IMPLICATIONS OF THE VERTICAL COMPONENT OF GROUND MOTION ON BUILDING RESPONSE

A. Tzortzis¹, N. Paul², and I. Almufti³

ABSTRACT

Recent earthquakes have demonstrated that the vertical component of ground motion can be damaging to building structural and nonstructural components. Recorded vertical accelerations from these recent earthquakes significantly exceed the vertical spectral demands used for seismic design prescribed in building codes in the United States. The focus of this paper is to determine, through non-linear time history analyses, the expected behavior of a code-compliant 3-story steel moment frame structure under vertical accelerations consistent with the design level earthquake intensity. This study differs from prior research because all components, including gravity elements, are explicitly modeled to capture potential non-linear behavior. Prior studies compared elastic demand-capacity ratios for some structural elements, which may not provide a meaningful measure of damage due to the high frequency nature of vertical response. Insights from the analysis are used to recommend future research needs.

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Recent earthquakes have demonstrated that the vertical component of ground motion can be damaging to building structural and nonstructural components. Recorded vertical accelerations from these recent earthquakes significantly exceed the vertical spectral demands used for seismic design prescribed in building codes in the United States. The focus of this paper is to determine, through non-linear time history analyses, the expected behavior of a code-compliant 3-story steel moment frame structure under vertical accelerations consistent with the design level earthquake intensity. This study differs from prior research because all components, including gravity elements, are explicitly modeled to capture potential non-linear behavior. Prior studies compared elastic demand-capacity ratios for some structural elements, which may not provide a meaningful measure of damage due to the high frequency nature of vertical response. Insights from the analysis are used to recommend future research needs.

Introduction

In the United States, current code-based building design practice extensively considers the horizontal component of ground motion but often neglects the effects of the vertical component, even though high vertical accelerations have been measured in recent earthquakes. Vertical ground motions are generally considered in the seismic design of hazardous facilities, but this practice is not required in the design of all new buildings. For typical buildings, the building codes in the United States account for vertical seismic demands by augmenting gravity loads on the structure, based on a proportion of the horizontal spectral acceleration at short period (S_ds). The magnitude of this design vertical acceleration is 0.2S_ds, but this value has unclear scientific backing. The practice prescribed in the building code has been attributed to the common misunderstanding that the vertical component of ground motion is always smaller than the horizontal component. However, comprehensive research and case studies from recent earthquakes have shown that this

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is not always the case, and have continued to highlight the importance of vertical ground motion considerations in seismic design [1, 2]. Additionally, the introduction of the vertical earthquake effect into the building codes occurred partially as a response to the reduction in prescribed design dead loads that occurred simultaneously [1, 3].

The code approach may have significant limitations: 1) it is not clear what the intended code performance objectives of the structure are in the vertical direction, and 2) it is not clear how code-designed structures would realistically behave under vertical motions consistent with the design level earthquake intensity. This study is the first in a series of planned studies aimed at determining the circumstances under which vertical ground motions will have significant effects on both structural and nonstructural performance, and in which cases the vertical component is most critical to be considered in structural design.

Characteristics of the Vertical Component of Ground Motion

Seismic compression waves (P-waves) correspond to vertical ground shaking, and seismic shear waves (S-waves) are responsible for horizontal components of ground motion [4]. These two components of ground motion are different not only in directionality but also in frequency content. Vertical components of ground motion are much higher frequency than the corresponding horizontal components. These vertical accelerations are highest in the short period range, and typically occur over a small range of high frequencies [5]. The most significant effects of vertical ground accelerations occur with high-intensity earthquakes with high magnitude and in near-fault environments. In these cases, the vertical accelerations can greatly exceed horizontal accelerations [5]. Due to the differences in the magnitudes and the frequencies corresponding to the peak vertical and horizontal spectral accelerations, a linear scaling of the vertical acceleration from the horizontal design spectrum, as is practiced in current codes, will likely be an inappropriate estimation of the demands on the structure.

