Tenth U.S. National Conference on Earthquake Engineering
Frontiers of Earthquake Engineering
July 21-25, 2014
Anchorage, Alaska

EFFECTS OF THE SEISMIC VERTICAL COMPONENT ON STRUCTURAL BEHAVIOR – AN ANALYTICAL STUDY OF CURRENT CODE PRACTICES AND POTENTIAL AREAS OF IMPROVEMENT

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ABSTRACT

The 2013 California Building Code requires that the vertical component of an earthquake be accounted for in almost all building analyses and designs. However, ASCE 41-06 and the draft version of ASCE 41-13 only require consideration of the vertical seismic component for long-span structures, pre-stressed structures, and structures with highly stressed elements under gravity load. Even then, portions of ASCE 41 provide recommendations that conflict with those in the CBC. In many projects, the effect of using vertical time histories is more pronounced and the performance of the structure can be very sensitive to such vertical acceleration demand. This study provides a comparison between code-prescribed pseudo-static vertical seismic forces and the results of a non-linear response-history analysis that incorporates vertical ground motions. Additionally, recommendations are proposed for the necessary level of beam element meshing to suitably capture vertical ground motion effects.

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Dana *et al.* Effects of the Seismic Vertical Component on Structural Behavior - An Analytical Study of Current Code Practices and Potential Areas of Improvement. *Proceedings of the 10th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Anchorage, AK, 2014.

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This study provides a comparison between code-prescribed pseudo-static vertical seismic forces and the results of a non-linear response-history analysis that incorporates vertical ground motions. Additionally, recommendations are proposed for the necessary level of beam element meshing to suitably capture vertical ground motion effects.

Introduction

Two investigative analyses were performed for the purposes of this paper. The first analysis (Analysis 1) was a full building 3D non-linear model subjected to horizontal and vertical ground motions. The results of this analysis will be used to compare component demands from a response-history analysis to those determined using the commonly code-prescribed static vertical force of $0.2S_{DS}$. The second analysis (Analysis 2) was performed on a series of 2D frames subjected to vertical and horizontal ground motions. The results of this analysis will be used to investigate the effect of beam meshing on component demands and joint accelerations.

This study has been undertaken by the Computer Applications Committee of the Structural Engineers Association of Northern California (SEAONC).

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Background

The vertical effects of earthquakes on buildings have traditionally been less of a concern than the horizontal effects. This is primarily due to the misbelief that both the amplitude of the vertical component is smaller than the horizontal component and that the building is stiffer in the vertical direction than in the horizontal direction. Building codes around the world, in concert with this general belief, have given less attention to the effects of vertical shaking in buildings [1]. It was not until the 1988 UBC that design considerations for vertical effects first appeared in a prominent building code [2]. Even then, the requirement was minor: a force of $0.2*W_p$ and only for horizontally cantilevered components. Over the years, the 0.2 coefficient has been carried from code edition to code edition and is still present today in the form of $0.2S_{DS}$ found in ASCE 7-10. Today, even the two most prominent building codes in the United States, ASCE 7-10 and ASCE 41-13 handle the vertical component in very different ways [3, 4]. ASCE 7-10 requires consideration of vertical accelerations in design for all elements, whereas ASCE 41 only requires it to be considered for cantilevered members, pre-stressed members and heavily gravity-loaded members. ASCE 7-10 requires considering the horizontal and vertical components simultaneously while ASCE 41-13 does not. Finally, ASCE 41-13 determines vertical forces either by using a site-specific vertical response spectrum or by scaling the horizontal response spectrum by a factor of 2/3. ASCE 7-10 simply requires using a gravity amplification factor of $0.2S_{DS}$.

Despite the relatively minor ways in which ASCE 7-10 and ASCE 41-13 address the vertical component of ground motion, detailed analytical and experimental studies, as well as a plethora of supporting observations from the damage patterns of the 1989 Loma Prieta, 1994 Northridge, 1995 Kobe, 1999 Chi Chi, 2010 Darfield and 2011 Christchurch (where a vertical PGA of 2.21 was recorded [5]) earthquakes emphasize the significance of vertical seismic effects, especially in near-field conditions. Hence, their effects cannot be ignored in design [5-12]. Recent studies by several researchers conclusively indicate that certain failure modes (including compressive, tensile, shear, and flexural failures) can only be attributed to high seismically-induced vertical forces [11]. In steel structures subject to near-field ground motions, ultra-low cycle fatigue in connections may be attributed to vertical motions. In reinforced concrete structures, vertical effects can reduce column compressive forces, decreasing shear and flexural capacity, and ultimately reducing column ductility.

