resisting Systems (LFRS) for an economical design of steel buildings. LFRS are, in general, arranged on the perimeter of a building while interior frames comprise Gravity Frames (GF), which are responsible for withstanding vertical loads. Traditionally, lateral stiffness and strength of the gravity frames in steel buildings, which incorporate shear connections are neglected in structural analysis. However, the comprehensive observations after the past earthquakes, indicated that the effect of the gravity columns on overall structural performance can be substantial. For instance, majority of the pre-Northridge moment connections experienced brittle fractures during the 1994 Northridge and 1995 Kobe earthquakes but still none of the steel structures collapsed partially or entirely (FEMA 267, 1995). This was attributed to the impact of the reserve capacity provided by GFs (FEMA 267, 1995; Lui and Astaneh-Asl 2000). Subsequent to unveiling the potential significance of the gravity columns, recent research showed that bending moment transferred from beam to column through shear connections, depending on the composite action, may reach 20% to 40% of the beam’s plastic moment capacity (Lui and Astaneh-Asl 2000;
2004). It is, in fact, revealed that shear connections act as semi-rigid (SR) connections rather than pin connections (Lui and Astaneh-Asl, 2000; 2004). This paper focuses on the contribution of gravity frames to seismic performance of steel buildings that conventionally designed to resist seismic loads solely by special moment resisting frames (SMRFs). For this purpose, low- and mid-rise steel buildings are designed in accordance with the current design codes (TSC, 2016; AISC 360, 2010). The influence of the gravity frames is evaluated in terms of drift, roof displacement and rotational demands through Incremental Dynamic Analysis (IDA). Finally, conclusions are drawn to extend our understanding of the actual behavior of steel buildings with perimeter SMRFs.

2. STRUCTURAL MODELS

2.1. Design of Low-Rise (4-Story) and Medium-Rise (9-Story) SMFRs
Two typical steel moment frames representing the low-rise (4-story) and medium-rise (9-story) steel buildings, as shown in Figures 1 and 2, were designed based on the seismic design requirements of Draft Turkish Seismic Code-2016 (TSC, 2016) and the provisions of AISC 360 (2010). The footprint of each building is symmetrical. The four-story building has plan dimensions of 36.0 m by 54.0 m with four 9-m bays and six 9-m bays in the two orthogonal directions, respectively, and a typical story height of 4 m (Figure 1).

![Figure 1. Plan (a) and elevation (b) of the 4-story frame](image)

The nine-story building has plan dimensions of 45.0 m by 45.0 m and consists of five bays of framing in both orthogonal directions spaced at 9 m on center. The building has a basement level (level B1 in Figure 2b).

![Figure 2. Plan (a) and elevation (b) of the 9-story frame](image)
The typical story height is 4 m except at the ground and basement levels, where it is 5.40 m and 3.60 m, respectively (Figure 2b). The structural system for each building consists of steel perimeter moment resisting frames and interior simply connected framing for gravity; that is, lateral loads are merely carried by two special moment resisting frames (SMRFs) arranged on the perimeter as shown in Fig. 1(a) & Fig. 2(a). The buildings were assumed to be located at high seismic area which are designed for a site in the main campus of Gebze Technical University, where short period (S_s) and long period (S_l) map spectral acceleration coefficients are 1.58g and 0.82g, respectively. Site class ‘ZB’ is considered in design (TSC, 2016). The SMRFs were designed using structural system behavior factor of R=8 and overstrength factor of D=3. In addition to the beam and column element self-masses, super-dead and live loads of 5.0 kN/m² and 3.0 kN/m² (4.0 kN/m² & 1.0 kN/m² for the roof), respectively, were used in design. The TSC-2016 design base shears corresponding to the 4- and 9-story buildings were determined as 5875 kN and 6306 kN, respectively, based on the Modal Response Spectrum Method. The beams and columns are made of steel European I-Sections with minimum specified yield strength of F_y = 345 MPa. The final member sizes were selected considering the seismic design requirements stipulated in AISC 341 and are summarized in Table 1. Braced frames, shown in Figures 1 and 2, are used as the seismic force resisting system in the direction perpendicular to the moment frames.

