ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ASSESSMENT ON EFFECT OF VERTICAL COMPONENT EARTHQUAKE GROUND ACCELERATION ON BUILDING STRUCTURE

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering)

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By

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Declaration

I declare that this thesis is my original work. This thesis has not been presented for any other university and is not concurrently submitted in candidature of any other degree, and that all sources of material used for the thesis have been acknowledged.

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ABSTRACT

This thesis is concerned with effect of vertical component earthquake ground acceleration on the response of reinforced building structure. Basically the study focuses to assess the effect of vertical component earthquake ground acceleration within zone 4 and near fault area of Ethiopia. To achieve these objectives varying source distance and soil profile were used to analyze selected building in linear response spectrum analyses. The results like axial force, shear force and bending moment were used to know effect of vertical component of earthquake ground acceleration. From these, ignoring vertical component of earthquake ground acceleration at near fault causes considerable amount of internal response. The effect of vertical earthquake ground acceleration decreased as epicentral distance increased and become almost similar with effect of lateral earthquake ground acceleration at far distance. To sum up this, source distance, soil profile and magnitude of earthquake were a main factor to determine peak response of vertical earthquake component at near source.

Generally, including vertical component earthquake ground acceleration in design of reinforced concrete building structure within zone 4 of Ethiopia is not that much important compared to the lateral components. But vertical component earthquake ground acceleration should be considered if the proposed building structure is sited around near faults.

KEYWORDS: Vertical earthquake component, internal response, near fault, near source factor

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Ah(Th) = Design spectrum acceleration
a_{vg} = is the design ground acceleration
Av(Tv) = Design spectrum acceleration

CA =Seismic coefficients

C/c =Center to center

Ch =Seismic coefficient

Cv =Seismic coefficients

DL =Dead load

EQ = Earthquake

Ex = Earthquake in x-direction

Ey = Earthquake in y-direction

Fh = Inertia force produced by the horizontal

Fv = Vertical motion respectively
g = Earth gravitational acceleration

GB = Grade beam

I = Factor that depends on building importance

IB = Intermediate beam
K_W = Factors reflecting prevailing failure in structural system

LL = Live load

NA = Near-source factors

Nv = Near-source factors
q = is the behaviour factor
Sd(T) = is the design response spectra
S = is the soil factor
T = is the vibration period of a linear single-degree-of-freedom system;

Tb = is the lower limit of the period of the constant spectral acceleration branch;

TB = Tie beam
Tc = is the upper limit of the period of the constant spectral acceleration
Td = is the value defining the beginning of the constant displacement

W = total loads (self-weight, dead load and live load)

v_p = P - wave velocity

Z = Factor that depends on zones

η = is the damping correction factor with a reference value of η = 1 for 5% viscous damping

ABBREVIATION

ATC = Applied technology council

HEGA = Vertical earthquake ground acceleration

PHGA = Peak horizontal ground acceleration

PVGA = Peak vertical ground acceleration

RSAX = Response spectrum function in the x-direction

RSAY = Response spectrum function in the y-direction

RSAZ = Response spectrum function in the z-direction

VEGA = Vertical earthquake ground acceleration

1. INTRODUCTION

This thesis mainly focuses on effect of vertical component earthquake ground acceleration on reinforce concrete building structures. Basically civil engineering structures are subjected to three-dimensional earthquake ground acceleration. In the past several decades, horizontal earthquake ground acceleration has been studied extensively and considered in the design process; whereas the vertical component of earthquake ground acceleration had generally been neglected in design, and rarely studied from the hazard viewpoint for some selected structures. However, recent studies supported with increasing numbers of near-fault records, indicate that the ratio of peak vertical-to-horizontal ground acceleration can be exceed unity. Because of this, the effect and significance of vertical ground acceleration has become a concern in this study. To this end, seismic waves at different source distance, two types of soil profiles on selected sample building were considered in the numerical analysis software to show the effect and significance of vertical component earthquake ground acceleration. Structural responses like axial force, shear force, bending moment and stress at column; beams and slabs due horizontal and both horizontal and vertical earthquake ground accelerations were used as a parameters for comparison.