Due to differences in the inherent damping mechanisms, buildings usually have less damping in the vertical direction than in the horizontal direction. Additionally, vertical ground motions are typically high-frequency motions due to the very high vertical stiffness of buildings, which in turn leads to very high fundamental vertical frequencies (the vertical period of reinforced concrete buildings typically range from 0.05 sec to 0.25 sec) [13]. Furthermore, building vertical frequencies are not greatly influenced by a change in building height or lateral stiffness. This can lead to resonance and significant component demands when the fundamental vertical frequencies of the structures are in the range of the vertical pulses of ground motions [9, 11]. Regardless of evidence to the contrary, most building codes are unconservative when calculating vertical seismic forces. Codes usually calculate vertical demand by the use of a vertical-to-horizontal spectral acceleration (V/H) ratio. Recent studies show that the vertical-to-horizontal (V/H) spectral ratios can reach as high as 1.7 for short periods and 0.7 for long periods. This implies that the commonly used ratio of V/H = 2/3 in engineering applications is far exceeded, at least, in the short-period range [14]. This effect is especially pronounced in near-fields of high-frequency ground motions and in unconsolidated soil environments where the site-specific spectra for vertical ground motions would be an ideal alternative [6, 7, 15, 16].

It is well-known that peak ground vertical accelerations increase with earthquake magnitude and attenuate with distance. In near-fields environments, the P and S waves reach the base of the structure almost simultaneously. The interval between the instances that P and S waves reach the base of the building disperses with the source-to-site distance from the hypocenter of the earthquake [6]. According to Snell's law and refraction of the seismic waves, compression waves (P-waves) are likely to dominate the vertical ground motions in short-periods while the Shear waves (S-waves) dominate the horizontal ground motions in long-periods. Therefore, it is necessary to study the vertical component of ground motions in near-field earthquakes because of the simultaneous occurrence of vertical and horizontal components [12].

Vertical components of ground motions typically have less pronounced effects in the perimeter/corner columns than in the interior columns. Perimeter/corner columns receive more seismic forces from horizontal motions than those at the interior area as the perimeter/corner columns provide resistance to overturning. In addition, the contribution of gravity forces is larger for interior columns since the effect of overturning is negligible at interior columns. For the first vertical mode response of moment frame buildings, effects of the vertical components of ground motion are less in the columns at the lower stories than in the upper stories. This is because the relative change in the pre-existing static axial load is larger in the upper stories. In other words, the sensitivity of the columns to the vertical component of ground motions is more in the upper stories than in the lower stories [17]. A similar pattern can be observed in beams. The effects of the vertical components of ground motion are less in beams that have smaller forces due to horizontal ground motions and smaller tributary areas. Usually, the effects of the vertical component of ground motions are more pronounced in three beam conditions: interior beams, upper story beams and long span beams.

Analysis 1 Methodology

A Nonlinear Dynamic Procedure (NDP) utilizing non-linear response-history analysis was used for this part of the study. A 3D model of a 7-story steel special moment frame structure including a basement was built in PERFORM-3D v5 [18]. Moment frames were modeled using nonlinear elements. Moment frame columns were modeled with two concentrated axial-flexural (PMM) plastic hinges at the ends and an elastic section in between with the end zones outside the flexural hinges at the joints. The moment frame beams were modeled with two concentrated moment-rotation (MR) plastic hinges at the ends and an elastic section in between with the end zones outside the flexural hinges at the joints. The location of the plastic hinge was at the joint or a distance of one-third of the beam depth from the joint depending on the strength of the panel zone, as defined in FEMA 351 [19]. Appropriate reduction factors were applied based on ASCE 41-06 for panel zone demand ratios of less than 0.6 or greater than 0.9 [20].