Table 1. Structural Member Sizes in SMRFs and GFs

<table>
<thead>
<tr>
<th>Story</th>
<th>SMRFs_Columns</th>
<th>SMRFs_Beams</th>
<th>Gravity Frames_Columns</th>
<th>GFs_Beams</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Exterior</td>
<td>Interior</td>
<td>Exterior (Braced B.)</td>
<td>Interior</td>
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<tr>
<td>4-Story B.</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>4</td>
<td>HD 400 \times 421</td>
<td>HD 400 \times 551</td>
<td>HE 700 A</td>
<td>HE 280 B</td>
</tr>
<tr>
<td>3</td>
<td>HD 400 \times 421</td>
<td>HD 400 \times 551</td>
<td>HE 700 A</td>
<td>HE 280 B</td>
</tr>
<tr>
<td>2</td>
<td>HD 400 \times 551</td>
<td>HD 400 \times 634</td>
<td>HE 800 A</td>
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</tr>
<tr>
<td>1</td>
<td>HD 400 \times 551</td>
<td>HD 400 \times 634</td>
<td>HE 800 A</td>
<td>HD 320 \times 245</td>
</tr>
<tr>
<td>9-Story Building</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>HD 400 \times 382</td>
<td>HD 400 \times 463</td>
<td>HE 600 A</td>
<td>HE 280 B</td>
</tr>
<tr>
<td>8</td>
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<td>HD 400 \times 463</td>
<td>HE 600 A</td>
<td>HE 280 B</td>
</tr>
<tr>
<td>7</td>
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<td>HE 700 A</td>
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<td>HD 400 \times 818</td>
<td>HE 800 A</td>
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</tr>
<tr>
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<td>HD 400 \times 818</td>
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<tr>
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<td>HD 400 \times 818</td>
<td>HE 800 A</td>
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<tr>
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<tr>
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<td>HD 400 \times 900</td>
<td>HE 900 A</td>
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<tr>
<td>Basement</td>
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<td>HD 400 \times 900</td>
<td>HE 600 A</td>
<td>HD 400 \times 634</td>
</tr>
</tbody>
</table>

2.2. Beam-to-Column Connections in SMFRs and GFs

In this study ‘Full Penetration Welded Joint’ connection detail, as described under Annex 9B item 9B.4 of TSC (2016), has been considered as the typical beam-to-column assembly for the whole SMRFs’ connections (Fig. 3a). Typical beam-to-column connection at GFs is also shown in Fig. 3(b). Three analytical models (Case A, B and C) were compared in order to investigate the contribution of the interior GFs in seismic performance of SMRFs. Case-A model represented the typical analytical model, in which the contribution of the GFs is not included in the model. On the other hand, the connections in GFs were considered as semi-rigid moment connections in Case-B and Case-C models. Case B is modeled as bare steel frame while the composite deck effect on M-θ relationship of shear connections is considered in Case-C. P-Delta effects are taken into account for all cases.

Figure 3. Typical SMRF (a) and GF (b) systems beam-to-column connection details
2.3. Special Moment Resisting Frame Elements’ (Nonlinear Behavior) Model
Nonlinear modeling parameters described in Table 9-6 of ASCE 41 (2013) were adopted for the moment connections in SMRFs. Fig. 4(a) presents the force-deformation relationship used for the beam and beam-column elements. Relevant a, b and c values corresponding to Q-θ relation are taken as 9θ_y, 11θ_y and 0.6, respectively.

![Generalized Force-Deformustion Relation for Steel Elements or Components (ASCE 41-13)](a) Generalized Force-Deformustion Relation for Steel Elements or Components (ASCE 41-13)

![Software Input Data of SMRF Elements’ Nonlinear Hinge Model](b) Software Input Data of SMRF Elements’ Nonlinear Hinge Model

Figure 4. Typical nonlinear behavior model for SMRF elements

Figure 4(b) represents the typical nonlinear parameters modelling input data of beam-columns. Note that the strain hardening is assumed to be zero. Acceptance criteria for IO, LS and CP were implemented as 1θ_y, 6θ_y and 9θ_y, respectively, in accordance with the limits prescribed in TSC (2016).

