EFFECT OF MOMENTS DUE TO LATERAL DEFORMATION IN GRAVITY COLUMNS

Dane E. Huggard[1] and Paul W. Richards[2]

[1] *Masters Student, Dept. of Civil and Environmental Engineering , Brigham Young University, Utah, USA*
[2] *Assistant Professor, Dept. of Civil and Environmental Engineering, Brigham Young University, Utah, USA*
Email: huggardce@gmail.com, prichards@et.byu.edu

ABSTRACT :

In steel buildings, all columns of a story experience similar drifts under seismic loads when floor diaphragms are rigid. Columns in lateral-force-resisting frames are designed with these deformations in mind, but gravity columns often are not. When gravity columns are continuous, moments develop when interstory drifts occur. Using nonlinear dynamic analysis software, moments due to lateral deformation from seismic forces were obtained for lateral-force-resisting columns within 3- and 9-story buckling-restrained-brace frames (BRBFs). Moments in gravity columns were then determined. Gravity columns were evaluated with interaction equations to determine whether load demands exceeded column capacity. For most gravity columns, column capacity was exceeded. Recommendations for improved gravity column design are given.

KEYWORDS: Gravity column design, lateral deformation, moment demand, BRBF

1. INTRODUCTION

In steel buildings, all columns of a story experience similar drifts under seismic loads when floor diaphragms are rigid. Columns in lateral-force-resisting frames are designed with these deformations in mind, but gravity columns are often not. When gravity columns are continuous, moments develop when interstory drifts occur.

Recent experiments have indicated that columns in lateral force resisting frames can carry large axial loads after flexural hinges form at each end. Newell and Uang tested nine W14 specimens under combined axial and lateral loads (Newell and Uang 2006). Columns achieved drifts of 7 to 9% while carrying significant axial loads. The columns in that study were relatively heavy sections with seismically compact flanges. It is unclear if columns with non-compact flanges can perform acceptably at large drifts.

This paper discusses analyses performed to determine whether the combination of axial loads and moments due to lateral deformation exceeds the elastic capacity of gravity columns designed according to U.S. provisions. From the results, possible recommendations for modifications of gravity column design will be discussed.

2. METHODS

2.1. Lateral-Force-Resisting Frame Design

Eight buildings were designed representing one system (BRBFs), two heights (3-story, 9-story), and four strength levels (discussed later). Building plan dimensions and floor masses matched those used in moment frame studies (Gupta and Krawinkler 1999). Seismic weights for the 3- and 9- story buildings were 31.8 and 97.3 MN (1790 and 5470 kips). Braced bays were located around the perimeter of the buildings. BRBFs had braces in the two-story-X configuration (Fig. 1).

Braced frames were designed according to the 2006 International Building Code (ICC 2006) equivalent lateral force procedure and Seismic Provisions (AISC 2005a). A Los Angeles, California site was used for design with SDS=1.11 and SD1=0.61, where SDS and SD1 are the site design spectral accelerations at 0.2 and 1.0 seconds in terms of gravity. Base shear coefficients, Cs, for an importance factor, I, of 1, for 3- and 9- story frames are 0.16 and 0.08 respectively (ICC 2006).

The importance factor in the base shear equations introduces the possibility of different design base shears for buildings of similar height and system at the same site. Four strength levels (design base shears) were considered for each building height (Cs=0.15, 0.20, 0.25, 0.30 for 3-story buildings and Cs= 0.06, 0.09, 0.12, 0.15 for 9-story buildings). Buildings with Cs values lower than those corresponding to an I of 1 were included in the study to investigate the effects of low lateral strength on column demands.

Brace areas and column shapes for the frames are given in Tables 2.1 and 2.2.