For this analysis, slabs were modeled as rigid diaphragms. Foundations were not modeled and a fixed base at the basement was assumed. Due to the open floor plan layout, limited number of partitions at most levels and the fact that the damping ratio in steel buildings is inherently less than concrete buildings, a lower damping ratio was used. Rayleigh damping was implemented to yield a damping ratio of approximately 2.5 percent at the fundamental period of the structure in each direction. From the modal analysis, the first mode period of the building was approximately 1.8 seconds with a mass participation ratio of 71 percent. The second mode period was approximately 0.6 seconds with a 10 percent mass participation ratio. The remaining seismic mass participation ratio was associated with the ground floor and basement wall excitation, which were rigid compared to the flexible superstructure.

Two versions of this model were built for this study. The first model did not discretize the beams and the tributary mass was only assigned to the nodes at either end of the beams. This model presents the simplest analytical model that may be used when attempting to capture the effects of the vertical component of seismic events. The other model divided the beams into 8 elements and assigned mass to all of the intermediate nodes and the two end nodes. The second model was used to set a benchmark for comparison of some of the engineering demand parameters between the two models. For each model, two load combinations were analyzed and the results were compared. One load combination included the earthquake vertical effect by applying the CBC approach of 0.2 S_{DS}. The other load combination explicitly applied the vertical acceleration response-history to the model. The comparison of the results from the two approaches will help to evaluate the adequacy of the building code approach for estimating different engineering demand parameters (EDPs).

For the model with meshed beams, 2 of the 7 ground motion records were selected to reduce the analysis time and computational space for this model. EDPs selected for comparison were: column axial force, beam flexural force and joint relative acceleration. Beams, columns and joints at the interior, edge, and corner of the structure, as well as at the building top (roof), mid-height (5th floor), and base (2nd floor) were selected for observation.

Analysis 2 Methodology

Analysis 2 consists of a series of linear response-history analyses performed on three different 2D building frames. The steel moment frame building geometry, loads and member sizes were taken from analytical models developed for the SAC Steel Project [21]. The 3-story, 9-story, and 20-story buildings were modified in two significant ways. First, the subterranean levels were removed and the bases were fixed to simplify the analysis. Secondly, the frames were assumed to have the same tributary mass in the lateral and vertical directions. The original source models utilized only perimeter moment frames. In that design only one half bay of vertical mass was tributary to the moment frames but multiple bays of lateral mass were tributary.

The 2D building frames were created and modeled in SAP2000 (v15.1.0) [22]. Building mass was applied as a line load to the beams. Panel zones were modeled as 50% rigid and all columns were meshed into 4 frame elements between floors. From this point 4 different models, each with different beam meshing, were created for each of the 3 buildings. Beams were meshed into 1, 2, 4, and 8 frame elements for a total of 12 different analysis models. The building modal properties were determined using Ritz Vector analysis with the maximum number of modes either limited to the number of degrees of freedom of the system or 1000. After determining the primary frequency of the buildings, the same ground motions used in Analysis 1 were re-scaled for each of the three structures. Only the fault normal and vertical motions were applied concurrently to the structures. The linear response-history was performed using the same modal properties described above.

The maxima and minima of column axial force, beam flexural force, and joint relative acceleration for each response-history analysis were recorded. Beams, columns, and joints at the interior and exterior of the structure, as well as at the building top, bottom and mid-height were selected for observation.

Ground Motion Records

7 ground motion records were selected and scaled to the design response spectrum at the theoretical site in downtown San Francisco, CA following the guidelines of ASCE 7-05. Scaling was repeated for the four structures under study, three 2D frames and one 3D structure. The same scale factor is applied to the vertical and horizontal components of each record. Table 1 includes the records and relating information for each record.

Table 1. Ground motion information

Event/Station	Magnitude	Distance (km)	Fault Type	PGA (g)
Manjil/Abhar (GM1)	7.37	40	Oblique	0.5
Duzce/Duzce (GM2)	7.14	6.6	Strike-Slip	0.43
Erzikan (GM3)	6.69	4.4	Strike-Slip	0.45
Imperial Valley/Holtville (GM4)	6.53	7.7	Strike-Slip	0.24
Kocaeli/Duzce (GM5)	7.51	15.4	Strike-Slip	0.33
Loma Prieta/LGPC (GM6)	6.93	3.9	R-Oblique	0.71
Landers/Yermo (GM7)	7.28	23.6	Strike-Slip	0.22

Analysis 1 Observations

The un-meshed model is used for the comparison study utilizing 7 ground motion records. The results are presented for EDPs by applying the vertical component of the time history record and the code recommendation of $0.2S_{DS}$ times the dead load. Table 2 shows detailed results for the seven ground motion records used as input motions for this study. As seen in Table 2, corner and edge column compression demands from the CBC code approach are within 10% of those of the vertical time history accelerations. For the interior columns, the code approach underestimates the compression demand by 20% or more in 4 out of 7 records. The compression demand on interior columns is underestimated by an average of 15% to 20% when the code approach is applied in analysis. Column compression demands in the meshed and un-meshed models are similar. The effect of meshing was limited to a 5% change in the column compression force.

