Explicit Consideration of Vertical Earthquake Ground Motion in the Design of Structures

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ABSTRACT

Current U.S. design practice includes consideration of vertical earthquake ground motion in the design of structures. The dead load factor of 1.2 is increased by $0.2S_{DS}$ in the additive seismic load combination where gravity effects add to earthquake effects. The dead load factor of 0.9 is decreased by the same $0.2S_{DS}$ in the counteractive seismic load combination where gravity effects counteract earthquake effects. S_{DS} is the design spectral response acceleration at short periods.

This paper is directed at exploring whether explicit consideration of vertical earthquake ground motion should be required for certain structures. To arrive at a determination, the treatment of vertical earthquake ground motion in the seismic design of structures in U.S. Codes and standards and in Eurocode 8 is reviewed and documented. A survey of information available in the literature on the features of vertical earthquake ground motion and its potential impact on structural design is carried out and documented. On the basis of both surveys, structures and structural members that appear to be more sensitive to the consideration or non-consideration of vertical earthquake ground motion in their designs are identified. In the absence of detailed studies to investigate the adequacy of the code-specified design force of $0.2S_{DS}D$ for structural members subjected to vertical ground motion, an interim measure that might be considered for structures identified to be sensitive is proposed.

INTRODUCTION

The effects of vertical earthquake ground motion on buildings have traditionally been given much less attention than the effectsofhorizontal ground motion. This is largely due to the belief that the peak vertical ground acceleration is considerably smaller than the peak horizontal ground acceleration.A fairly large safety factor against static vertical loads also exists in engineered buildings. Building codes around the world, in keeping with this thinking, have given less attention to the effects of vertical shaking in buildings.

ATC 3-06 (Applied Technology Council, 1978), published in 1978, the predecessor to the NEHRP *Provisions* (Building Seismic Safety Council,1985 and subsequent), which has formed the basis of seismic design provisions in U.S. codes and standards since the late 1980s, was the first significant document to recognize the importance of vertical earthquake ground motion in seismic design by incorporating it in the seismic design load combinations for structural members. The ATC 3 provisions in this regard evolved through various editions of the NEHRP *Provisions* and U.S. codes and standards. The latest provisions from ASCE 7-16 (American Society of Civil Engineers, 2016) are presented in this paper.

ATC 3 made a separate provision for horizontal cantilevers and horizontal prestressed concrete members, because they are particularly vulnerable to vertical earthquake ground motion. Those provisions also evolved through various editions of the NEHRP *Provisions* and through various U.S. codes and standards. The latest provisions from ASCE 7-16 (American Society of Civil Engineers, 2016) are also presented in this paper.

Explicit consideration of vertical earthquake ground motion in seismic design was raised as a possibility for the first time in the 1997 UBC (International Conference of Building Officials, 1997). It stated that a vertical ground motion spectrum, if needed, could be obtained by simply scaling the horizontal ground motion spectrum of the 1997 UBC by a factor of two-thirds. No mention of a vertical spectrum was made in U.S. codes and standards until that changed in ASCE 41-13 (American Society of Civil Engineers, 2013) and ASCE 7-16 (American Society of Civil Engineers, 2016). The 2009 NEHRP *Provisions* (Building Seismic Safety Council, 2009) was the first edition of that document to provide a procedure for defining the design vertical response spectrum, based on the studies of horizontal and vertical ground motions by Campbell and Bozorgnia (2003) and Bozorgnia and Campbell (2004). The vertical ground motion spectrum is discussed in this paper.

Seismic Design by ASCE 7-16 and 2018 IBC

Basics

Figure 1 shows the idealized force-displacement relationship of a structure subjected to the design earthquake of the 2018 IBC (International Code Council, 2018) and ASCE 7-16 (American Society of Civil Engineers, 2016). On the horizontal axis are the earthquake-induced displacements. The quantity *V*, comparable to V_B of IS 1893 (Bureau of Indian Standards, 2016), along the vertical axis is the code notation for design base shear, a global force quantity. The curve in the figure may be thought of as the envelope or the backbone curve of hysteretic force-displacement loops that describe the response of a structure subjected to reversed cyclic displacement histories of the type imposed by earthquake ground motion.

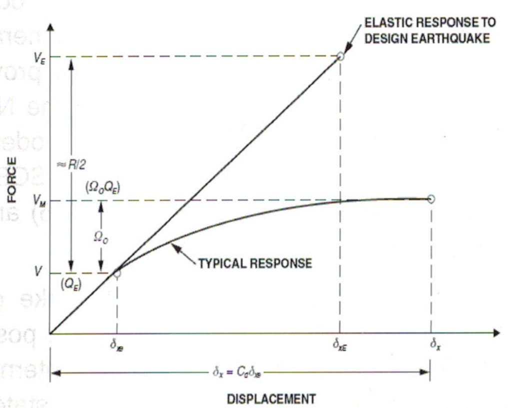


Fig. 1: Idealized force-displacement relationship of a structure subjected to the design earthquake of the 2018 IBC or ASCE 7-16

The base shear V is to be distributed along the height of the structure as required by ASCE 7-16 (the IS 1893 distribution of VB along the height of the structure is parabolic, starting from zero at the base; ASCE 7 uses this distribution for long-period structures, uses a linear distribution for short-period structures, and uses a linear interpolation between a linear and a parabolic distribution for structures with intermediate periods). The distribution results in a series of lateral forces concentrated at the various floor levels (F_i at Floor Level *i* in ASCE 7-16; Q_i at Floor Level *i* in IS 1893). Next, a mathematical model of the structure is to be elastically analyzed under these lateral forces (Figure 2). The quantity δ_{xe} represents the lateral displacement at floor level *x* obtained from this analysis, and Q_E represents the member forces (bending moments, shear forces, axial forces, etc.). This procedure is called the equivalent lateral force procedure.