1.1. Background

There is renewed interest in the assessment of vertical component of earthquake ground acceleration as it has been observed that the ratio of vertical-to-horizontal peak ground acceleration can be larger in near-fault than far-fault records (Bozorgnia, 2004). The vertical component of ground motion is mainly associated with the arrival of vertically propagating compressive primary waves (P-waves), whilst secondary shear waves (S-waves) are the main cause of horizontal components. The wavelength of P-waves is shorter than that of S-waves, which means that the former are associated with higher frequencies. Normally, the vertical component of ground motion has lower energy content than the horizontal component over the frequency range. It tends to have all its energy concentrated in a narrow, high frequency band. The studies made by Akiyama and Yamada have clarified that the amount of total energy input is the sum of the energy input caused by horizontal ground motions and that caused by vertical ground motions, both of which have an independent value as disproved by (Akiyama, 1992)

Many of the current seismic design codes and damage estimation tools do not include the effect of vertical ground motions on the seismic response of structures. However, the observed damage on the columns during historical seismic events such as the 1994 Northridge earthquake and the 1995 Kobe earthquake was partly attributed to the effect of vertical motions (Broderick, 1994).

Recently more refined evaluation methods allowed having better estimations of the vertical earthquake ground motion. Now it is evident that the earlier data conducted to an underestimation of the PVGA/PHGA ratio (Abramson, 1989) and that this ratio in the vicinity of moderate to strong earthquakes can also exceed unity. All the above facts were the major one to study the effects of vertical component earthquake ground acceleration on reinforced concrete building structure.

1.2. Objective

1.2.1. General Objective

The purpose of this thesis is to assess effect of vertical component earthquake ground acceleration on reinforced concrete building structure.

1.2.2. Specific Objective

This thesis has the following specific objectives which were addressed at consecutives steps of this study:

- → To compare response of reinforced concrete building frame elements due to horizontal earthquake ground acceleration components (X and Y) against horizontal with vertical components earthquake ground acceleration (X, Y, and Z) at zone 4 of Ethiopia and near source distance.
- → To evaluate effect of vertical component of earthquake ground acceleration at various source distance and soil profile.
- → To assess the interaction effect between horizontal and vertical component of earthquake ground acceleration.

→ To create awareness about vertical component of earthquake ground acceleration to structural design personnel's.

→ To encourage about consideration of vertical component earthquake ground acceleration, if the proposed building is constructed around faults (highly suspected area for earthquake).

1.3. Methodology

The tasks used to accomplish this study and come up with relevant conclusions and recommendations were outlined as follows.

1.3.1. Literature Review and Earthquake Parameters

General review of previous studies in different literature concerning effect of vertical component earthquake ground acceleration on building structures, effect of vertical earthquake ground acceleration on different civil engineering structures, and some parameter directly or indirectly related to building structures were deeply addressed.

1.3.2. Identification of Sample Study Buildings

To accomplish these study building that contain plan and elevation irregularity was selected. The building is moment resisting frame and used for mixed use purpose only. This building was modeled for excitation source distance of less than 2km, less than 15km, and greater than15km and two type of soil profile. Comparison of effect of vertical earthquake ground acceleration response from variation of source distance and soil profile were done, and finally effect of horizontal vs horizontal and vertical earthquake ground acceleration response was compared. This helps to justify effect of vertical earthquake ground acceleration at highly suspected area.

1.3.3. Analysis of Building Models

An analysis of building frame system was done using linear response spectrum analysis. The commercial computer software package Sap2000 v14 was used for modeling and analysis of sample building. In addition to this the following programs has been applied: Microsoft Word and Microsoft excel to prepare this report.

1.3.4. Comparison of the Effect of HEGA Vs HEGA and VEGA Response

Following the structural analysis software, results were being in hand with the help of Excel sheet. After organization of all outputs, the following structural responses were used for comparison:

- Axial force, shear force and bending moment at selected interior and exterior column in each storey
- stress at selected panel

1.4. Application of the Study

The results associated with this thesis work will be pin point, to the risk associated with neglecting high magnitude and near fault vertical component of earthquake ground acceleration in design and analysis of reinforced concrete structures at zone 4 of Ethiopia and near fault area.

1.5. Scope and Limitations of the Study

The scope of the study has been limited to assess and know effect of vertical component earthquake ground acceleration on building structure which is located in zone 4 and near fault areas. This needs different parameters considered in the assessment. For this particular study, near source distance and soil profile were considered as varying parameters. As a limitation soil structure interaction and base-isolated building structure were not considered.

1.6. Thesis Organization

This thesis is divided into six chapters as follows.

Chapter 1: Gives a general introduction, background, objective, scope and limitations, methodology and thesis organization.

Chapter 2: Presents a review of previous studies in different literature concerning effect of vertical component earthquake ground acceleration on building structures, effect of vertical earthquake ground acceleration on different civil engineering structures and some parameters directly or indirectly related to earthquake ground acceleration.