For tension load combinations, the code approach does not predict tension demand in interior columns. However, the response-history analyses with vertical time history records show tension in interior columns for 3 out of 7 records. Considering that column splices in many existing steel structures only include small partial joint penetration (PJP) welds with weld material lacking minimum toughness requirements for seismic demands, these welds may be susceptible to premature fracture particularly under the net tension demands. Considering that column splices are essential to the integrity of the columns under gravity and seismic loads, predicting and monitoring their axial and flexural demands are critical in evaluating columns as well as the overall building performance under seismic events.

Beam flexural demands are compared, at the face of the column, between the code and the vertical time history approaches. The results show that the negative flexural demands are comparable between the two approaches. Even though the positive flexural demand results are close between the two approaches for the perimeter beams, the demands are underestimated in interior beams, especially at the roof level. Comparing the beam results, both the positive and the negative flexural demands are underestimated in the unmeshed model for the interior beams at lower floors. For the meshed model, positive flexural demands are reported at beam mid-span. Although one time history record shows that the demands are underestimated at the roof level, the other record shows otherwise. Therefore, no final conclusion is reported for this EDP.

Floor vertical accelerations are compared at the column locations between the two approaches. The results show that the code approach is significantly un-conservative in all locations and under all time history records. Comparing the vertical acceleration results shows that even though the meshed model predicts lower demands in the vertical time history case, it predicts smaller demands in the code approach case. In addition, the results from the meshed case confirm that not only is the vertical acceleration at the column locations much higher in the vertical time history case—compared to the code approach—but it is also higher in the beam mid-span.

As an overall building demand parameter, story drift was compared between the two approaches and it was found that the results are within an acceptable range and that story drifts are insensitive to the beam mesh size.

Analysis 2 Observations

Figure 1 shows the range and average of absolute maximum vertical accelerations at 6 pre-selected column joint locations (interior and exterior columns at the roof, mid-height, and 2nd story). All of the accelerations have been normalized by the accelerations determined by the 8 element beam mesh model.

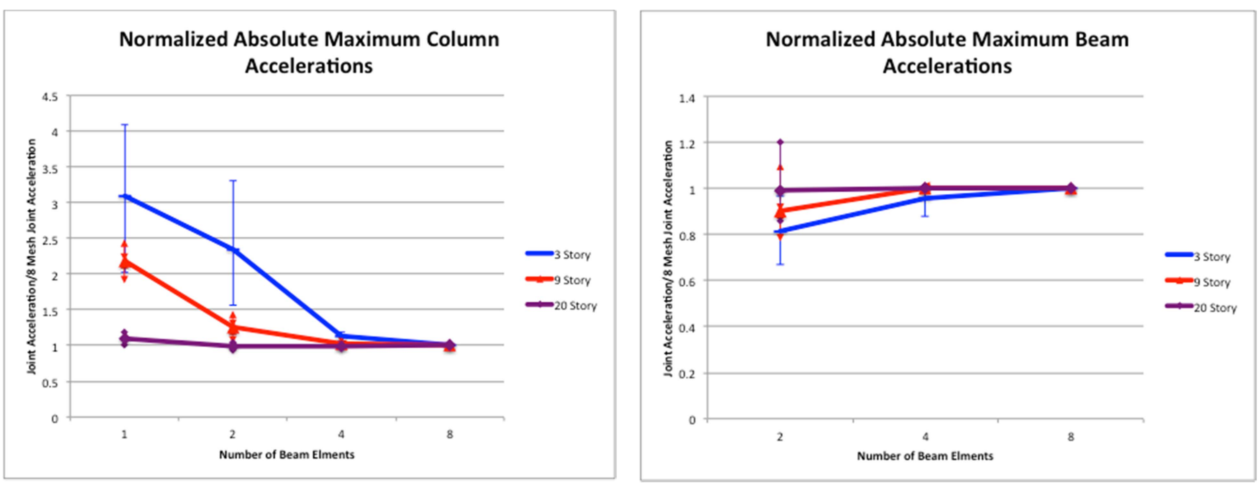


Figure 1. Average and range of normalized maximum joint accelerations for analysis 2.