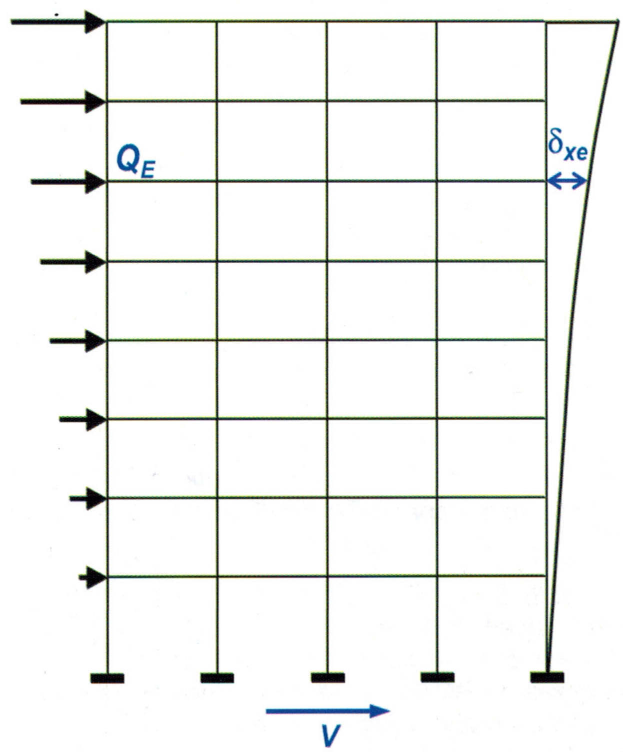


Fig. 2: Distribution of lateral forces along height of structure

As the structure responds inelastically to the design earthquake of ASCE 7-16 and the 2018 IBC, the lateral displacement at floor level *x* increases from δ_{xe} to $C_d\delta_{xe}$, and the member forces increase from Q_E to $\Omega_o Q_E$. Both the deflection amplification factor C_d and the overstrength factor Ω_o depend on the structural system used for earthquake resistance, and are given in ASCE 7-16 Table 12.2-1. Quantities V_E and δ_{xE} are the base shear and the lateral displacement at floor level *x*, respectively, corresponding to the hypothetical elastic response of the structure to the design earthquake of ASCE 7-16 and the 2018 IBC. Figure 1 suggests that a response modification factor *R* of 2 used in design would result in an essentially elastic response of a structure to the design earthquake. The basis for this is explained in (Ghosh and Henry, 2009).

Seismic Design Category

Every structure is assigned to a Seismic Design category (SDC), which is used in the IBC and ASCE 7 to determine permissible structural systems, limitations on structural height and irregularity, the components of the structure that must be designed for seismic resistance, and the types of lateral force analysis that must be performed. Very importantly, the SDC is also used to determine the level of detailing that is required to be done for a structure: ordinary, intermediate, or special. Note that an SDC need not be determined for those structures for which earthquake effects need not be considered.

The seismic design category is a function of occupancy or use (risk category) and of soil-modified seismic risk at the site of the structure in the form of the design spectral response accelerations at short periods, S_{DS}, and at 1-sec period, S_{D1}. Risk Category (RC) I structures (such as a barn) represent low risk to human life in the event of failure. RC II structures (an office building or an apartment building) are all structures not assigned to RC I, III, or IV. The failure of RC III structures (such as a school) could pose a substantial risk to human life. Structures designated as essential facilities (hospitals, fire stations, police stations) are assigned to RC IV. A structure located where S_1^{3} 0.6gis assigned to SDC E if its risk category is I, II, or III and to SDC F if its risk category is IV. $S_1$is the mappedspectral response accelerationat 1-sec period. For structures not assigned to SDC E or F, the SDC needs to be determined twice – first as a function of S_{DS} by ASCE 7-16 Table 11.6-1 and a second time as a function of S_{D1} by ASCE 7-16 Table 11.6-2; the more severe category governs.

Design Load Combinations Including Vertical Earthquake Effect

The two seismic design load combinations of ASCE 7-16 and the 2018 IBC, when reduced to just dead loads, live loafs, snow loads, and earthquake effects, are:

- $1.2D + 1.0E + 0.5L + 0.2S$(IBC Eq. 16-5)
- $0.9D + 1.0E$(IBC Eq. 16-7)

The first is the additive load combination in which gravity effects add to earthquake effects and the second is the counteractive load combination in which gravity effects counteract earhquake effects.In the additive load combination, the live load factor is required to be 1.0, rather than 0.5, in thedesigin of parking structures, places of public assembly, and areas supporting more than 100 psf (4.8 kN/m^2)of live load.

Seismic Force Effect, *E*

According to 2018 IBC Section 1602, Notations, $E$is the combined effect of horizontal and vertical earthquake forces, $E_h \pm E_v$, as defined in ASCE 7-16 Section 2.3.6. According to that section, for use in 2018 IBC Eq. 16-5, $E = E_h + E_v$. With E_h and E_v as defined by ASCE 7-16Eqs. 12.4-3, and 12.4-4a, respectively, when the effects of gravity and seismic ground motion are additive,

$$E = \rho Q_E + 0.2 S_{DS} D$$

where Q_E = effect of the F_x forces obtained by distributing the design base shear V along the height of the structure in the manner prescribed by ASCE 7-16 Eqs. 12.8-11 and 12.8-12.

ρ = redundancy factor determined in accordance with ASCE 7-16 Section 12.3.4 (see below) for structures assigned to SDC D, E, or F

= 1.0 for structures assigned to SDC A, B, or C

Also according to ASCE 7-16 Section 2.3.6, for use in 2018 IBC Eq. 16-7, $E = E_h - E_v$. With E_h and E_v as defined by ASCE 7-16Eqs. 12.4-3, and 12.4-4a, respectively, where the effects of gravity and seismic ground motion are counteractive:

$$E = \rho Q_E - 0.2 S_{DS} D$$

Substituting for E in 2018 IBC Eqs. (16-5) and (16-7), the seismic design load combinations of the 2018 IBC become:

- $(1.2 + 0.2S_{DS})D + 1.0\rho Q_E + 0.5L + 0.2S$(IBC Eq. 16-5)
- $(0.9 - 0.2S_{DS})D + 1.0\rho Q_E$(IBC Eq. 16-7)

Thus the dead load factor is increased by $0.2S_{DS}$ in the additive load combination by considering vertically downward earthquake effect and it is decreased by $0.2S_{DS}$in the counteractive load combination by considering vertically upward earthquake effect because, in each case, that is the conservative way to go.

