CONSIDERATIONS OF VERTICAL ACCELERATION ON STRUCTURAL RESPONSE

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SUMMARY

Building codes generally characterize the vertical acceleration of an earthquake as being equal to two-thirds of the horizontal acceleration. Studies have found that the ratio of the horizontal acceleration to the vertical acceleration varies due to period and also due to distance from the fault. Close to the fault, the ratio of vertical to horizontal acceleration can exceed 1.0. Further from the fault, the ratio is less than 1.0. The peak of the vertical spectra is at a higher frequency (lower period) than the horizontal spectra.

Three buildings have been subjected to horizontal and vertical excitation using a linear response spectra analysis. The structures were modelled using a general-purpose finite element analysis program and accounted for vertical excitation by discretizing the floor mass and including all significant structural elements. Four response spectrum combinations were applied to each building: one without vertical component, and three with various vertical components to represent the variations in the ratio of vertical to horizontal acceleration.

Modelling buildings to accurately capture the vertical response modes involves consideration of the likely predominant vertical response modes. Typically, discretely modelling every element in the building and distributing the mass to each of theses elements is time consuming and unnecessary. Parametric studies can be useful in determining the precision with which the modelling needs to be done.

The results of the analyses for each building were compared for the different vertical response spectra to evaluate the effects of distance from the epicenter and structural system on the significance of the vertical response. For most typical structures, the effects of vertical acceleration are small compared to the horizontal acceleration effects. Some elements in a building may experience a significant vertical force due to the vertical acceleration, but this is generally on the order of magnitude of the dead load. Buildings in the near fault area will experience a more significant contribution of the effects of vertical acceleration.

INTRODUCTION

Earthquakes generate vibrational motion in both horizontal and vertical directions. For the most part, engineers are generally concerned with the effects of the horizontal motion on buildings. Building codes for seismic design specify design forces on the building in the horizontal direction. For certain types of components, such as cantilevers and prestressed components, some building codes also specify that vertical earthquake forces be applied [ICBO 1994]. When conducting dynamic analyses, the building codes specify that the vertical response spectrum should be scaled to two-thirds of the horizontal response spectrum.

The strong motion records that were produced following the 1994 Northridge earthquake, identified large horizontal and vertical acceleration values, some of which exceeded 1g. The knowledge of the high vetical

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acceleration values and the failure modes of some structures led some engineers to speculate that some of the observed damage was attributed to the high levels of vertical acceleration [Papazoglou and Elnashai 1996]. The ground motion records from Northridge have shown that except for a few isolated cases, the peak vertical accelerations were typically about two-thirds of the horizontal accelerations [Shakal et al 1996]. Some of the damage to freeway structures though initially to have been caused by vertical accleration was later shown to have been due to other causes, the perception of the significance of vertical accelerations has remained.

Studies have been conducted to evaluate the relative magnitude of vertical accelerations with respect to the horizontal acceleration. Following the 1971 San Fernando earthquake, the significance of the vertical accelerations measured at the Holiday Inn building in Van Nuys was evaluated. The effect of the vertical acceleration was found to be small compared to the gravity loads on the building [Blume 1972]. Some of these previous studies however, did not consider the significance of near fault amplification of the vertical response.

CASE STUDY BUILDINGS

To evaluate the significance of vertical accelerations on the design of buildings, three case study buildings have been developed. Each of the buildings has been analyzed using a response spectrum analysis for a series of earthquakes. The earthquake response spectra used in the analyses have identical horizontal spectra applied in the horizontal directions and varying vertical accelerations. The horizontal spectra used was based on the standard response spectra specified by the Uniform Building Code for soil type S2. The following is a description of the vertical acceleration spectra that were used:

1) No vertical response spectra
2) A vertical response spectra scaled to 2/3 of the horizontal spectra
3) A vertical response spectra scaled to represent a distance 5 kilometers from the fault. Scaling was done following the recommendations of Bozorgnia et al, 1996.
4) A vertical response spectra scaled to represent a distance 10 kilometers from the fault. Scaling was done following the recommendations of Bozorgnia et al, 1996.

The scaling of the verical spectra to account for fault distance was applied by multiplying the period for each point on the horizontal spectra by 0.5. The spectral acceleration of each point was then multiplied by a factor of 0.75 for a distance of 5 kilometers and by a factor of 0.65 for a distance of 10 kilometers. The response spectra used are presented in Figure 1. In this figure, the vertical spectra scaled to 2/3 of the horizontal spectra is significantly higher than the other vertical spectra in the period range above about 0.4 seconds. In the short period range however, the vertical spectra representing a distance of 5 kilometers from the fault is greater than the 2/3 of horizontal spectra. At a period of about 0.1 seconds, the vertical spectra representing a distance of 5 kilometers is slightly greater than the horizontal spectra.