A number of trends are apparent from column accelerations presented in Figure 1. First, the range of column accelerations reduces dramatically with increasing building height. The 3-story 1 beam element model shows accelerations that vary from 200-400% of the same 8 beam element model. The 20-story model shows only 1-20% variability for the same mesh. Secondly, the average and range of all 3 buildings reduces dramatically at the 4 element mesh level. The maximum divergence from the mean accelerations of any of the 3 buildings is only 12% for

Table 2. Analysis 1 results

- A_C Accelerations at columns
- B_{EM} Beam end moments
- C_A Column axial force
- LC_1 $1.2D+0.5L+a_H(t)+a_v(t)$; Compression load case
- LC_2 $0.9D+a_H(t)+a_v(t)$; Tension load case
- LC_3 $1.41D+0.5L+a_H(t)$; Compression load case
- LC_4 $0.69D+a_H(t)$; Tension load case
- 2^{nd} Base (Columns); 2nd floor (Beams)
- 5^{th} Between 4^{th} and 5^{th} stories (Columns), at 5^{th} floor (Beams)
- R Between 7th and 8th stories (Columns); at Roof (Beams)
- T Column in tension under $a_v(t)$

			GM_1			GM_2		
			CORNER	EDGE	INTERIOR	CORNER	EDGE	INTERIOR
C_A	LC_3/LC_1	R	1.09	1.10	1.03	1.08	1.03	0.74
		5th	1.07	1.09	1.03	1.06	1.05	0.75
		2nd	1.07	1.09	1.05	1.05	1.07	0.81
	LC_4/LC_2	R	0.23	0.45	0.93	0.06	0.66	177.81
		5th	10.84	0.67	0.94	1.25	1.00	10.53
		2nd	2.20	0.74	0.89	1.19	0.56	3.22
B_{EM}	$LC_3(+)/LC_1(+)$	R	0.98	-	0.74	1.00	-	0.36
		5th	0.99	-	0.75	0.97	-	0.84
		2nd	0.99	-	0.95	0.99	-	0.94
	$LC_3(-)/LC_1(-)$	R	1.01	-	1.03	1.01	-	1.05
		5th	1.03	-	1.02	1.03	-	1.04
		2nd	1.01	-	1.03	1.01	-	1.03
A_C	LC_3/LC_1	R	0.27	0.14	0.18	0.04	0.02	0.01
		5th	0.24	0.11	0.13	0.05	0.02	0.02
		2nd	0.30	0.28	0.34	0.14	0.03	0.02

			GM_3			GM_4			GM_5			GM_6			GM_7		
			CORNER	EDGE	INTERIOR	CORNER	EDGE	INTERIOR	CORNER	EDGE	INTERIOR	CORNER	EDGE	INTERIOR	CORNER	EDGE	INTERIOR
C_A	LC_3/LC_1	R	1.08	1.09	0.95	1.00	0.97	0.60	1.07	1.08	0.76	1.00	1.01	0.68	1.06	1.03	0.90
		5th	1.07	1.08	0.95	1.08	0.99	0.63	1.06	1.07	0.77	1.02	1.04	0.71	1.05	1.02	0.91
		2nd	1.07	1.08	0.98	1.10	1.04	0.68	1.06	1.09	0.83	1.04	1.04	0.76	1.03	1.02	0.94
	LC_4/LC_2	R	1.08	0.58	1.15	0.93	1.25	T	4.35	2.73	T	0.44	T	T	T	1.24	1.98
		5th	1.95	0.52	1.08	0.66	0.70	T	21.47	3.00	T	0.78	0.13	T	1.11	0.19	1.44
		2nd	1.37	0.56	0.99	0.72	0.56	T	T	2.73	3.75	0.83	0.27	T	1.08	1.92	1.28
B_{EM}	$LC_3(+)/LC_1(+)$	R	0.97	-	0.63	0.98	-	0.61	0.99	-	0.79	0.94	-	0.85	0.90	-	0.79
		5th	1.00	-	0.64	0.99	-	0.87	0.98	-	0.84	0.99	-	0.90	0.98	-	0.47
		2nd	0.99	-	0.94	0.99	-	0.96	0.99	-	0.96	0.93	-	0.95	0.97	-	0.96
	$LC_3(-)/LC_1(-)$	R	1.01	-	1.05	1.01	-	1.03	1.03	-	1.04	1.02	-	1.03	1.04	-	1.04
		5th	1.02	-	1.05	1.02	-	1.05	1.03	-	1.04	1.01	-	1.03	1.01	-	1.05
		2nd	1.01	-	1.02	1.01	-	1.02	1.01	-	1.03	1.01	-	1.02	1.01	-	1.02
A_C	LC_3/LC_1	R	0.12	0.04	0.05	0.01	0.03	0.01	0.08	0.04	0.02	0.04	0.02	0.01	0.04	0.04	0.01
		5th	0.15	0.04	0.06	0.01	0.03	0.00	0.08	0.03	0.02	0.04	0.02	0.01	0.04	0.03	0.01
		2nd	0.41	0.13	0.21	0.09	0.19	0.01	0.27	0.20	0.07	0.06	0.02	0.01	0.10	0.04	0.03