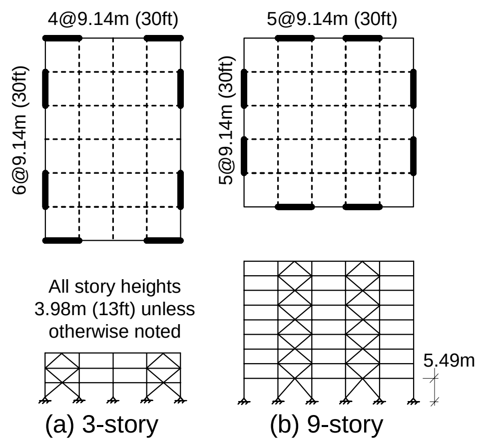

Figure 1 Plan and elevation view of BRBF buildings

Table 1 3-Story BRBF Designs

	Shape (U.S. Designation) or BRBF Brace Area (cm^2)			
Member[a]	BRBF C_s=0.15	BRBF C_s=0.20	BRBF C_s=0.25	BRBF C_s=0.30
BR1[b]	29	39	52	58
BR2	26	32	39	52
BR3	16	19	26	29
C1-C3[c]	W10x39	W10x49	W10x54	W10x68

a. Brace and column sizes are indicated in the table; beam sizes governed by gravity with all beams W16x40
b. BR1 is first story brace, BR2 second story…
c. C1 is first story column, C2 second story…

Table 2 9-Story BRBF Designs

Member	BRBF C_s=0.06	BRBF C_s=0.09	BRBF C_s=0.12	BRBF C_s=0.15
	Shape (U.S. Designation) or BRBF Brace Area (cm^2)[a]			
BR1	42	65	84	100
BR2	35	52	65	84
BR3	35	52	65	84
BR4	32	45	65	77
BR5	29	42	58	71
BR6	26	35	52	65
BR7	19	29	42	52
BR8	16	23	29	35
BR9	9.7	13	16	19
C1-C2	W14x159	W14X233	W14X311	W14X370
C3-C4	W14x 90	W14x132	W14x176	W14x233
C5-C6	W14x61	W14x90	W14x99	W14x120
C7-C9	W14x48	W14x48	W14x48	W14x61

a. See all notes from Table 2.1.

2.2. Gravity Column Design

Critical interior gravity columns for the buildings were designed according to current U.S. provisions (ICC 2006). All gravity loads used for the analysis were taken from Gupta and Krawinkler (1999). Load values are shown in Table 2.3. The column sizes picked for the critical interior columns are shown in Table 2.4. These were the most efficient columns, by weight; the majority do not have seismically compact flanges.

Table 3 Gravity loads for 3- and 9-story buildings

Floor dead load:	96 psf
Roof dead load excluding penthouse:	83 psf
Penthouse dead load:	116 psf
Reduced live load per floor and for roof:	20 psf

Table 4 Critical Interior Column Design

Stories in Frame	Stories	Shape
3	All	W10x49
9	1-2	W14x120
	3-4	W14x90
	5-6	W14x74
	7-9	W14x53

2.3. Obtaining Moments in Lateral-Force-Resisting Columns

Individual frames were modeled as two dimensional systems using the nonlinear dynamic analysis program Ruaumoko (Carr 2004). Standard beam elements with bi-linear flexural-axial hinges at each end were used to represent beams and columns. Buckling-restrained braces were modeled with truss elements that had multi-linear kinematic-type hardening. This approach has been used by Sabelli et al. (2003) and Coy (2007). Experimental data for calibration was taken from Merritt et al. (2003) and Reavely et al. (2004).

Columns at the base of the frames were considered fixed. Beam-column connections were considered rigid when a gusset plate was present, and pinned when not.

Each frame was analyzed under ten earthquakes. See Richards (2008) for details on the ground motions. Moments due to lateral deformation from seismic forces were obtained for the right column within individual frames. Maximum moments for each earthquake were obtained at each floor level for each column. Maximum moments from all ten earthquakes were averaged at each location.

2.4. Obtaining Moments in Gravity Columns

Moments in the lateral-force-resisting columns were used to obtain moments in gravity columns that experience equal lateral deformation, but have different cross-sectional properties. Moments in the lateral-force-resisting columns were normalized by dividing them by the respective moments of inertia of the column at each story. This normalized moment was then multiplied by the moment of inertia of a gravity column to obtain the moment due to lateral deformation.

A derivation of the normalized moment formula may be helpful. Moments can be expressed as a function of the deflection (Δ) of a member, the member's length (L), modulus of elasticity (E), and moment of inertia (I):

$$M = \frac{CEI_x\Delta}{L^2} \tag{2.1}$$

Where M is the moment in the member, kip-in.; E is the modulus of elasticity of the member, ksi; I_x is the moment of inertia of the member with respect to the x-axis, in.^{4}; Δ is the member deformation, in.; L is the unbraced member length, in.^{4}; and C is a constant dependent upon boundary conditions of the member.