Chapter 3: Presents linear analytical modeling of selected building subjected to earthquake ground acceleration with different source distance and soil profile.

Chapter 4: In this chapter mainly organization and comparison of results obtained from effect of lateral and orthogonal earthquake ground acceleration in columns, beams and slabs were performed.

Chapter 5: The effect of vertical earthquake ground acceleration on: axial force, shear force, bending moment and stress demand in columns, beams and slabs were presented and discussed.

Chapter 6: The main conclusion and recommendations that could be drawn from the current study were summarized, and also suggestions for future studies were outlined.

2. LITERATURE REVIEW

2.1. Earthquake

Earthquake is a shaking of the earth's surface caused by rapid movement of the earth's rocky outer layer. Earthquakes occur when energy stored within the earth, usually in the form of strain energy in rocks, and suddenly releases. This energy is transmitted to the surface of the earth by earthquake waves. Most earthquakes are caused by the sudden slip along geologic faults. The faults are slip-up because of movement of the earth's tectonic plates. This type of earthquake is referred as tectonic earthquakes. Other than tectonic earthquake, there are many different types of earthquakes based on the way they are generated like volcanic earthquake, collapse earthquake and explosion earthquake (Charles, 2002). These entire earth quakes have both horizontal and vertical component at the beginning.

Earthquake force on building, generated by the inertia of building as they dynamically responds to ground motion. The dynamic nature of the response makes earthquake loadings markedly different from other building loads.

2.1.1. Cause of Earthquake

The surface of the earth is in continuous slow motion. This is plate tectonics, the motion of rigid plates at the surface of the Earth in response to flow of rock within the Earth. The plates cover the entire surface of the globe. Since they are all moving they rub against each other in some places (like the San Andreas Fault in California), sink beneath each other (like the Peru-Chile Trench along the western border of South America), or spread apart from each other (like the Mid-Atlantic Ridge). At such places the motion is not smooth, the plates are stuck together at the edges but the rest of each plate is continuing to move, so the rocks along the edges are distorted (what we call "strain"). As the motion continues, the strain builds up to the point where the rock cannot withstand any more bending. With a motion, the rock breaks and the two sides move. In short, earthquakes are caused by faulting, a sudden lateral or vertical movement of rock along a rupture (break) surface (Dr.Gerard, 1998). These cause both horizontal and vertical component of seismic waves in the earth.

2.1.2. Earthquake Resistant Design Philosophy

Severity of ground shaking at a given location during an earthquake can be minor, moderate or strong. Thus relatively speaking, minor shaking occurs frequently; moderate shaking occurs occasionally and strong shaking occurs rarely. For instance, on average annually about 800 earthquakes of magnitude 5.0-5.9 occur in the world while about 18 for magnitude range 7.0-7.9. So we should design and construct a building to resist that rare earthquake shaking that may come only once in 500 years or even once in 2000 years, even though the life of the building may be 50 or 100 years (Charles C. W., 2003).

Structural engineers do not attempt to make earthquake proof buildings that will not get damaged even during the rare but strong earthquake since designing such kind of buildings will be too expensive. Instead the engineering intention is to make buildings that resist the effects of ground shaking, although they may get damaged severely but would not collapse during the strong earthquake. These kinds of buildings will assure safety of people, and by this means a disaster is avoided. This is a major objective of seismic design codes and standards. The primary objective of earthquake resistant design is to prevent building collapse during earthquakes thus minimizing the risk of death or injury to people in or around those buildings.

2.1.3. Type of Earthquake (Seismic) Waves

Among the many types of seismic waves, one can make a broad distinction between body waves and surface waves (Peter, 2009).

Body waves travel through the interior of the earth whereas surface waves travel across the surface of the earth. Surface waves decay more slowly with distance than do body waves, which travel in three directions.

Body waves: Includes Primary (P-waves) and Secondary (S-waves) waves.

Primary waves (P-waves) are compressional waves that are longitudinal in nature and mainly causes of vertical component of earthquake ground acceleration (VEGA). P-waves are pressure waves that travel faster than other waves through the earth to arrive at seismograph stations firstly, hence the name

"Primary". These waves can travel through any type of material, including fluids, and can travel at nearly twice the speed of S-waves. In air, they take the form of sound waves; hence they travel at the speed of sound. Typical speeds are 330 m/s in air, 1450 m/s in water and about 5000 m/s in granite.

2.1.4. Earthquake Ground Acceleration Components

There are two horizontal, vertical and rotational components of earthquake ground motion. From these vertical and rotational components of ground motion are almost always ignored in analysis and design of building structure. But this is not always true according to different parameters (Graizer, 2006).