For most building design analyses, engineers are not able to accurately model the stiffness and mass of every element in the building. Simplifying assumptions are generally made to reduce the time and effort required for the analysis. In modeling the case study buildings, the analysis focused on the vertical vibrational behavior that was thought to produce the most significant effect. This was done to attempt to capture the significant effects of vertical acceleration while reducing the analysis effort to one that might be acceptable in a typical design. To verify that the predominant vibrational characteristics were captured, the analysis effort was adjusted so that 90 percent of the effective modal weight was cpatured in both horizontal and the vertical direction for each model. This value is based on the requirements of the UBC dynamic analysis provisions.

Case Study Building 1

The first case study building is a two to three story concrete parking garage constructed with precast concrete beams and girders. A plan of the building is shown in Figure 2. The length of the building is about 320 feet long and the width is about 140 feet. An expansion joint is located across the short direction of the building at approximately the middle of the building. The driving ramp is created by diving the slab in the longitudinal direction and sloping each section of slab. The floor framing consists of precast double-tee beams that span about 27-½ feet between precast concrete I-girders. The I-girders span about 70 feet and are supported on precast concrete columns. A reinforced concrete topping slab is placed over the top of the double-tee beams. Lateral forces are resisted by reinforced concrete shear walls in each direction.

The predominant modes of vibration of the building in the horizontal directions were judged to be bending and shear deformation of the shear walls and in-plane bending and shear deformation of the floor slabs between the shear walls. The primary shear walls and floor slabs were modeled as shell elements using the SAP2000 computer program. Dead loads were applied using the self weights of the individual elements.

Initially, the building mass and stiffness was distributed in accordance with the appropriate mass and stiffness of the individual structural elements. In modeling the building, the precast double-tee beams, acting compositely with the topping slab, were converted into an equivalent plate bending element to capture the appropriate stiffness for resisting vertical loads without the need to discretely modeling all of the double-tee beams. With this model however, the stiffness of the equivalent plate bending element was accurate for bending around the axis parallel to the direction of the I-girders, but the stiffness was too high for bending in the perpendicular direction since the shell elements could not be modeled with different stiffnesses in the two orthogonal directions. With this model, the artifically high stiffness of the equivalent floor slab elements prevented the I-girders from behaving as a uniform beam element for resisting vertical loads. To eliminate this problem, the mass of the floors was lumped along the length of the I-griders and the plate bending stiffness of the equivalent floor slabs was reduced artificially to prevent the floor slabs from affecting the behavior of the I-girders.

For vertical modes of vibration, the bending of the long span I-girders was identified as the primary structural response. A simplified hand analysis of the typical I-girder found that for a simple span condition, the fundamental period of the vibration of the I-girder would be about 0.54 seconds. For a fixed-span condition, the fundamental period of vibration would be about 0.23 seconds.

Initially, an eigen vector analysis was conducted to calculate the vibrational modes of the building. The analysis results indicated that the first 54 modes for the building were attributed to vertical vibration of the I-girders and most of those modes had periods in the range of 0.44 to 0.39 seconds. However, using 120 modes of vibration did not capture 90 percent of the effective modal mass in any direction. The analysis was changed to using Ritz vectors as a means of reducing the computational effort [Wilson 1997]. Using the Ritz vector approach, 90 percent of the effectiv emodal mass was cpatured after just 60 modes and the first 34 modes represented vertical vibration of the beams. The large number of vertical modes is necessary because the sloped floor slab for the ramp creates a slightly different effective stiffness of the columns along the center of the garage. By reducing, and effectively eliminating, the stiffness of the double tee beams each of athe I-girders are uncoupled and free to vibrate independantly.

The element forces from the analyses were tabulated for typical elements to compare the effect of the various response spectrum cases. Since vertical motion of the beam was an important consideration, shear and bending moments in a typical beam was included. The axial load in an interior and an exterior column were also compared. These results are presented in Table 1.