2.4. Gravity System Beam-to-Column Connections’ Model
Analytical model of composite beam-to-column connection proposed by Shen et al. [6, 9] and Wen et al. [7], was used in this study (model Case-C). A simplified shear connection model (model Case-B) [9], was also included for the sake of the comparison. Fig. 5(b) shows the details of the GFs connection models in Case-B and Case-C.

![Idealized GF Joint Connection Model](a) Idealized GF Joint Connection Model

![Beam-to-column connection models in GFs](b) Beam-to-column connection models in GFs [9]

Figure 5. Beam-to-column connection models in GFs (Case-B & Case-C)

In Case-B, GF connections were represented by semi-rigid connection model with a rotational stiffness of 10% of (4EI/L) and a flexural strength of 20% (M_p) of the plastic moment capacity of the connecting beams (IPE A 500). Note that the Case-B model underestimates the positive bending strength by neglecting composite action, but overestimates the negative bending strength, as shown in Fig. 5(b).

2.5. Modelling Assumptions
3D modeling of 4-Story and 9-Story buildings are shown in Figure 6. There were two SMRFs in both buildings whereas five and four GFs existed in the direction of excitation, respectively, for 4- and 9-story buildings. The plastic hinges were assigned at the column face in SMRFs. As described in section 2.3, material non-linearity is incorporated in the models through the nonlinear hinge models, which are assigned to beam and beam-column elements in SMRFs as (M3) and (P-M3) hinges, respectively.
Concrete deck is assumed to be rigid. In Case-A, shear connections were modeled as pin connections. However, in Case-B and Case-C models, these connections were rectified in compliance with the prescribed connection models as identified in item 2.4 above. To represent the true behavior of the connections, Link element properties (multilinear M-θ relation) were used as shown in Figure 7 below.

3. NONLINEAR DYNAMIC ANALYSIS PROCEDURE

3.1. Earthquake Ground Motion (GM)
The ground motion (GM) acceleration record of 1999 Kocaeli earthquake (recorded at Düzce) was selected in order to initially investigate the 4-Story and 9-Story buildings. Details of GM record and the corresponding Response Spectrum (with %5 damping) is shown in Fig. 8 below. The peak ground acceleration (PGA₀) of GM equals to 0.31 g.
3.2. Nonlinear Incremental Dynamic Analysis (IDA)
The Incremental Dynamic Analysis (IDA) approach was utilized to investigate the seismic responses in terms of story drifts and base shear vs. roof displacement. Each time history analysis was carried out using non-linear direct integration method available in ETABS. The fundamental periods of the 4-story buildings are 1.14 s, 1.12 s and 1.09 s, respectively, for models Case-A, B and C. In the 9-story buildings, the fundamental periods for the three cases were 2.44 s, 2.40 s and 2.33 s, respectively. Note that the influence of the GFs on the initial stiffness of the buildings was noticeable. The spectral accelerations were increased with 0.10g, 0.05g increments for 4-story and 9-story structures, respectively. The scale factor (SF) was determined in such a way that the ground motion intensity with a scale factor of SF would have a spectral acceleration, $S_a$, equal to $(SF) g^{[10]}$.

4. RESULTS AND DISCUSSION

Figure 9 plots the IDA curves of 4- and 9-story buildings, relating peak story drift ratios obtained from the frames to the “first-mode” spectral accelerations. It seems that the impact of GFs became more substantial for both 4-and 9-story buildings, as the intensity of the GMs increases. It is also noteworthy that 4-story buildings followed a similar trend while the response obtained from Case C of 9-story buildings differ from other two cases after a story drift ratio of 4%. This can be attributable to unsymmetrical hysteretic behavior of shear connections combined with higher-mode effects. The 4-story and 9-story frames reach their 4.7% design drift ratio approximately at spectral accelerations, $S_a$, of 1.0 g/1.05 g/1.15 g and 0.70 g/0.75 g/0.80 g, respectively, for Case-A, B and C.