Redundancy

The basic premise of the redundancy provisions in ASCE 7-16 Section 12.3.4 is that the most logical way to determine lack of redundancy is to check whether a component's failure results in an unacceptable amount of story strength loss or in the introduction of extreme torsional irregularity. In ASCE 7-16, the redundancy factor, ρ, is equal to either 1.0 or 1.3, depending upon whether or not an individual element can be removed (deemed to have failed or lost its moment-resisting capabilities) from the seismic force-resisting system without causing the remaining structure to suffer a reduction in story strength of more than 33 percent or creating an extreme torsional irregularity (Horizontal Structural Irregularity Type 1bin ASCE 7-16 Table12.3-1).

Maximum Seismic Force Effect, E_m

2018 IBC Section 1605.1 requires that buildings and other structures and portions thereof be designed to resist the seismic load effects including overstrength factor in accordance with ASCE 7-16 Sections 2.3.6 and 2.4.5 where required by ASCE 7-16 Chapters 12, 13, and 15. There are three sections in Chapter 12 that require design to resist seismic load effects including overstrength factor: 12.2.5.2 - foundation and other elements used to provide overturning resistance at the base of cantilever column elements (SDC B or higher), 12.3.3.3 - elements supporting discontinuous walls or frames of structures having certain horizontal or vertical irregularities (SDC B or higher), and 12.10.2.1 - diaphragm collector elements, splices, and their connections to resisting elements (SDC C and above). The load combinations including overstrength factor in ASCE-16 Section 2.3.6 are:

- $1.2D + E_v + E_{mh} + L + 0.2S$(ASCE 2.3.6 Combination 6)
- $0.9D - E_v + E_{mh}$(ASCE 2.3.6 Combination 7)

where $E_{mh} \pm E_v = E_m$ is the maximum effect of horizontal and vertical forces determined from:

$E_m = W_o Q_E + 0.2S_{DS}D$
(when effects of gravity and seismic forces are additive;

ASCE Eqs. 12.4-5, 12.4-7, and 12.4-4a)

$E_m = W_o Q_E - 0.2S_{DS}D$ (when effects of gravity and seismic forces counteract;
ASCE Eqs. 12.4-6, 12.4-7, and 12.4-4a)

The overstrength factor Ω_o, which is given in ASCE 7-16 Table 12.2-1 for the various seismic force-resisting systems, increases the effects of code-prescribed seismic forces to represent the actual forces that may be experienced in a structural member as a result of the design earthquake ground motion (see Figure 1, which also shows the corresponding displacement that is expected when Q_E are amplified by Ω_o). The term $\Omega_o Q_E$ need not exceed the maximum force that can be transferred to an element by the other elements of the seismic force-resisting system.

Special Requirements For Horizontal Cantilevers And Horizontal Prestressed Members

ASCE 7-16 contains the following provision:

12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D trough F. In structures assigned to Seismic Design Category D, E, or F, horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations in Section 12.4.

Thus there are three "seismic" load combinations for horizontal cantilevers and horizontal prestressed members:

a. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$
b. $(0.9 - 0.S_{DS})D + \rho Q_E + 1.6H$
c. $-0.2D$

VERTICAL GROUND MOTION SPECTRUM

Traditionally, vertical response spectra are taken as two-thirds of the horizontal spectrum developed for the site. While this is a reasonable approximation for most sites, vertical response spectra at sites located within a few kilometers of the zone of fault rupture can have stronger vertical response spectra than determined by this approximation. Chapter 23 of FEMA P750 [2009 NEHRP Provisions] provides additional information on vertical ground motions, including procedures to construct a separate vertical earthquake response spectrum.The procedure for defining the design vertical response spectrumis based on studies of horizontal and vertical ground motions conducted by Campbell and Bozorgnia (2003) andBozorgnia and Campbell (2004). These procedures are also generally compatible with theobservations of Abrahamson and Silva (1997) and Silva (1997) and the proposed design procedures of Elnashai (1997).

Development of site-specific response spectra for near-field sitesis recommended where vertical response must be considered for buildings. Kehoe and Attalia (2000) present modeling considerations that should be accounted for where analyzing for vertical effects.

ASCE 7-16 has just introduced a vertical design spectrum for optional use. ASCE 7-16 provisions in this regard, based on those of the 2009 NEHRP Provisions, are given below.

ASCE 7-16 11.9 Vertical Ground Motions for Seismic Design

ASCE 7-16 11.9.1 General

If the option to incorporate the effects of vertical seismic ground motions is exercised in lieu of the requirements of Section 12.4.2.2, the requirements of this section are permitted to be used in the determination of the vertical design earthquake ground motions. The requirements of Section 11.9 shall only apply to structures in Seismic Design Categories C, D, E, and F

ASCE 7-16 11.9.2 MCE$_R$ Vertical Response Spectrum

Where a vertical response spectrum is required by this standard and site-specific procedures are not used, the MCE$_R$ vertical response spectral acceleration, S_{aMv}, shall be developed as follows:

1. For vertical periods less than or equal to 0.025 sec, S_{aMv} shall be determined in accordance with Eq. 11.9.2-1 as follows:

$$S_{aMv} = 0.3C_vS_{MS} \quad (11.9.2\text{-}1)$$

2. For vertical periods greater than 0.025 sec and less than or equal to 0.05 sec, S_{aMv} shall be determined in accordance with Eq. 11.9.2-2 as follows:

$$S_{aMv} = 20C_vS_{MS}(T_v - 0.025) + 0.3C_vS_{MS} \quad (11.9.2\text{-}2)$$

3. For vertical periods greater than 0.05 sec and less than or equal to 0.15 sec, S_{aMv} shall be determined in accordance with Eq. 11.9.2-3 as follows:

$$S_{aMv} = 0.8\ C_vS_{MS} \quad (11.9.2\text{-}3)$$

4. For vertical periods greater than 0.15 sec and less than or equal to 2.0 sec, S_{aMv} shall be determined in accordance with Eq. 11.9.2-4 as follows:

$$S_{aMv} = 0.8C_vS_{MS}\left(\frac{0.15}{T_v}\right)^{0.75} \quad (11.9.2\text{-}4)$$

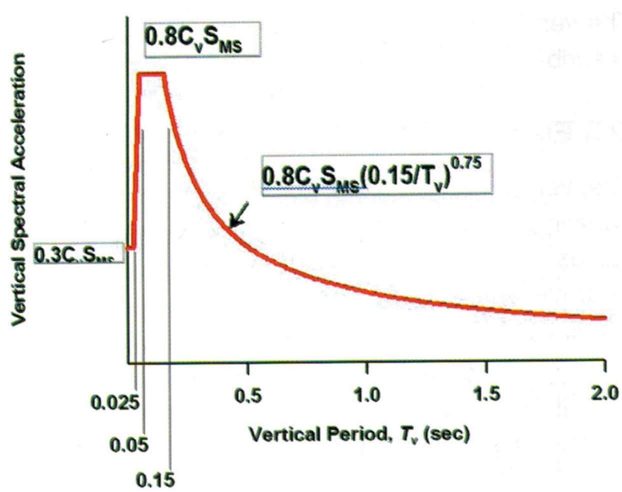
where:

C_v is defined in terms of S_S in Table 11.9.2-1,

S_{MS}= the MCE$_R$ spectral response acceleration parameter at short periods, and

T_v= the vertical period of vibration.