Significance & Case Studies

Both structural and non-structural components in buildings have short vertical periods, typically corresponding to the high frequency content of the vertical ground accelerations. Furthermore, buildings have less damping and energy dissipation in the vertical direction [5]. High vertical accelerations may be of concern for all buildings, as vertical frequencies and damping are relatively independent of building height and lateral stiffness.

Recorded motions show that there can be substantial vertical shaking that can significantly exceed the horizontal, and this has yet to be incorporated into code-based building designs. There are several case studies from past earthquakes that have shown the pronounced effects of vertical accelerations. Additionally, the effects of peak vertical ground accelerations may be even more detrimental to structural and nonstructural performance when combined simultaneously with peak horizontal accelerations. This is of concern especially at near-fault sites, where the coincident arrival of P-waves and S-waves occurs [6].

The damaging effects of earthquakes with high vertical accelerations have been the focus of several studies. Potential failures include compressive and tensile failure of columns and walls, shear failure of columns and beams, and shear connection failure. In the 2011 Christchurch earthquakes, several observed failures could have been attributed to high vertical accelerations, such as steel connection failure and shear failure of concrete columns due to a reduced shear capacity when columns experience net tension [6]. Similarly, the 1995 EERI report regarding the 1994 Northridge
Earthquake “highlighted cases of brittle failure induced by direct compression or by reduction in shear strength and ductility due to variation in axial forces arising from vertical motion” [4]. Recent research has further confirmed the potential detrimental effects of high vertical ground shaking [1]. All studies have shown that the effects and consequences of vertical components of ground motion vary with building system, materiality, building loads, construction type and date of construction of the building [1, 7].

**Description of Study Building**

The building used in this study is a fictitious 3-story steel SMRF office building, designed for Los Angeles by Forell/Elsesser Engineers Inc., that has been the subject of several research studies [8, 9, 10]. The lateral force resisting system consists of perimeter moment frames in both directions and inner moment frames in the transverse direction of the building, elevations of which are shown in Figure 1.

![Figure 1](image_url)

Figure 1. Elevations of perimeter and interior steel special moment-resisting frames (from Morgan 2008).
The building was designed per ASCE7-05, IBC 2006, and AISC 341-05 using the equivalent lateral force method and an importance factor of 1.0. Mapped spectral accelerations for its site and reference shear wave velocity (site class D) were $S_s = 2.2$ g and $S_i = 0.74$ g, which corresponds to design site-specific spectral accelerations in the short period range, $S_{DS}$, of nearly 1.5 g. The design was drift-controlled with a design drift limit of 2.5%.

Although the building was designed as described above, the analysis undertaken was for a site in Cupertino with an $S_{DS}$ of approximately 1.2 g. This is a lower design acceleration than the Los Angeles site for which the fictitious building was originally designed, so the structure is slightly overdesigned for lateral forces. The Cupertino site was 12 km from the San Andreas fault and 5.5 km from the Monte Vista-Shannon fault.
Figure 3. Comparison of site-specific response spectra used in analysis versus ASCE7-05 spectra for the building design.

The gravity system is a conventional structural steel system with 3 ¼ in. thick lightweight concrete fill over 2 in. deep composite metal deck spanning to steel wide-flange beams and girders. Columns grids are at 30 ft. in both directions. Intermediate framing is provided at 10 ft. spacing. The beams are W18x35’s and W16x31’s at the typical and roof levels, respectively. The typical girder section sizes are W24x55 at typical levels and W21x50 at the roof level. Collector beams are heavier than the typical beam sections, as shown on plan. The gravity columns have been re-designed for the loading assumptions used in this analysis and are all W12x53 columns. The bottom story height is 17 ft., while the typical upper levels have a 15 ft. story height.