2.2. Vertical Component of Earthquake Ground Acceleration

In codes, vertical earthquake motion is represented by scaling a single design spectrum derived for horizontal components. This procedure was devised by (Newmark, 1974) and has been widely used. Generally, the scaling factor, the vertical-to-horizontal ratio, has been taken as $\frac{2}{3}$. The weakness of this procedure is the assumption that horizontal and vertical components have the same frequency content which does not reflect either the frequency content of ground motion or the actual structural responses of building structures. In addition, the ratio of peak vertical-to-horizontal ground accelerations (V/H) may often exceed $\frac{2}{3}$. Abramson, N.A. suggests that the ratio of vertical to horizontal (V/H) earthquake ground acceleration exceed $\frac{2}{3}$ for near faults (Abramson, 1989).

As the wavelength of P-waves is shorter than that of S-waves, the vertical component of ground motion has much higher frequency content than the horizontal component in the near field earthquake. Although the energy content over the frequency range of the vertical ground motion is lower than that of the horizontal motion, the energy tends to be concentrated in a narrow, high frequency band. Such high frequency content leads to large amplifications in the short period range, which often coincide with the vertical period of reinforced concrete structures (Elnashai, 2004).

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 source amplification of the vertical response.

2.2.1. Frequency and Time Period Content of VEGA

Among various guides of earthquake ground motions, the maximum amplitude value and frequency content have been regarded as the most important guides for earthquake engineering. Also the fact that the duration of earthquake ground motion differs from site to site even during the same earthquake.

The wave-length of P- waves are shorter than the S-wave, and frequency content of the vertical component of the ground motion is higher than the horizontal component at near source. Although the content over the frequency range of the vertical ground motion is lower than that of the horizontal component, it has tendency to concentrate all its energy in narrow high frequency band. Therefore such high frequency content leads to largest response in short period range.

2.2.2. Vertical-to-Horizontal Response Spectral Ratios

The existing models for predicting the V/H ratio of response spectral ordinates can be grouped into two categories:

- Codes and regulations, which present standard ratios that usually vary only as a function of site classification; and
- Independent predictions of the vertical and horizontal components of motion, which allow the median V/H ratio to be calculated for a given scenario;

The significance of the vertical component of ground motion is often characterized by the vertical -to-horizontal peak ground acceleration (V/H) ratio. Many codes suggest scaling of a single spectral shape, originally derived for the horizontal component using an average V/H ratio of 2/3. This procedure was originally proposed by (Newmark, 1974). As a result, all components of motion have the same frequency content in almost all design codes. The frequency content, however, is demonstrably different, as discussed above.

2.2.3. Time Lag between Peak Vertical and Peak Horizontal Acceleration

One of the important features of the ground motion is the relationship between the arrival times of peak vertical motion with the peak horizontal motion. In general peak vertical ground motion occurs earlier than peak horizontal motion (Peak vertical acceleration occurs 1 sec earlier than the peak horizontal ground motion), (Elnashai and Collier, 2001). This is due to arrival time of seismic waves. The vertical component of ground motion is mainly associated with the arrival of vertically propagating compressive primary waves (P-waves), whilst secondary shear waves (S-waves) are the main cause of horizontal components. P-waves are faster than other waves through the earth to arrive at seismograph stations firstly. Primary waves can travel at nearly twice the speed of S-waves as described by (Peter, 2009).

2.2.4. Vertical Component of Earthquake Response Period (T)

Fundamental natural period ‘T’ is an essential property of a building. Any changes made to the building will change its ‘T’. The value of ‘T’ depends on the building flexibility and mass; more the flexibility, the longer is the ‘T’’. On the contrary, low- to medium-rise buildings generally have shorter ‘T’ (Murty.C.V.R, 2004).

Buildings seem to be much stiffer in the axial than in the transverse direction and hence have shorter periods in the vertical direction. Papazoglou indicates that for reinforced concrete moment resisting frames the ratio of vertical-to horizontal fundamental periods varies from 7 to 2.5 for a range of storeys from 8 to 1 (Popazaouglou, 1996).

2.3. Vertical Ground Acceleration According to Some Building Codes

2.3.1. Some Seismic Code Considerations for the VEGA

Several seismic design codes include recommendations related to the effects of the vertical component of the ground acceleration. A brief summary of the analytical expressions proposed in some building codes to consider the influence of the vertical component of the ground motion on structural response (both static and dynamic methods) is compiled by (Luis, 2004) as follows.