Table 1

	LOAD CASES				
Element	Dead Load	Horizontal spectra	Horizontal spectra w/ vertical as 2/3 horizontal	Horizontal spectra w/ vertical scaled for 5 km from fault	Horizontal spectra w/ vertical scaled for 10 km from fault
I-girder max bending moment	10,377 kip-in	823 kip-in	6218 kip-in	4768 kip-in	4139 kip-in
I-girder max shear	51 kip	1.4 kips	30 kips	24 kips	20 kips
Exterior lower story column axial load	111 kips	2.5 kips	64 kips	47 kips	41 kips
Interior lower story column axial load	217 kips	8.6 kips	94 kips	79 kips	69 kips
Uypper story transfer girder shear	69 kips	1.9 kips	36 kips	27 kips	24 kips

Case Study Building 2

The second case study building is a four story building used for heavy storage. The building has a 10 inch thick reinforced concrete slab at each floor. The concrete slab is supported by steel columns, uniformly spaced at about 15 feet in each direction. Around the perimeter of the building, there are steel beams that also support the floor slab. The exterior façade of the building consists of lightweight glass fiber reinforced concrete (GFRC) panels attached to the perimeter steel beams. Lateral forces are resisted by a series of X-braced frames located around the perimeter of the building. An isometric view of the

A majority of the mass of the building is concentrated in the concrete floor slabs. For response to horizontal accelerations, the mass could be lumped at the centroid of the floor. To evaluate the significance of the vertical response of the floor slabs, the mass and stiffness of the floors was modelled using shell elements with the mass distributed to each slab element. The mass of the exterior façade was modelled as nodal masses located at the points of attachment of the GFRC panels to the steel beams.

The predominant lateral mode of vibration was thought to be bending of the braced frame bays. In the vertical direction, the predominant mode of vibration was judged to be bending of the slab between supporting columns. Axial deformation of the columns was also thought to be a possible significant vertical response mode.

An initial analysis was performed using a single shell element representing a bay between column lines. In this analysis however, the axial deformation of the columns was observed, however the vertical modes of vibration of the shell elements were not properly captured. A second model was developed in which each bay was modelled with four shell elements. This model captured the vertical vibrational mode of the floor slab bending between columns. A third model was created with each bay modelled with nine shell elements in a three-by-three grid. The results of the four element per bay model were compared to the nine element per bay model. The difference in period of vibration of the vertical modes was less than 10 percent so the courser model was used to simplify the analysis.

The analysis indicated that the first significant vertical mode of vibration was the ninth mode, which has a period of about 0.08 seconds. Although only five modes were necessary to capture more than 90 percent of the effective mass in the horizontal directions, 30 modes were required to capture 90 percent of the effective mass in the vertical direction.

The forces from selected representative elements are presented in Table 2 to compare the force level for the various response spectra. Some of the columns experienced axial forces due to the response spectra that were on the order of magnitude as the dead load on the column. For most of the interior, the forces from the various response spectra cases that included vertical acceleration were of similar magnitude and the axial load from the vertical acceleration cases was greater than for the case using only a horizontal spectra. For the exterior columns, there was a significant axial force from the case with only a horizontal spectra. This was found for columns that were part of the braced frames as well as for other exterior columns. For these columns, the contribution of the vertical acceleration to the axial force was small.

Table 2

Element	LOAD CASES				
	Dead Load	Horizontal spectra	Horizontal spectra w/ vertical as 2/3 horizontal	Horizontal spectra w/ vertical scaled for 5 km from fault	Horizontal spectra w/ vertical scaled for 10 km from fault
Interior column axial load fourth floor	29.2 kips	0.5 kips	17.0 kips	34.7 kips	30.1 kips
Exterior column axial load second floor	50 kips	48 kips	50 kip	52 kips	51 kips
Braced frame col. axial load first floor	71 kips	1360 kips	1360 kips	1360 kips	1360 kips
Brace axial load second floor	10 kips	185 kips	185 kip	185 kips	185 kips
Third floor slab midspan bending	0.7 kip-in/in	3.3 kip-in/in	3.6 kip-in/in	3.9 kip-in/in	3.7 kip-in/in

Case Study Building 3

The third case study building is a 16-story steel building, 230 feet long by 100 feet wide [Figure 4]. Each floor has concrete slab over a metal deck and is supported by floor beams at about 11-foot on center. Floor beams are, in turn, supported on larger floor girders. The lateral system consists of perimeter steel moment frames and two additional interior moment frames along the short direction of the building.

Conventional lateral analysis of a building of this type would involve modelling the moment frames and the floors as diaphragm constraints. Gravity beams and columns are typically not modelled for lateral analysis. Each floor mass would be lumped at the floor center of mass. To account for the effects of vertical acceleration, a more elaborate model was created. Gravity columns were modelled in addition to the moment frames. Neither the floor beams nor the slabs were included in the model. The larger gravity girders were included with moment releases at the ends. Masses were distributed throughout each floor at the points where floor beams are connected to the larger floor girders. Mass of the exterior cladding was also distributed at third points of the perimeter moment frame girders.