**Description of Analysis Model**

The analysis is performed with a 3D nonlinear response history model using LS-DYNA. All structural components, including gravity beams and columns, are modeled with nonlinear properties in order to capture post-yield behavior. The post-flexural buckling behavior of the beams is not captured in this model, and thus a plastic hinge is modeled at mid-span of the beams to indicate when the flexural capacity of the beams have been exceeded. Fiber elements are used for the columns with 9 fibers across the section, modeled as bilinear (linear elastic, perfectly plastic) with isotropic hardening. Each column is discretized into 8 elements per story, and modeled with an imperfection of L/500 to capture axial buckling. Connections were not explicitly modeled, but were checked in post-processing for adequate capacity against member demands. A constant damping ratio of 2% was assumed in all directions at all relevant frequencies. The base of the model was fixed with ground motions applied as prescribed accelerations on the base nodes. The first and second modal periods of the building are 0.77 seconds and 0.74 seconds for translation in the longitudinal (X) and transverse (Y) directions, respectively. The first vertical mode occurs at a period of 0.16 seconds. The spectral accelerations expected based on these periods are listed below. The ground motions used in the analysis are for the maximum considered earthquake (MCE).
<table>
<thead>
<tr>
<th>Horizontal X Acceleration (g)</th>
<th>Horizontal Y Acceleration (g)</th>
<th>Vertical Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>1.2</td>
<td>1.16</td>
</tr>
</tbody>
</table>

Figure 4. Site-specific response spectra used in analysis

The vertical uniform hazard spectra (UHS) was created using vertical to horizontal spectrum ratios (V/H) from Bozorgnia and Campbell 2004 and Gulerce and Abrahamson 2011. Per probabilistic seismic hazard assessment, San Andreas contributed 75% and Monte Vista-Shannon 25% to the hazard at short periods (e.g. 0.5 seconds). Thus, seven seed ground motions were selected with fault styles in proportion accordingly (i.e. five strike-slip and two reverse). These seed ground motions spectrally matched over a period range of 0.1 to 3.0 seconds.

**Analysis Results**

The metrics tracked in the analysis were interstory drifts, floor accelerations, axial demands in the columns, permanent column deformations and shear and flexural demands in the beams for the Cupertino ground motions. These results were compared for the two analyses with bi-directional motion (horizontal components only) and tri-direction motion.

**Interstory Drifts**

The effects of including the vertical ground motions on interstory drift ratios are not significant. For both sites, the average and maximum drift ratios increased by no more than 5% for tri-
directional motion. These findings are consistent with [5], and these results from this study are presented in Table 1.

Table 1. Interstory drifts.

<table>
<thead>
<tr>
<th></th>
<th>Average</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bi-directional</td>
<td>1.8%</td>
<td>2.2%</td>
</tr>
<tr>
<td>Tri-directional</td>
<td>1.9%</td>
<td>2.3%</td>
</tr>
<tr>
<td>Increase</td>
<td>5%</td>
<td>5%</td>
</tr>
</tbody>
</table>

**Vertical Floor Accelerations**

Vertical floor accelerations experience a noteworthy increase for the tri-directional motions at both sites. The floor slab accelerations are amplified significantly up the height of the building, measuring up to 3.0 g at the roof level as shown in Table 2. The maximum and average vertical floor accelerations for GM1 are plotted below. Typically, the average accelerations are about 80-90% of the maximum accelerations. The vertical accelerations directly at the column nodes were also recorded in the analysis and were typically about 20-30% of the slab acceleration at each level.

Table 2. Vertical floor accelerations.

<table>
<thead>
<tr>
<th></th>
<th>Average Acceleration (g)</th>
<th>Maximum Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RF</td>
<td>L3</td>
</tr>
<tr>
<td>Vertical</td>
<td>2.4</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Figure 5. Maximum (blue) and average (red) acceleration time history of the roof for GM1.