S_{aMv} shall not be less than one-half (1/2) of the corresponding S_{aM} for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.



ASCE 7-16 Figure C11.9.2-1.Illustrative example of the vertical response spectrum

For vertical periods greater than 2.0 sec, S_{Mav} shall be developed from a site-specific procedure; however, the resulting ordinate of S_{aMv} shall not be less than one-half (1/2) of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

ASCE 7-16 Table 11.9.2-1
Values of Vertical Coefficient C_v

Mapped MCE_R Spectral Response Parameter at Short Periods [a]	Site Class C	Site Class D, E, F
$S_S \geq 2.0$	1.3	1.5
$S_S = 1.0$	1.1	1.3
$S_S = 0.6$	1.0	1.1
$S_S = 0.3$	0.8	0.9
$S_S \leq 0.2$	0.7	0.7

In lieu of using the above procedure, a site-specific study is permitted to be performed to obtain S_{aMv} at vertical periods less than or equal to 2.0 sec, but the value so determined shall not be less than 80 percent of the S_{aMv} value determined from Eqs. 11.9.2-1 through 11.9.2-4.

ASCE 7-16 11.9.3 Design Vertical Response Spectrum

The design vertical response spectral acceleration, $S_{av,}$ shall be taken as two-thirds of the value of S_{aMv} determined in Section 11.9.2.

The vertical ground motion spectra of Eurocode 8 are described below.

D.5 Eurocode 8 (1994)

The vertical spectrum S_{av}, in EC-8 was tied to the horizontal spectrum S_{ah}, both in terms of shape and values. The following expressions were used in order to arrive at an elastic spectrum for vertical response [CEN, EC8, 1994):

if $T<0.15$s then $S_{av} = 0.7\ S_{ah}$;

if $T> 0.50$ sthen $S_{av} = 0.5\ S_{ah}$;

and for 0.15 s $\leq T \leq 0.5$ s, we linearly interpolate from the above.

Furthermore, the code distinguished between the behavior factor used for the horizontal and vertical spectrum and explicitly stated that the latter should always be assumed equal to 1.0. However, no distinction was made for equivalent viscous damping used in design against vertical motion, and in the case of RC frames, it was proposed that 5%be used for both cases.

The Eurocode8 vertical spectrum was a significant improvement over other codes, since it recognized, to some extent, the difference in frequency content between vertical and horizontal motion. It was, however, seriously unconservative in the near field, as demonstrated in Elnashai and Papazoglou (1997).

D. 6 Eurocode 8 (2004)

The design vertical spectrum in the 2004 edition of the Eurocode 8 was completely revised from its 1997 counterpart, and is as shown below:

$$0 \leq T \leq T_B : S_d(T) = a_g \left[\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \leq T \leq T_C : S_d(T) = a_g \frac{2.5}{q}$$

$$T_C \leq T \leq T_D : S_d(T) = \begin{cases} a_g \dfrac{2.5}{q}\left[\dfrac{T_C}{T}\right] \\ \geq \beta a_g \end{cases}$$

$$T_D \leq T : S_d(T) = \begin{cases} a_g \dfrac{2.5}{q}\left[\dfrac{T_C T_D}{T^2}\right] \\ \geq \beta a_g \end{cases}$$

wherea_{vg} is the design vertical ground acceleration on Type A ground. Its value is a fraction of the design horizontal ground motion acceleration, and is given in Table 3.4 of the code. T_B and T_C are the periods marking the lower and upper limits of the constant-acceleration part of the response spectrum, respectively, and T_D is the beginning of the constant-displacement part of the response spectrum. Thesevalues are also provided in Table 3.4 of the code. The factor *q* is the behavior factor of the structure (similar to *R*-factor in ASCE 7), which is to be generally taken as 1.5 for response in the vertical direction. β is the lower bound factor for the design spectrum, which is to be taken as 0.2 or as provided in National Annex for a particular country.

The remainder of this paper provides background to the code-related developments above.

Vertical-To-Horizontal Peak Ground Acceleration Ratio

Historically, the amplitude of vertical ground motion has been inferred to be two-thirds (2/3) the amplitude of the horizontal ground motion. However, studies of horizontal and vertical ground motions over the past 25 years have shown that such a simple approach is not valid in many situations (e.g., Bozorgnia and Campbell, 2004, and references therein) for the following main reasons: (a) vertical ground motion has a larger proportion of short-period (high-frequency) spectral content than horizontal ground motion and this difference increases with decreasing soil stiffness and (b) vertical ground motion attenuates at a higher rate than horizontal ground motion and this difference increases with decreasing distance from the earthquake.

The observed differences in the spectral content and attenuation rate of vertical and horizontal ground motion lead to the following observations regarding the vertical/horizontal (*V/H*) spectral ratio (Bozorgnia and Campbell, 2004):

1. The *V/H* spectral ratio is relatively sensitive to spectral period, distance from the earthquake, local site conditions, and earthquake magnitude (but only for relatively soft sites) and relatively insensitive to earthquake mechanism and sediment depth;
2. The *V/H* spectral ratio has a distinct peak at short periods that generally exceeds 2/3 in the near-source region of an earthquake; and
3. The *V/H* spectral ratio is generally less than 2/3 at mid-to-long periods.

Therefore, depending on the period, the distance to the fault, and the local site conditions of interest, use of the traditional 2/3 *V/H* spectral ratio can result in either an underestimation or an overestimation of the expected vertical ground motions.