Table 1: Summary of expressions in some codes to consider vertical ground motion

Country	Reference	Static method	Dynamic method
USA	UBC 1997	Fv = 0.7*Cv*I*DL - for cantilever on 3 & 4 seismic zone	Av(Tv) = 2*Ah(Th)/3
	FEMA-368-200	Fv= 0.2* DL- for beam, cantilever and pre stressed structures	Av(Tv) = 2*Ah(Th)/3
Mexico	RCEG 1990	Fv= 2/3 * Ch*W	Av(Tv) = 2*Ah(Th)/3
	RCMP 1999	Fv = 2/3* Ch* W	Av(Tv) = Cv/h=2/3, Cv/h = 3/4 - for industries
Ecuador	CEC, 2000	Fv = 2*Ch*I*S*W/3	Av(Tv) = 2*Ah(Th)/3
Europe	CEN, 1994	Fv = Cv/h(Tv)*W	Av(Tv) = Cv/h *Av/h(Th)
		Cv/h =0.7, Tv <0	
		Cv/h =0.4, Tv > 0-5	
		Cv/h = 11/14- 4TV/7 , 0.15 < Tv <0.5	
Japan	AJJ, 1990	Fv = 0.5*Cv*W	Av(Tv) = 0.5*Ah(Th)

Where:

- Fh and Fv = Inertia force produced by the horizontal and vertical motion respectively
- Ch and Cv = Seismic coefficient
- Ah(Th) and Av(Tv) = Design spectrum acceleration
- I and Z = factor that depends on building importance and zones
- W = total loads (self-weight, dead load and live load)

Regarding the recommended provisions presented in Table -1, it is observed that the seismic intensity for vertical design is obtained, as a rule, applying a factor to the seismic intensity for horizontal design. However, some disagreements are observed; for example, some of them only consider the effects of the vertical seismic component in structural elements that are supposedly sensitive to such component. Others suppose that the problem is solved increasing the gravity loads with static loads equivalent to those that would be produced by the vertical acceleration of the ground motion. Few codes clarify the need of including in the analysis vertical degrees of freedom that simulate the load distributed with the aim of estimating vertical periods and to make rational modal combinations.

2.3.2. VEGA According to Ethiopian Building Code of Standard (EBCS-8)

EBCS-8 at present recommends the use of vertical acceleration, in its spectral form, for the verification of structural integrity only in special cases of structural configurations that include: horizontal or nearly horizontal structural members spanning 20 m or more; horizontal or nearly horizontal cantilever components longer than 5 m; horizontal or nearly horizontal pre-stressed components; beams supporting columns; and base-isolated structures.

If a_{vg} is greater than 0.25g (2.5m/s^2) the vertical component of the seismic action should be taken into account in the cases listed above:

In such cases the EBCS-8 Code recommends that vertical component of the seismic action, the design spectrum is given by the following expressions:

$$S_d(T) = a_{vg} * s * [\frac{2}{3} + \frac{T}{Tb} * (\frac{2.5}{q} - \frac{2}{3})] \text{ ------------ } 0 \leq T \leq Tb$$

$$S_d(T) = a_{vg} * s * \lambda * \frac{2.5}{q} \text{ ---------------------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \left\{ \begin{matrix} = \mathrm{avg} * \mathrm{s} * \frac{2.5}{q} \left[\frac{Tc}{T}\right] \\ \geq B * avg \end{matrix} \right\} \text{ ----------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \left\{ \begin{matrix} = \mathrm{avg} * \mathrm{s} * \frac{2.5}{q} \left[\frac{Tc * Td}{T * T}\right] \\ \geq B * avg \end{matrix} \right\} \text{ ---------- ---} Tb \leq T$$

Where:

- Sd(T) = Is the design response spectra
- T= is the vibration period of a linear single-degree-of-freedom system;

- a_{vg} = is the design ground acceleration
- B = is found in the National Annex. The recommended value for B is 0.2
- q = is the behaviour factor; for the vertical component of the seismic action a behaviour factor q up to 1.5 should generally be adopted for all materials and structural systems.
- Tb = is the lower limit of the period of the constant spectral acceleration branch;
- Tc = is the upper limit of the period of the constant spectral acceleration branch;
- Td = is the value defining the beginning of the constant displacement response range of the spectrum
- S = is the soil factor and taken as 1
- λ = is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping

Until now the Code does not consider the need to take into account vertical ground acceleration component for building analysis and design in the near-field of medium to large earthquakes.