These amplified vertical accelerations can have severe consequences for both structural and non-structural components. The current code requirement prescribes a vertical design acceleration of $0.2S_{DS}$, or approximately 0.24g for the considered site, which is about 10% of the roof average vertical accelerations experienced in this analysis. Furthermore, the code does not account for the amplification of vertical accelerations with height up the building. For non-structural components in particular, both architectural components and mechanical equipment will experience higher accelerations at higher stories, especially at mid-bays.
Column Axial Demands
The effect of including the vertical component of ground motion varies for moment frame columns and for gravity columns, though amplification in axial demands was experienced for both. The moment frame columns are W14x211 at the ground level with a height of 17 ft. and an expected capacity is 2549 kips. All gravity columns are W12x53, with a first story height of 17 ft. The expected capacity of the W12x53 columns is 392 kips. These capacities are obtained using an effective length factor, K, of approximately 1.2 per AISC 360-10 Table C-A-7.1 and Figure C-A-7.2 for columns not prohibited from side sway.
The amplification in axial force is more noteworthy for the gravity columns than for the moment frame columns as shown in Table 3. In comparison, a study of reinforced concrete columns modeled for the 2009 L’Aquila Italy earthquake experienced an amplification of compressive loads between 59% and 174% [2]. These findings are consistent with the findings from nonlinear studies of RC frame buildings in [1], in which the columns experienced an average increase of 60% at incipient collapse when damaging vertical ground shaking are considered in addition to the horizontal components of ground motion [1].

<table>
<thead>
<tr>
<th></th>
<th>Moment Frame Columns</th>
<th>Gravity Columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>GM1</td>
<td>574</td>
<td>675</td>
</tr>
<tr>
<td>GM2</td>
<td>532</td>
<td>693</td>
</tr>
<tr>
<td>GM3</td>
<td>557</td>
<td>568</td>
</tr>
<tr>
<td>GM4</td>
<td>539</td>
<td>565</td>
</tr>
<tr>
<td>GM5</td>
<td>546</td>
<td>606</td>
</tr>
<tr>
<td>GM6</td>
<td>564</td>
<td>564</td>
</tr>
<tr>
<td>GM7</td>
<td>543</td>
<td>658</td>
</tr>
<tr>
<td>Avg.</td>
<td>551</td>
<td>618</td>
</tr>
</tbody>
</table>

The expected axial capacity of the gravity columns is exceeded for all ground motions when considering tri-directional motion. Similar axial demands are seen when the building is subjected to a linear elastic response spectrum analysis for both the gravity and lateral columns as with the nonlinear time history analysis.
In addition to an increased compressive force, some of the columns briefly experience net tensile forces at the roof level in most of the ground motions run. This net tension is concentrated only at the roof level where the gravity axial demands on the columns are the lowest, and does not exceed the tensile capacity of the columns.

Column Residual Deformations
Column buckling was tracked through residual deformations, both horizontally and vertically at the gravity column nodes. The vertical deformation was measured at the top of the column at each story, while the horizontal displacements were taken at mid-height of the column and subtracted from the average residual displacement at the top and bottom of the story.
The residual displacements were minimal both vertically and horizontally, thus indicating that buckling was not observed. A separate buckling analysis was performed to confirm that buckling was capable with the elements as modeled in this analysis. The maximum residual displacements
were all less than \( \frac{1}{4} \) in. though a general increase of residual displacements both vertically and laterally was observed.

Furthermore, the residual deformations increased down the building, with the largest horizontal and vertical displacements at columns in the first story. This can be seen for the vertical behavior in Figure 6 below. The horizontal deformations followed similar patterns.

![Residual Vertical Displacements](image)

**Figure 6.** Worst case vertical deformation at each story of a gravity column.

![Column Axial Force Time History](image)

**Figure 7.** Axial force time history of a gravity column.

Though the gravity column demands exceed their respective capacities and residual deformation is observed, buckling of the columns was not initiated in the analysis model. Since the vertical accelerations occur at such high frequencies, it is possible that there is not enough time for the initiation of buckling to occur before the direction of the acceleration reverses. However, buckling behavior was exhibited when the vertical component of motion was arbitrarily increased in amplitude and decreased in frequency. Further study would be required to investigate when buckling might occur in reality.