Even the earliest strong-motion records showed that the *V/H* ratio can exceed the value of 2/3, but these records were often excluded asoutliers or as non-representative of general trends. An example ofsuch anearlyrecord is the 1933 Long Beach earthquake (M_L = 6.2), recorded 6kmfrom thesource, which had a *V/H* that exceeded the value of 1.0. Even the most frequentlycited strong-motion record, the 1940 El-Centro, had a *V/H* ratio equal to 0.98.Since these early events, many near-field records have consistently shown *V/H* values in excess of 2/3, thus suggesting that such observations are not unusual.In general, the *V/H* ratio exceeds 2/3 for distances of less than 10 km from fault rupture and exceeds unity at high magnitudes for distances of less than about 5 km (Abrahamson and Litehiser, 1989; Bureau, 1981; Campbell, 1982; Niaziand Bozorgnia,1991). On the contrary, the 2/3-rule is over-conservative at larger distances, with a value of 0.50 suggested for distances on the order of 50 km (Abrahamson andLitehiser, 1989; Bureau, 1981; Campbell, 1982; Niazi and Bozorgnia, 1991). The above statements suggest that vertical motion attenuates more rapidly with distance than the horizontal components, a fact consistent with the frequency content associated with each of the two components.

Elnashai and Papazouglou(1997)produced the graphs in Figure 3,indicating vertical-to-horizontal peak ground acceleration ratios for varying magnitude and distance. Part of the data, from the Imperial College data bank, was for near-field earthquakes with a magnitude M_s> 5.0, whilst the remainder was from Borzognia and Niazi(1993).This figure may be used to give values of vertical acceleration if the horizontal acceleration, magnitude, and the distance from the source are known. Alternatively, the vertical peak ground acceleration can be estimated by useof an attenuation relationship, such as that from Ambraseys and Simpson (I995).

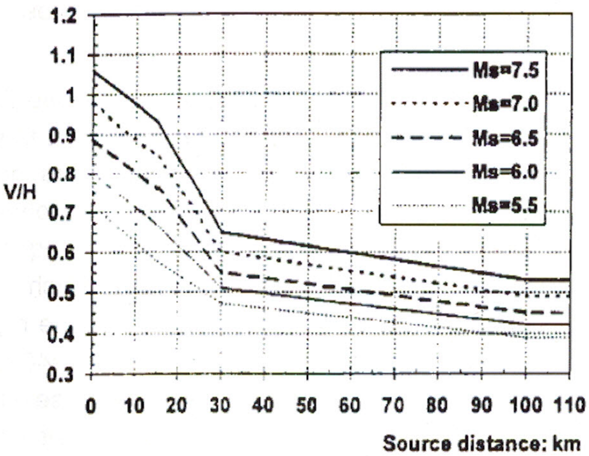


Fig. 3: V/H ratios for different magnitudes and source distances (Elnashai and Papazouglou, 1997)

Elnashai's study of the Imperial Valley and Morgan Hill earthquake records confirmed that the *V/H* ratio is greater than 1.0 in the very near field, but diminishes to less than half with distance, as shown in Figure 4.Greater *V/H* ratios were observed for the higher magnitude (Imperial Valley) records near the source, which agrees with the results of studies by Borzognia and Niazi (1993), Abrahamson and Litehiser (1989) and Elnashai and Papazoglou (1997).

However, these ratios are also shown to decrease more rapidly than those of lowermagnitude. This penalty on structures close to the source is mitigated by the bonusof reduced *V/H* to much less than 2/3 further away.

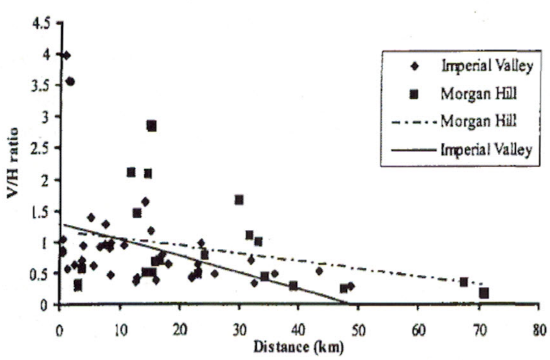


Fig. 4: V/H ratios for Imperial Valley and Morgan Hill earthquake records (Elnashai and Papazouglou, 1997)

Shrestha (2009) has produced a table, reproduced as Table 1, of some "landmark earthquakes" with significant *V/H* ratios. The very high values are not inconsistent with some of the values in Figures3 and 4at very short distances from source.

Table 1
***V/H* ratios from several "landmark earthquakes" (Shrestha, 2009)**

Event	Station (Mw)	Horl(g)	Hor2(2)	Ver(g)	V/H
Gazli, Uzbeksitan 1976	Karkyr (6.8)	0.71	0.63	1.34	1.89
Imperial valley, USA 1979	El cenro array 6 (65)	0.41	0.44	1.66	3.77
Nahhani, Canada 1985	Site 1 (6.8)	0.98	1.10	2.09	1.90
Morgan hill, USA 1984	Gilroy array#7(6.2)	0.11	0.19	0.43	2.25
Loma-prieta, USA 1989	LGPC (6.9)	0.56	0.61	0.89	1.47
Northridge, USA 1994	Arleta fire station (6.7)	0.34	0.31	0.55	1.61
Kobe, Japan 1995	Port Island (6.9)	0.31	0.28	0.56	1.79
Chi Chi, Taiwan 1999	TCU 076 (6.3)	0.11	0.12	0.26	2.07

Ground Motion Frequency Content

A vertical ground motion spectrum, when needed, is fairly commonly established by scaling of a single spectral shape, originally derived for the horizontal component of ground motion, using an average *V/H* ratio of 2/3, as originally proposed by Newmark et al. (1973). This approach assumes that all components of motion have the same frequency content.In reality, the vertical component is associated with higher frequencies, as observed in most strong-motion records – see, for instance, the horizontal and vertical components of the most frequently cited 1940 El Centro earthquake ground motion record in Figure 5. The reason for this lies in the fact that the vertical component ofmotion is mainly associated with the arrival of vertically propagating P-waves inthe epicentral region, whilst S-waves are the main cause of horizontal components. The wavelength of P-waves is shorter than that of S-waves, meaning that theformer are associated with higher frequencies. Travel path effects tend to filter low-frequency vertical vibrations and high-frequency horizontal vibrations.