these models. Overall, accelerations at column locations tend to be overestimated at lower meshing levels. The 20 story building shows underestimated column accelerations at the 2 element mesh, but otherwise accelerations are consistently overestimated.

Beam mid-span accelerations are also plotted in Figure 1. Accelerations were measured at 6 beam mid-points (interior and exterior beams at the roof, mid-height, and 2nd story). No 1 element mesh results are displayed since beam mid-point accelerations cannot be determined in that configuration. Like the column accelerations, variability reduces with building height and convergence seems to occur at the 4 mesh model. However, unlike the column accelerations, beam mid-point accelerations are mostly underestimated at lower meshing levels.

In general, the maximum column and beam vertical accelerations increased with story height. Typically, interior columns had higher maximum vertical acceleration than exterior columns. Similar to the columns, the interior beams have a slightly higher relative vertical acceleration.

In additional to vertical accelerations, beam moments and column axial forces were also studied. All forces discussed were determined by taking the average of the absolute maximums of each of the 7 ground motions. For the purposes of this analysis, no differentiation was made between positive/negative beam moment and compression/tension column force. The unfactored member forces are solely from the scaled simultaneously applied vertical and horizontal ground motions.

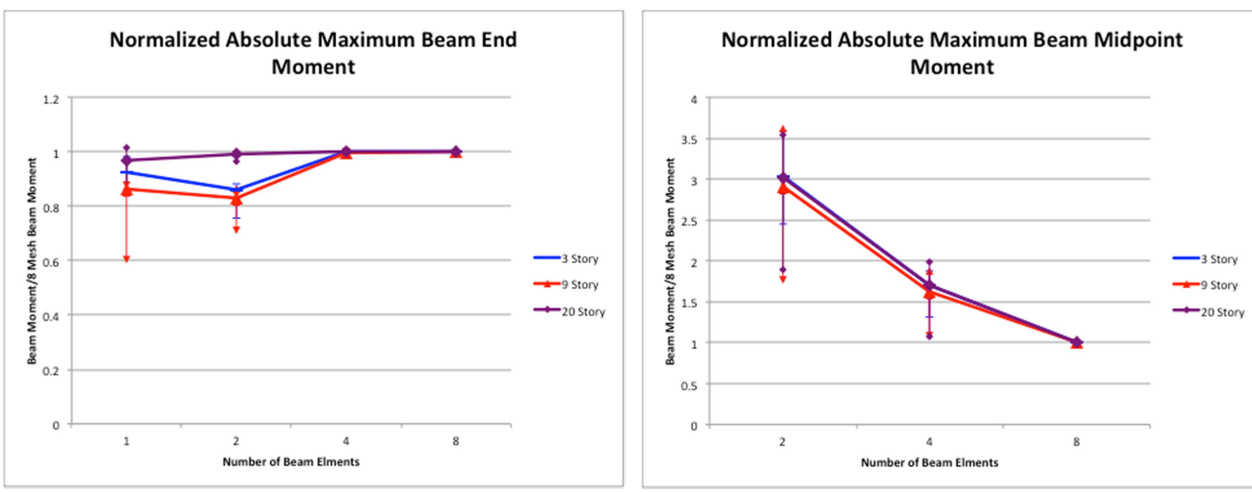
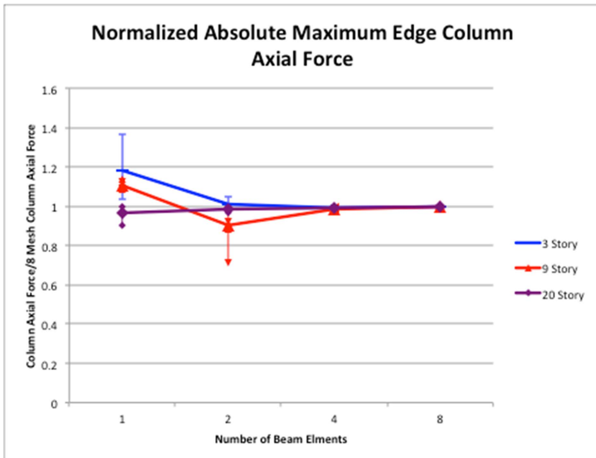