2.3.3. Vertical Ground Acceleration According to Eurocode-8 (Eu-8)

EUROCODE-8 at present recommends the use of vertical acceleration, in its spectral form, for the verification of structural integrity only in special cases of structural configurations that include long cantilevering elements, planted columns, large eccentricities or other configurations which enhance vulnerability to vertical motions. Generally the recommendation of both Ethiopian building code standard and Eurocode about effect of vertical earthquake ground acceleration on building structure is almost similar.

It is common to obtain the vertical design coefficient by multiplying the corresponding horizontal coefficient by a factor, regardless of the differences between the dominant periods that characterize each component. This factor is usually obtained from the ratio of vertical to horizontal peak ground accelerations. However, it is not considered that this quotient is highly dependent of the period and not constant, as most of the codes propose. To omit this observation implies to overestimate the effects of the vertical component, which does not necessarily lead to a higher level of structural safety, in particular if a possible change in the collapse mechanism is produced if beams are made stronger without checking their interaction with columns.

Other common consideration in most of codes is not to permit the reduction by inelastic behavior. In addition, on the values of the percentage of critical damping (ξ) associated with the vertical vibration may not be available.

2.4. Structural Damage Related to Vertical Earthquake Ground Acceleration

During the recent earthquakes, the vertical earthquake component of the ground motion found to be exceeding the horizontal component, which directly contradicts with some current code provisions that assumes the value of the vertical ground motion to be $\frac{2}{3}$ of the horizontal component. Seismic design of the structure without the consideration of the vertical ground motion component may result in unquantifiable risk from the collapse, especially those structure constructed to near the fault. However there seems to be no agreement as to the importance on damage due to vertical motions, and little that has been learned from the recent earthquake in Loma-Prietto, Northridge, or Kobe which indicates conclusively that damage to structures was predominantly by vertical motions (Sherestha, 2009).

Vertical earthquake motion does not only influence ductility and shear capacity supply relevant to transverse response. Compressive failure can also occur in vertical bearing elements such as walls and columns. (Masek, 1994).

Damaging of vertical earthquake ground motion

Figure 1: Shear- bond failure in 3rd story of Holiday Inn Hotel in Van Nus

Figure 2: Second story collapse of Kaiser permanent building in Balboa Boulevard at Northridge

Figure 3: Shear - compression failure in column of cast-in -place garage in Sherman Oaks

Figure 4: Column brittle failures observed in a steel Mega-truss at Ashiya-hama

2.5. Effect of VEGA on Selected Civil Engineering Structures

2.5.1. Effect of VEGA on Longer Span frame system

To examine the effect of vertical earthquake component on the various span frames in dynamic analysis, first the building with three components X, Y and Z is studied. At next step, the same earthquake has been studied without considering the effect of vertical earthquake component and behavior of structure.

In each models, moment, shear, and axial force of long span frames caused by horizontal component and exerting vertical and horizontal components, have been studied. The result is shown both cases in two models. The values indicate that the vertical earthquake component is very effective on increasing the mid-span moment. However, this component does not have much impact on increasing the shear of span. Also, the component increases the axial force in long span frame in adjacent columns.

The studies show that by applying the vertical earthquake component, the moment of the middle long span frame is increased around 50%, shear values beams are almost constant and axial force increases about 40% and column and maximum drift of the middle span has been increased about 30%. Also the values of moment, axial force and the drift are increased by increasing the length of long span beams (Hosein, 2011).

2.5.2. Effect of VEGA on Bridges Structures

Most of bridge designers suppose that the effect of vertical component of earthquake ground acceleration doesn’t have any important effect on bridges.

This may be due to the point that the codes do not point to these effects directly. However, some codes try to consider the effect of vertical component of earthquake ground acceleration by increasing or decreasing the amount of dead load. This method has been used in seismic guidance of AASHTO (AASHTO, 1996). Load factors of 0.8 and 1.2 of dead load (increasing and decreasing of 20% of dead load) are considered by this code. However, these factors do not consider the effects of earthquake magnitude, fault distance and bridge types.