Figure5 shows the acceleration response spectrum of the same 1940 El Centro ground motion. This figure confirms the higher frequency content of the vertical component of the ground motion, which results in a higher ratio of vertical to horizontal spectral accelerations in the short period range. Although the energy content over the entire frequency range of the vertical ground motion is lower than that of the horizontal component, the vertical component has most of its energy concentrated within a narrow high frequency band. Such high frequency content leads to very large response in the short period range, which often coincides with the vertical period of structures, thus causing significant response amplification, as shown in Figure6.

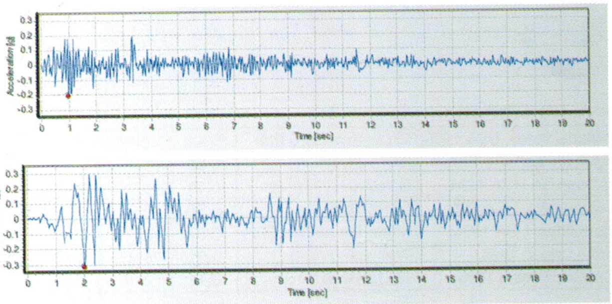


Fig. 5: Difference in frequency content of vertical (top) and horizontal (bottom) components of a ground motion record from the 1940 El Centro earthquake

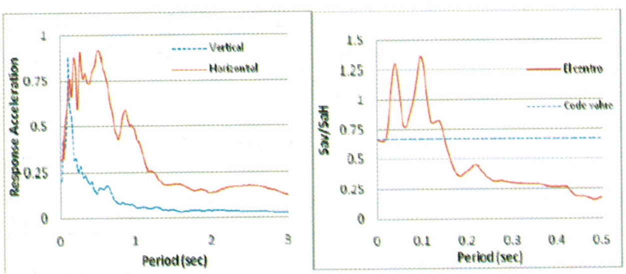


Fig. 6: Comparison of response amplifications under the vertical and horizontal components of ground motion

Time Lag Between Peak Vertical And Peak Horizontal Motion

In general, peak vertical ground motion occurs earlier than peak horizontal motion; however, near-coincidence can also occur in the time domain. In caseswhere peak verticalmotion occurs significantly before the peak horizontal motion, it may be valid to design a structure separately for the effects of vertical and horizontal ground motions.Obviously,when these two components are nearly coincidental, the consideration of their combinedeffects in design becomes necessary. Collier and Elnashai (2001) investigated the time intervalby using records from the 1979 Imperial Valley (M_w = 6.5) and the 1984 Morgan Hill (M_w = 6.3) earthquakes. They considered 32 records at various distances with similar site conditions.

Aplot of minimum time between peaks (positive or negative) versus distance is shown in Figure7. This was plotted removing records with peak accelerations less than 0.10g. The results indicate that the time interval between peak horizontal and peak vertical acceleration increases with distance from source and is influenced by earthquake magnitude. It also indicates that horizontal and vertical peak ground motion can be coincident when the distance from sourceis less than 5 km.

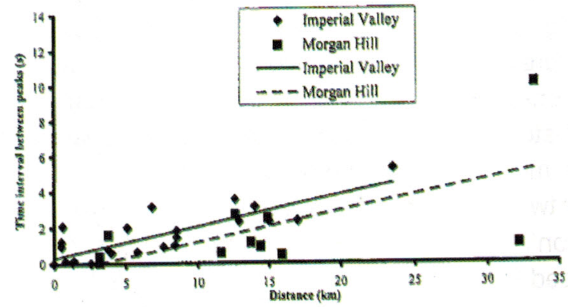


Fig. 7: Time interval between occurrence of peak ground motions in vertical and horizontal directions (Collier and Elnashai, 2001)

VERTICAL RESPONSE PERIOD

Of fundamental importance are the natural periods for vertical vibration of buildings. Buildings seem to be much stiffer in the axial than in the transverse direction and hence possessshorter periods in the vertical direction. Papadopoulou (1989)indicates that for RC moment resisting frames, the ratio of horizontal-to-vertical fundamental periods varies from 7 to *2.5* for a range of stories from 8 to 1. The results of this study are summarized in Table 2.

Table 2
Relationship between first mode horizontal and vertical periods for RC building, based on data from Papadopoulou (1989)

Number floors	Horizontal period (s)	Vertical period (s)	Ratio
1	0.1	0.040	2.50
2	0.2	0.064	3.13
3	0.3	0.082	3.66
4	0.4	0.091	4.40
5	0.5	0.099	5.05
6	0.6	0.106	5.66
7	0.7	0.114	6.14
8	0.8	0.120	6.67

Eigenvalue analysis of a 3-bay, 8-story reinforced concrete coupled wall-frame structure designed according to EC-8 (2004), and studied at Imperial College (Georgantzis, 1995), identified vertical periods of 0.075 s or less. This compares with a horizontal period of 0.534 s. These periods for reinforced concrete buildings are calculated on the basis of compressive column stiffness, while no account is taken of the reduction of this stiffness due to cracking whena column is in tension. Similar patterns are also observed for steel buildings. Papaleontiou and Roesset(1993) have studied four 3-bay steel moment resisting frames having spans of 4.5 to 8.4 m. It is noted that these structures are taken from several other studies and are not consistent in terms of their design. In particular, the 4-story and the 10-story frames are comparatively much more flexible in the horizontal direction than the other two and clearly this affects the ratio of vertical to horizontal periods. However, these examples can still be used to

indicate a very broad trend of this important ratio, as it applies to steel moment resisting frames. The relationship between vertical and horizontal periods is shown in Table 3.

Table 3
Relationship between first mode horizontal and vertical periods for steel buildings, based on data from Papaleontiou and Roesset(1993)

Number of Floor	Horizontal period (s)	Vertical period (s)	Rotio
4	1.0	0.16	6.25
10	2.22	0.20	11.10
16	1.54	0.19	8.11
20	2.27	0.25	9.08

The above values support the comments made earlier, consistent with field observations, that vertical periods are not significantly influenced by building height and lateral stiffness. This suggests that a wide range of buildings experience approximately the same dynamic amplification during vertical excitation.