Figure 2. Average and range of normalized maximum beam moment demands for analysis 2.

Figure 2 shows very different results for the beam end and midpoint moment demands. Beam end moments are not particularly sensitive to the mesh level but midpoint demands are. Moment demands at both beam locations are not particularly sensitive to the building height. Convergence is seen at the 4 element mesh level for end moments. Midspan moments do not converge as thoroughly at the same level of meshing.

Column axial force demands are shown in Figure 3. Overall, the axial demands are not particularly sensitive to building height and have nearly complete convergence at the 4 element mesh level.



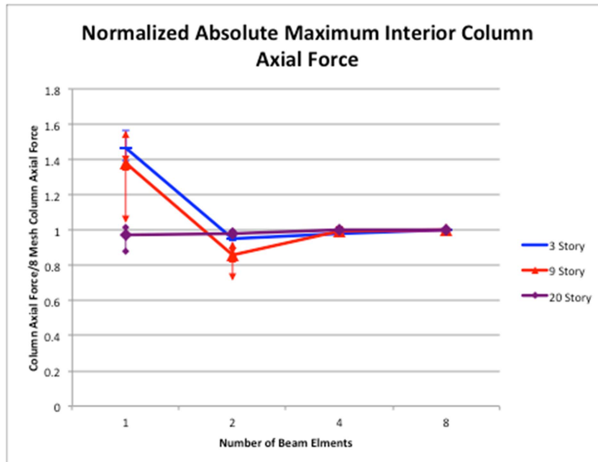


Figure 3. Average and range of normalized maximum column axial demands for nalysis 2.

Conclusion and Further Research

One significant observation from Analysis 1 was that the code approach did not capture the net tension demands at the interior columns. Considering the non-ductile behavior of low-toughness weld material at pre-Northridge groove welds, this issue can be of a major concern for the evaluation of existing buildings at life-safety and collapse-prevention performance objectives. It was found that the code approach underestimates the interior column compression demands by as much as 40% with an average of almost 20%. For interior beam demands, the positive moments at the face of the columns were lower for the code approach compared to the explicit application of the vertical accelerations by a maximum of about 65% with an average of roughly 30%. The magnitude of difference was higher at the upper story beams compared to the beams at the lower floors. The analyses confirm that the change in column axial and beam flexural demands between the code and explicit vertical acceleration approaches is more pronounced in interior building elements than edge and corners. The vertical accelerations in beams and columns were found to be substantially affected by the inclusion of vertical acceleration time histories. This can cause major damage to acceleration-sensitive non-structural components in the buildings, to the equipment installed at floors, and to the anchorage of the equipment.

The results of Analysis 2 showed that for most of the parameters that were tracked, a 4 element beam mesh provided the same results as an 8 element beam mesh. Overall, the taller buildings were less sensitive to the level of beam mesh level and would often converge at the 2 element mesh level. Our recommendation is that using a 4 element beam mesh should sufficiently capture overall behavior due to the vertical time history accelerations.

We recommend conducting further research on this topic. Both Analysis 1 and 2 were steel moment frame systems and both used the same set of ground motions scaled to a particular site. Analytical research should be expanded to buildings with different lateral systems, building materials, building orientations and different site specific ground motions.

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