A study of bridges by Button, Cronin, and Mayes provides further insight into the effect of vertical ground motion (Button, 2002). The authors performed dynamic analyses on a group of representative highway bridges and recommended when vertical motion should be clearly included in design, when the effects can be adequate when vertical motions may be safely ignored. For comparison among the bridges they used a ratio of the difference between three-component and two component response divided by the dead load response. This ratio decreased as fault distance increased. Soil site conditions and a magnitude 7.5 event produced the highest ratios for pier axial force for all distances. For distances less than 10 km rock site conditions produced the highest ratios for deck shear at the pier and moment at the mid-span. The authors discovered that the early arrival of strong vertical ground motion has little effect on the bridge response compared to the strong vertical ground motion arriving at the same time as the horizontal ground motion. They learned this from time history analyses. The authors came to many important conclusions:

- the impact of vertical ground motion increases greatly as the bridge site gets closer to the fault;
- the horizontal response is not significantly influenced by the vertical component of motion; and
- Bridges with the greatest percentage of modal mass attributed to periods near the peak of the vertical response spectrum are affected the most by vertical ground motions.
- They recommend ignoring the effect of vertical ground motion for bridges located more than 50 km away from an active fault.

- For bridges at an intermediate distance, a site specific study may be performed to determine the effects of the vertical ground motion.

2.5.3. Effect of VEGA on Base-Isolated Structures

The insertion of an isolation system at the base of a structure produces (in comparison with the fixed-base structure) a large amount of its horizontal deformability, which can give rise to an amplification of the structural response during near-fault earthquakes. Moreover, this amplification can be emphasized due the combination of the actions produced by the horizontal and vertical components of the ground motion; in addition, in the vertical direction a base-isolated structure behaves like a fixed-base and exhibits a low damping capacity.

For base isolated reinforced concrete framed superstructure, some studies proved that the vertical component of the ground motion can induce a significant variation of the axial load in the columns: when assuming increasing α_{PGA} values, the axial load can reach even the ultimate load in compression (with brittle failure) or in tension. In all the examined cases, when considering the recorded vertical ground motion, the compressive load exceeded that corresponding to the balanced failure, at all the storeys, except at the top storey; on the contrary, the ductility demand for all the reinforced concrete frame members exhibited a small variation in comparison with the case in which the vertical component of motion was neglected. Lastly, the vertical component of motion, even assuming increasing values of αPGA, did not produce unfavorable effects on the ductility demand in all the cases (in some cases produced even a favorable effect), because this depends on how the combined characteristics of the horizontal and vertical components of the ground motion interact with the dynamic properties of the structure (Vulcano, 2004).

2.5.4. Effect of VEGA on Water Tank

The vertical component of the ground acceleration induces a hydrodynamic wall pressure in addition to that induced by the horizontal component. The pressure is uniformly distributed in the circumferential direction and varies in the axial direction as well as with respect to time. If the peak response of the vertical component of an earthquake occurred simultaneously and in the same direction with the peak response of the horizontal component, it may significantly increase the exerted hydrodynamic forces on the tank wall. Therefore vertical acceleration shall be taken into consideration on water tank design. Seismic load case must include the horizontal and vertical components of earthquakes, and the load combinations could be (100% Horizontal + 40% Vertical) and (40% Horizontal + 100% Vertical) to be safe (wang, 2012).

2.5.5. Effect of VEGA on Stair Case

Recent research has shown that the presence of staircases has altered the mode shapes of the structure making, in several cases, the torsional mode shape the fundamental one of the structure (Wuitikii, 2010). Furthermore, in case of frame type of structures the columns and the beams which are close to the staircases are the most vulnerable part of the structure, while shear is the most prominent cause of failure in case of short columns that are formed in the staircases area.

Iohannis A. Tegos model and analyze the slab and the staircase with 3 node shell elements with six degrees of freedom per node. With these elements the behavior of slabs and staircases can be modeled, either as 3D structures taking the bending into account or as 2D structures acting as membranes. In order to get more accurate results the mesh near the supports of the slabs and staircases, i.e., near the columns, as well as at the corners of shear walls and of the cores of the shear walls was refined. Finally he concludes that influence of the vertical component of earthquake is crucial in staircases with a free landing as well as to the helical ones (Tegos, 2013).

3. MODELING AND ANALYSES OF BUILDING FRAME

3.1. General

To assess the effect of vertical component earthquake ground acceleration on selected building structure, first the building with two earthquake components (X and Y) was modeled. At next step, the same earthquake on the same building has been modeled with considering the effect of vertical earthquake component in addition to gravity loads. In the model, the following responses of structures like axial force, shear force, bending moment and stress were used to evaluate effect of vertical component earthquake ground acceleration on building element.