EFFECT ON BUILDING STRUCTURES

Analysis of lumped parameter multi-degree-of-freedom structural models employing bilinear stiffness characteristics in tension and compression applicable to reinforcedconcretecolumn behavior by Papadopoulu (1989) indicated that strong vertical motion can lead to column tension. A study involving a range of earthquake records and multi-story buildings was undertaken. Initial gravity loads were explicitly considered in the nonlinear analysis. The study, which involved buildings having a uniform distribution of stiffness and mass with height, indicated that intermediate and the top stories are more likely to undergo tensile deformations, depending on the relationship between the building and strong-motion periods, as wellas the intensity of ground shaking. For the buildings and records examined, column tension in upper stories always occurred for peak ground accelerations exceeding 0.43 g and

for buildings having more than two stories, even though horizontal motion was not considered in the analysis. Papaleontiou and Roesset (1993) performed linear time-history analysis of the steel buildingsmentioned earlier (see section on Vertical Response Period) using the 1989 Loma Prieta record from Capitola. The analysis did not involve gravity loads. The maximum compressive (-) and tensile (+) axial forces for exterior columns, where the effect of overturning moments is larger, are shown in Table 4.

Table 4
Effect of vertical motion on axial force response of steel frames, calculated using data from Papaleontiou and Roesset (1993)

	Axial forces (kN)				Contribution vertical motion to total axialfore (%)	
No. of storeys	Roof H	Roof H+v	Ground H	Ground H +v	Roof	Ground
4	-22/+40	±110	±200	±450	72	56
10	±22	±150	±490	±850	85	42
16	-58/+80	±290	±5400	±7100	76	24
20	±49	±135	±3300	±4000	64	21

It is observed that the axial forces caused by vertical motion, having a comparable magnitude tothat of horizontal motion, are larger than the corresponding forces due to horizontal motion in most cases. This pattern is always more significant for upper floors than for lower floors. Still, for a 10-story steel frame, almost 50 per cent of the axial force variation in the ground floor exterior columns comes from contributions of vertical motion. This contribution is ignored in routine design. In interior ground floor columns the axial force variation arising from vertical motion is even more significant, since the effect of overturning moments is minimal.

STRUCTURES, MEMBERS, AND COMPONENTS SENSITIVE TO VERTICAL GROUND MOTION

Various Codes and Standards

As indicated in Section C of this report, various U.S. codes and standards, for many years, have required explicit consideration of vertical earthquake ground motion in the design of

- Horizontal cantilevers
- Horizontal prestressed concrete members

Eurocode 8 (1994)

It is stressed in the code that it is only necessary to verify the structure under vertical motion if it exhibits one or more special features, such as horizontal structural members spanning more than 20 m, horizontal cantilever members, horizontal prestressed elements or planted columns[CEN, EC8, 1994). In any case, it is considered adequate to base the analysis on a partial model of the structure involving only the elements under consideration.

Eurocode 8 (2004)

1. If a_{vg}(design ground acceleration in the vertical direction) is greater than 0.25g, the vertical component of the seismic actionshould be taken into account in the cases listed below:

- for horizontal or nearly horizontal structural members spanning 20 m or more;
- for horizontal or nearly horizontal cantilever components longer than 5 m;
- for horizontal or nearly horizontal pre-stressed components;
- for beams supporting columns;
- in base-isolated structures.

2. The analysis for determining the effects of the vertical component of the seismic action may be based on a partial model of the structure, which includes the elements on which the vertical componentis considered to act (e.g. those listed in the previous paragraph) and takes into account the stiffness of the adjacent elements.

3. The effects of the vertical component need be taken into account only for the elements under consideration (e.g. those listed in (1)above) and their directly associated supporting elements or substructures.

4. If the horizontal components of the seismic action are also relevant for these elements, all three of the following combinations may be used for the computation of the action effects:

a. $E_{Edx} + 0.30\ E_{Edy} + 0.30\ E_{Edz}$

b. $0.30\ E_{Edx} + E_{Edy} + 0.30\ E_{Edz}$

c. $0.30\ E_{Edx} + 0.30\ E_{Edy} + E_{Edz}$

Where

+implies "to be combined with";

E_{Edx} represents the action effects due to the application of the seismic action along the chosen horizontal axis *x* of the structure;

E_{Edy} represents the action effects due to the application of the same seismic action along the orthogonal horizontal axis *y* of the structure.

E_{Edz}represents the action effects due to the application of the vertical component of the design seismic action as defined in 3.2.2.5(5) and (6).

5. If non-linear static (pushover) analysis is performed, the vertical component of the seismic action may be neglected.

Summary

Based on the above, the following is a fairly comprehensive list of structures and structural components that have been identified as being particularly sensitive to vertical earthquake ground motion.

Building Structures

- horizontal or nearly horizontal structural members spanning 65 ft (20 m) or more;
- horizontal or nearly horizontal cantilever components longer than 16 ft (5 m);
- horizontal or nearly horizontal prestressed components;
- building components, excluding foundations, in which demands due to gravity loads exceed a high percentage of the nominal strength of the component;

- vertical elements of the gravity force resisting system that are discontinuous;
- base-isolated structures.

Non-Building Structures

- suspended boilers,
- long span roof structures (stadiums and high bay aircraft assembly plants),
- and horizontal cantilevers
- liquid storage tanks,
- materials storage facilities
- electric power generation facilities

It is worth noting that Eurocode 8 (2004) requires consideration of vertical earthquake ground motion only when the design ground acceleration in the vertical direction exceeds 0.25g.

SUGGESTED INTERIM MEASURE

In ASCE 7, the seismic load effect, E, is determined as the following combination of horizontal and vertical load effects;

$$E = E_h \pm E_v = \rho Q_E \pm 0.2S_{DS}D$$

The logic behind this combination is as follows. The code-specified member force of $0.2S_{DS}D$ was simplistically derived by considering a design vertical ground motion component that is 2/3rd (0.67) of the corresponding horizontal component. This resulted in a maximum vertical design spectral acceleration value of $0.67S_{DS}$. This was combined with the member force due to design horizontal ground motion component by using the "100+30" orthogonal combination rule similar to that specified in ASCE 7-16 Section 12.5.3.1(a), where 100% of the member force due to horizontal ground motion component is combined with 30% of the member force due to vertical ground motion component. 30% of $0.67S_{DS}$ produces the code-specified value of $0.2S_{DS}D$.

In the absence of a detailed study to investigate the adequacy of the code-specified design force of $0.2S_{DS}D$ for structural members subjected to vertical ground motion, it is suggested that a designer might consider incorporating the following additional expression for earthquake effect for structural members that are particularly vulnerable to vertical ground motion:

$$E = \pm 0.3E_h + 0.67S_{DS}D$$

The above combination simply considers a situation where 30% of the member force due to horizontal ground motion component is combined with 100% of the member force due to vertical ground motion component. This combination is not currently required for structures assigned to RC I through IV.