![Figure 9. Peak Drift Ratio development of the 4-story (a) and 9-story (b) buildings subject to GM](image)

The 4-story and 9-story frames reach their design roof displacement of 474 mm and 936 mm, respectively, approximately at spectral accelerations, $S_a$, of 0.95 g/1.10 g/1.15 g and 0.55 g/0.60 g/0.70 g, respectively, for Case-A, B and C. As shown in Figure 10, base shear forces corresponding to abovementioned design roof displacements were obtained as 20751 kN/22942 kN/24080 kN and 26375 kN/27311 kN/28597 kN, respectively, for 4-story and 9-story building models. Note that the design base shear forces for 4-story and 9-story buildings were 5875 kN and 6306 kN, respectively.

![Figure 10. Base Shear vs. Top Displacement relation of the 4-story (a) and 9-story (b) buildings](image)
Plastic hinge distributions are also presented in Fig.11 and Fig.12, respectively, for the 4-story and 9-story building models. Note that the contribution of the interior GFs to the overall seismic performance of the structural system become noticeable in 4-story building, in which the near collapse behavior was prevented. Furthermore, in 9-story building models, contribution of the GFs was apparent at SF=0.75 g. Hinge rotations representing the IO, LS and CP performance levels are figured as: \( \text{IO} \leq 10^\circ, \text{LS} \leq 60^\circ, \text{CP} \leq 90^\circ \), as described in item 2.3 above.

![Figure 11. Hinge Formation of 4-story SMRF at SF=1.15 g](image)

![Figure 12. Hinge Formation of 9-story SMRF at SF=0.75 g](image)

Fig. 13 plots the peak story drift ratio distributions along the building height for the 4-story and 9-story buildings when the scale factors were, SF=1.15 g and SF=0.75 g, respectively. Finally, the maximum rotational demand corresponding to these SFs on beam-to-column connections of SMRFs were examined and found to be 0.041 rad/0.036 rad/0.031 rad and 0.045 rad/0.039 rad/0.035 rad., respectively, for the 4-story (story level 1) and 9-story (story level 2) buildings analysis models (Case-A, B & C).

![Figure 13. Drift Ratio distributions along 4-story (a) and 9-story (b) buildings’ height](image)

4. CONCLUSION

Typical 4-story and 9-story buildings with SMRFs were designed in accordance with the requirements of Draft Turkish Seismic Code-2016 and subjected to ground motion records in order to investigate the contribution of the
GFs to the overall performance of the lateral load resisting system. The goal was to assess whether or not the additional stiffness and strength provided by the GFs will contribute to the performance of the SMRFs. The resulting seismic responses were discussed in terms of peak drift ratio, top displacement and base shear versus top displacement, and major conclusions are noted as follows;

(a) The lower and upper bound of design drift ratio, as per TSC-2016, were identified as 2,35% and 4,7%. The 4- and 9-story buildings were designed considering the upper bound of drift ratio as 4,7%. Ground motion intensities fulfilling the design drift ratio were noted as maximum as SF=1,15 g and SF=0,75 g, respectively, in Case-C models of the 4-story and 9-story buildings, in which the contribution of composite slab were included in the numerical models.

(b) Considering the all the response indicators used to evaluate the performance, such as GM spectral acceleration intensity, $S_a$ and the base shear forces corresponding to the peak roof displacement, it can be concluded that the response of the GFs with bare shear tab beam-to-column connections (Case-B) and GFs beam-to-column connections with the composite concrete slab (Case C) were %7 and %15 larger than that of the conventional SMRF models, respectively.

(c) It is apparent that including the contribution of GFs in numerical models improved the overall performance of the steel buildings, especially when composite slab was involved. However, further investigation is needed to improve the assessment.

REFERENCES