An eigenvalue analysis of the model showed that 20 modes are sufficient to capture about 95 percent of the vertical mass and 99 percent of the horizontal mass in both horizontal directions. Among these 20 modes, four significant vertical modes were captured, which are modes 14, 16, 18 and 20, with periods ranging from 0.23 to 0.07 seconds. The first vertical mode [Figure 4] had a period of almost one-tenth of the first horizontal mode of the building (2.5 seconds). The vertical modes involve different combinations of vertical vibration of the floor girders or the moment frame girders.

Forces in selected moment frame and gravity members are summarized in Table 3. Moment frame end columns experience large axial forces resulting from overturning effects due to horizontal ground motions. Since the axial forces resulting from overturning are smaller in the upper floors, effects of vertical ground acceleration become more significant in the moment frame columns of the upper stories. These effects are also significant for intermediate moment frame columns and gravity columns. Vibration of the floor girders caused by vertical acceleration introduces a substantial increase in the axial forces in these columns. The axial forces may exceed those due to dead loads in the upper floors. Frame girders experience little to no increase in moments due to vertical acceleration. However, gravity floor girders are subjected to significant increase in the mid-span moment due to vertical vibration.

Table 3

Element	LOAD CASES				
	Dead Load	Horizontal Spectra	Horizontal spectra with vertical as 2/3 horizontal	Horizontal spectra with vertical scaled for 5 km from fault	Horizontal spectra with vertical scaled for 10 km from fault
Axial load in moment frame intermediate column, upper floor	58 kips	0	59 kips	66 kips	57 kips
Axial load in moment frame end column, lower floor	402 kips	1671 kips	1677 kips	1681 kips	1679 kips
Axial load in gravity column, upper floor	52 kips	0	42.5 kips	47.8 kips	41.4 kips
Axial load in gravity column, lower floor	859	4.3 kips	409 kips	466 kips	404 kips
Maximum moment in floor girders (mid-span moment)	76.5 k-ft	0.64 k-ft	57 k-ft	64.3 k-ft	55.8 k-ft
Maximum moment in frame girder (end moment)	64.3 k-ft	310 k-ft	316 k-ft	318 k-ft	316 k-ft

CONCLUSIONS

Earthquake excitation occurs in horizontal and vertical directions. For the most part, engineers are concerned with the horizontal shaking caused by an earthquake and little attention has been given to the vertical acceleration. After the Northridge earthquake in 1994, increased attention has been given to the significance of vertical accelerations from an earthquake on the response of structures. Some engineers have speculated that vertical accelerations are partially responsible for some of the damage.

Ground motion records from Northridge and other earthquakes have shown that in general the magnitudes of vertical accelerations are less that the horizontal accelerations. However, in the near fault region the vertical acceleration may exceed the horizontal acceleration. The effect of these vertical accelerations on the response of buildings has been studied for some typical buildings. Based on this limited study, the following conclusions have been made.

Proper modelling of the building to account for the vertical excitation is necessary to capture the significant vertical modes of vibration. Seldom is it necessary to discretely model every element in the building with the proper mass and stiffness. If the effects of vertical modes of vibration need to be included, it is important to identify the likely modes of significance and to develop the analysis model so that the stiffness and mass of the elements necessary to capture those modes are properly modelled. Modelling of other elements can result in confusing results or excessive computational effort.

Analysis of a building considering vertical response can often require calculating a large number of modes of vibration in order to capture 90 percent of the effective mass in each direction. The use of Ritz vectors can greatly reduce the computational effort by reducing the number of modes necessary to capture most of the effective mass.

The forces on the structural elements induced by vertical acceleration are often much less than the effects of dead load. The overturning effects of the horizontal accelerations of the building may also be more significant that the effects of vertical acceleration. In come cases, the forces due to vertical acceleration may be of the same magnitude as the dead load, indicating that the dead load may be overcome during earthquake shaking. This was found primarily for the long span concrete girders.

Variations in the shape and magnitude of the vertical response spectra produced slight variations in the response of some of the elements. The period of vibration in the vertical direction influenced which vertical spectra would govern for an element. For the long span beams with periods of about 0.5 seconds, the spectra based on scaling the horizontal spectra by two-thirds was critical. For other structures with shorter vertical periods of vibration, the spectra based on scaling the spectra using the approach set forth by Bozorgnia et al was more critical. In all cases, using the spectra scaling for 5-kilometer distance from the fault produced forces that were greater than that for the spectra scaled for 10 kilometers distance from the fault.

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