**Beam Demands**

The effect of tri-directional ground motion on the flexural demands of the gravity beams is also significant. The flexural demands at mid-spans exceeded their capacities in both positive bending (sagging) and negative bending (hogging) at several locations throughout the building on the two upper stories. Since post-buckling flexural behavior is not a well-understood phenomenon, the
demands were tracked by modeling a plastic hinge at mid-span of the beam that is initiated if the capacity of the beam is exceeded. The locations where the beam flexural capacity was exceeded are depicted in Figure 8.

![Figure 8](image)

**Figure 8.** Plastic hinge rotation of the gravity beams at the roof (left) and level 3 (right).

The gravity beams are all W16x31 sections at the roof and W18x35 sections at the two lower levels. The demands in several of the roof beams in the end bays of the building exceeded their capacities, while at level 3, only few of the beams exceed their capacities. None of the beams at level 2 exceeded their capacities. Typically, the beams with the highest plastic hinge rotations occur at the end bays that do not have stiffer moment frame beams on both sides. The beam shears were also investigated initially. While the shear demand on the beams and shear connections typically did not exceed their shear capacities, the demands at the connections were close and in reality may exceed typical connection capacities in some cases. The consequences of this behavior would depend on the governing connection and limit state of the connection, and thus this topic is left for future studies.

**Conclusions**

The results of this analysis have shown that vertical ground motions can have significant consequences on both structural and non-structural performance. In these analyses, the structural elements all experienced an amplification of demands with the inclusion of the vertical component of ground motion. Axial forces in the gravity columns exceeded their capacity, but it does not appear that this is of significant concern in the scenarios considered due to the high frequency of the vertical component of ground motion. In other words, by the time the column initiates buckling, the loading is reversed. The time history analysis has shown that though the demands may be high, they likely do not produce a large ductility demand required for design. Similar axial demands are experienced by the gravity columns with a linear-elastic response spectrum analysis. However, this study has shown that in order to assess realistic demands and their impact on design, a non-linear response analysis must be performed to capture the expected performance. In near-fault situations, the amplitude of vertical motions at lower frequencies may be high enough to exceed the column axial capacity and sustain the demand long enough to cause buckling. This remains a topic for further study.

The tensile forces that the columns experienced in this analysis were not of concern, since the tensile capacities are significantly higher than the capacities of the columns in compression. However, similar behavior may become problematic in concrete buildings, where the tensile
capacity may be less and where tensile forces reduce the shear capacity.
The effects of vertical motion are prominent at the upper stories, with the highest vertical accelerations and beam demands occurring at the roof. The vertical accelerations exceed the gravity loads in the roof beams, and thus the beams experience negative bending (hogging). Since the bottom flange of gravity beams are typically unbraced, the failure mode for beams exceeding their upward flexural capacity would be lateral-torsional buckling. The consequences of this behavior are unclear because post-buckling capacities are not well understood. If this behavior is observed with concrete gravity beams, significant damage would likely be expected as typically the tensile reinforcement at the mid-span of concrete beams is only provided at the bottom of the section for positive flexure.

Peak floor accelerations up to 3.0 g are observed, and are amplified significantly up the height of the building. Accelarations of this magnitude are significantly larger than the code design value and are thus likely to damage structural and non-structural components. Further research is required to validate these conclusions in the context of other buildings and hazard levels.

**Further Research**

The observations and conclusions derived from this report are for this particular building typology at a specific hazard level. Further study is required to fully understand the implications of vertical motions, including the potentially beneficial effects of soil-structure interaction. As previously mentioned, this study was the first in a series of planned studies, consisting of varying building materials, building heights and hazard levels in order to comprehensively evaluate the effects of vertical accelerations on building performance.

**References**