Eqn. 2.1 may be divided by I_x to obtain the normalized moment, as shown in Eqn. 2.2.

$$\frac{M}{I_x} = \frac{CE\Delta}{L^2} \tag{2.2}$$

The formula for obtaining moments in the gravity column is:

$$M_{grav} = \frac{M_{lat}}{I_{lat}} \times I_{grav} \tag{2.3}$$

where M_{lat} is the moment in the corresponding lateral-force-resisting column; I_{lat} is the moment of inertia in the lateral-force-resisting column; and I_{grav} is the moment of inertia in the gravity column.

2.5. Column Elastic Moment Capacity

In computing the bending capacity of a wide flange section, C_b accounts for the effect of moment gradient (AISC 2005b). A C_b value of 1 can be used when computing the moment capacity in a column, but this is likely conservative, since double curvature often occurs during seismic loading corresponding to a C_b value of 2 or greater. Computing a column capacity with C_b this large causes the capacity value to exceed the plastic moment capacity value, which is the limiting capacity of a member. For the columns considered (Table 4), any C_b value higher than 1.18 would cause the moment capacity of a column to reach the plastic moment capacity. Therefore, in the interaction equation the plastic moment capacity value was always used for the moment capacity of the column.

2.6. Interaction Equation

An interaction equation was used to analyze the gravity columns accounts for combined bending and axial loads on a member. For all columns, the axial demand was greater than 20% of capacity. Therefore, the governing interaction equation was:

$$\frac{P_u}{\varphi_c P_n} + \frac{8}{9}\left(\frac{M_u}{\varphi_b M_n}\right) < 1.0 \qquad (2.4)$$

where P_u is the axial demand; $\phi_c P_n$ is the axial capacity; M_u is the moment demand; and $\phi_b M_n$ is the moment capacity (AISC 2005b). P_u assumed that 1.2D+0.5L gravity loads were present during the earthquake (ICC 2006).

If the left side of Eqn. 2.3 exceeds a value of 1.0, demand exceeds capacity.

3. RESULTS

3.1. Interaction

Resulting interaction equation values for the 3- and 9-story gravity columns are shown in Figs. 2 and 3. For all 3-story buildings (Fig. 2), the load demand on the gravity columns exceeded column capacity. Capacity was not exceeded for any second or third stories within the 3-story buildings. This is because the same column size was used throughout, so in the second and third stories the columns were oversized for axial loads. For the 9-story buildings, capacity was exceeded for the majority of the columns (Fig. 3). Capacity was exceeded for all columns on the 1st and 7th stories, two columns on the 5th story, and none on the third story.