CONSIDERATION OF VERTICAL ACCELERATION IN IS 1893

IS 1893 (Part 1): 2016 Section 6.1.2 reads:

"Effects of earthquake-induced vertical shaking can be significant for overall stability analysis of structures, especially in structures (a) with large spans, and (b) those in which stability is a criterion for design. Reduction in gravity force due to vertical ground motion can be detrimental particularly in prestressed horizontal members, cantilevered members and gravity structures. Hence, special attention shall be paid to effects of vertical ground motion on prestressed or cantilevered beams, girders and slabs." However, the consideration of vertical acceleration is not mandated for any structure.

IS 1893 (Part 1) : 2016 Section 6.4.6 provides a vertical design spectrum that is essentially two-thirds of the design horizontal spectrum given in Section 6.4.2, with an upper-bound value of 2.5 used for S_a/g.

IS 1893 (Part 1) : 2016 Section 6.3.4.1 provides the same combinations of earthquake effects in two mutually perpendicular horizontal directions and the vertical direction – EL_x, EL_y, and EL_z of IS 1893 are the same as E_{Edx}, E_{Edy}, and E_{Edz}, respectively, of Eurocode 8 (2004). Section 6.3.4.2 provides an alternative to the procedure in Section 6.3.4.1, which probably is not all that sensible.

REFERENCES

1. Abrahamson, N.A. and Litehiser, J.J., 1989, "Attenuation of vertical peak acceleration,"*Bulletin of the Seismological Society of America*, Vol. 79, pp. 549-580.
2. Ambraseys, N. N.and Simpson, K.A., l995, "Prediction of vertical response spectra inEurope," Research Report ESEE-95/1, Imperial College.

3. American Society of Civil Engineers, 2005, 2010.*ASCE Standard Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05, ASCE 7-10, Reston, VA.

4. American Society of Civil Engineers, 2013.*Seismic Evaluation and Retrofit of Existing Buildings*, ASCE 41-13, Reston, VA.

5. American Society of Civil Engineers, 2005, *Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*, ASCE 43-05, Reston, VA.

6. American Society of Civil Engineers,1998.*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*, ASCE 4-98, Reston, VA.

7. Applied Technology Council, 1978.*Tentative Provisions for the Development of Seismic Regulations for Buildings*, ATC Publication ATC 3-06, NBS Special Publication 510, NSF Publication 78-8, U.S. Government Printing Office, Washington, D.C..

8. Bozorgnia, Y., and Niazi, M., 1993, "Distance scaling of vertical and horizontal response spectra of the Loma Prieta Earthquake," *Earthquake Engineering and Structural Dynamics*, Vol. 22, pp. 695-707.

9. Building Seismic Safety Council, 1985, 1988, 1991, 1994, (1997, 2000, 2003, 2009, 2015). *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings (and Other Structures)*, Washington, D.C.

10. Bureau, G.J.,1981, "Near-source peak ground accelerations," *Earthquake Notes* 52.

11. Bureau of Indian Standards, 2016.*Criteria for Earthquake Resistant Design of Structures*, IS 1893 (Part 1) : 2016 (Sixth Revision), New Delhi, India.

12. Campbell, K.W., 1982, "A study of the near source behaviour of peak vertical acceleration,"EOS 63, 1037.

13. Collier, C.J. and Elnashai, A.S., 2001,"A procedure for combining vertical andhorizontal seismic action effects,"*Journal of Earthquake Engineering*, Vol. 5, pp. 521-539.

14. Department of Defense, 2013.*Seismic Design of Buildings*, Unified facilities Criteria, UFC 3-310-04, 1 June 2013, Washington, D.C.

15. Elnashai, A.S. and Papazoglou, A.J., 1997,"Procedure and spectra for analysis of RCstructures subjected to strong vertical earthquake loads,"*Journal of Earthquake Engineering*, Vol. 1,pp. 121-155.

16. European Committee for Standardization, 2004.*Eurocode 8: Design of structures for earthquake* resistance- *Part 1 : General rules, seismic actions and rules for buildings*, EN 1998.1, Brussels.

17. Georgantzis, M.,1995."Effect of vertical motion on behaviour factors,"M.Sc. Dissertation,Imperial College, August.

18. Ghosh, S.K., and Henry, J., 2009.*2009 IBC Handbook—Structural Provisions*, International Code Council, Publications, Country Club Hills, IL.

19. International Code Council, 2000, 2003, 2006, 2009, 2012, 2015.*International Building Code*, Falls Church, VA, (2000, 2003), Country Club Hills, IL, (2006, 2009, 2012, 2015).

20. International Conference of Building Officials, 1997.*Uniform Building Code*, Whittier, CA.

21. Kunnath, S.K., Abrahamson, N., Chai, Y.H., Erduran, E., and Yilmaz, Z., 2008, "Development of guidelines for incorporation of vertical ground motion effects in seismic design of highway bridges," A Technical Report Submitted to the California Department of Transportation under Contract 59A0434, Department of Civil and Environmental Engineering Structural Engineering and Structural Mechanics, University of California at Davis.

22. Newmark, N.M.,Blume, J.A. and Kapur, K.K. ,I973, "Seismic design spectra for nuclearpower plants," Journal ofthe Power Division. Vol. 99, pp. 287-303.

23. Niazi, M. and Bozorgnia, Y., 1991, "Behaviour of near-source peak horizontal andverticalground motions over SMART-1 array, Taiwan," *Bulletin of the Seismological Society of America*, Vol.81,pp. 715-732.

24. Papadopoulou, O., 1989, "The effect of vertical excitation on reinforced concrete multistory structures," MSc. Dissertation, Imperial College.

25. Papaleontiou, C. and Roesset, J.M., 1993, "Effect of vertical accelerations on seismicresponse of frames," In Structural Dynamics - EURODYN193 (Eds. Moan et al.),Balkema, Rotterdam, pp. 19-26.

26. Shrestha, B., 2009, "Vertical ground motions and its effect on engineering structures: a state-of-the-art review," *International Seminar on Hazard Management for Sustainable Development*, Kathmandu, Nepal, November, pp. 29-30.