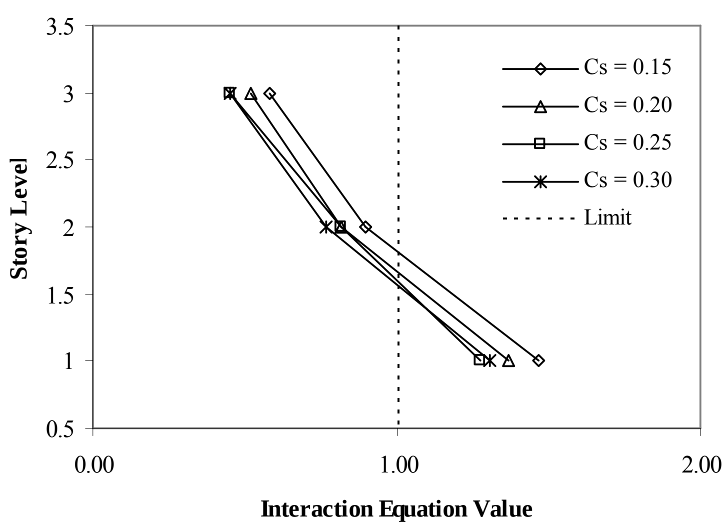

Figure 2 Interaction equation values for 3-story gravity columns

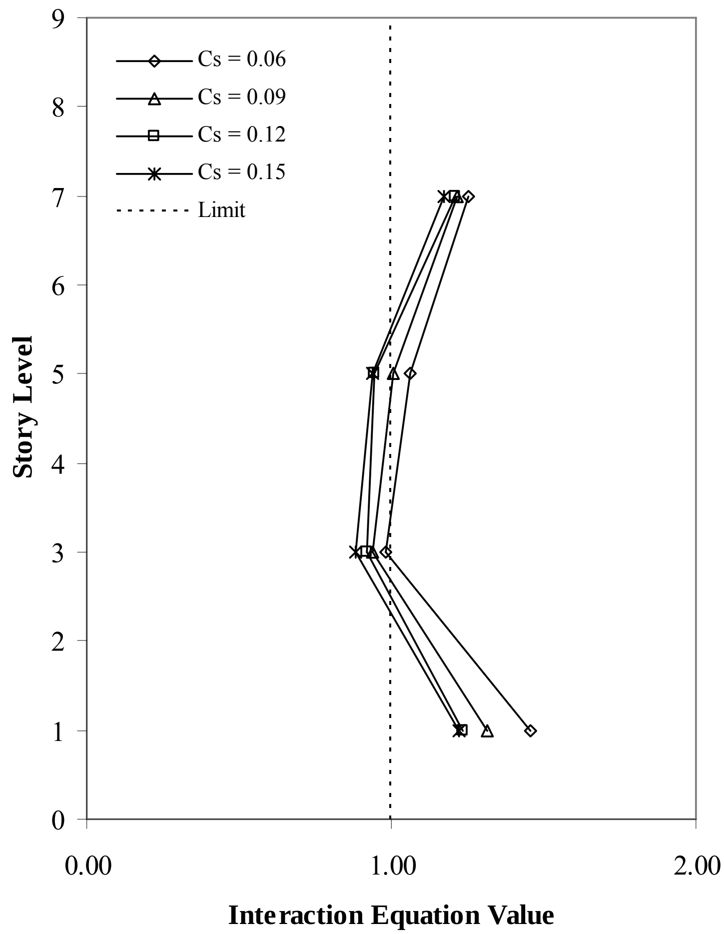

Figure 3 Interaction equation values for 9-story gravity columns

4. CONCLUSION

In this study, moment demands in continuous gravity columns were determined from time history analyses of eight BRBF buildings. When combined with axial loads, these moments are sufficient to cause yielding for the majority of the columns considered. The results of this small study indicate a potential problem, and may justify a more thorough investigation. Currently non-compact columns can be used for gravity columns in high seismic areas (AISC 2006). It may be good practice to select gravity columns with compact, or seismically compact flanges if there is a possibility that flexural hinges will form.

REFERENCES

ASCE. (2005). ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA

American Institute of Steel Construction (AISC). (2005a). Seismic Provisions for Structural Steel Buildings, AISC, Chicago.

American Institute of Steel Construction (AISC). (2005b). Steel Construction Manual, 13th ed. AISC, Chicago.

Carr, A. (2004). Ruaumoko Users Manual, Univ. of Canterbury, Christchurch, New Zealand.

Coy, B. (2007). Experimental Testing of Pinned Beam Connections for Buckling Restrained Braced Frames. MS Thesis, Brigham Young Univ., Provo, Utah.

Gupta, A., and Krawinkler, H. (1999). Prediction of seismic demands for SMRFs with ductile connections and elements. Report SAC/BD-99/06**,** SAC Joint Venture, Sacramento, Calif.

International Code Council (ICC). (2006). International Building Code. International Code Council, Inc., Whittier, Calif.

Merritt, S., Uang, C.-M., and Benzoni, G. (2003). Subassemblage testing of Corebrace buckling-restrained braces. TR-2003/01, Dept. of Struct. Eng., Univ. of California at San Diego, La Jolla, Calif.

Reaveley, L., Okahashi, T., and Fatt, C. (2004). Corebrace Series E Buckling Restrained Brace Test Results, Dept. of Civil and Env. Eng., Univ. of Utah, Salt Lake City, Utah.

Richards, P. (2008). Seismic column demands in ductile braced frames. *J. of Struct. Eng*. in press.

Sabelli, R., Mahin, S.A., and Chang, C. (2003). Seismic demands on steel braced-frame buildings with buckling restrained braces. *Eng. Structures*, 25, 655-666.