3.2. Description of Building Model

The floor plan, column spacing and other parameters were chosen intentionally, so that the selected study building structures can provide a critical structural layout to assess the effect of vertical component earthquake ground acceleration on reinforced concrete building structures. The building was assumed to be located on deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. The columns were assumed fixed at the base because they were supported at the foundation pad. The dimension of cross sections (columns and beams) was varied according to the coming load. Totally each cross section of the element is checked and designed using Euro code 2, 2004 to know:

- analysis and design section match for all concrete frame and
- all concrete frames elements passed the stress/ capacity check

The floor plan of building which can represent the building type and structural system was displayed in figure-5 below. During determination of building layout, plan and elevation irregularity are considered. Property/ Stiffness modification factors were properly used to consider stiffness of cracked section in frame sections.

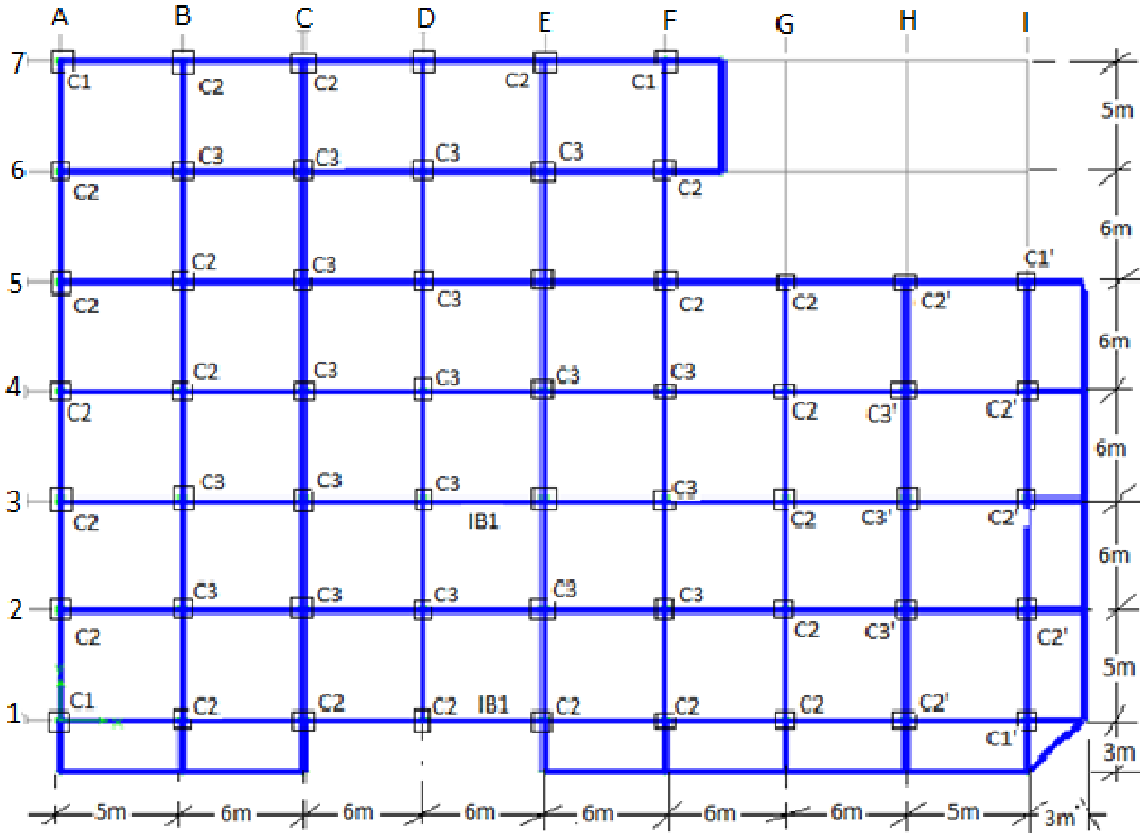


Figure 5: Floor plan for selected sample building model

Detailed descriptions of the structural elements were summarized in Table-2 that contains beam and column dimensions and others which were necessary for modeling.

Table 2: Description of building structural element

Beams		
Grade Beam (GB) = 45x30cm		Intermediate Beam 2 (IB2) =60x50cm
Intermediate Beam 1 (IB1) = 70x60cm		Top Tie Beam (TB) = 40x30cm
Columns		**Axis and assigned column**
C1 =80 x 80cm	C1' = 70 x 70cm	Axis A1 and F7 = C1
C2 = 85 x 90cm	C2' = 80 x 85cm	Axis F3 and B2 =C3
C3 = 90 x 90cm	C3' = 80 x 80cm	Axis I2 = C2'
Slab thickness =15cm		Axis A3 and C7 = C2
All c/c dimension except the edge pane is 6m		
Edge pane c/c dimension is 5m		
Cantilever is 3m		

3.1.1. Material Properties

Normal-weight concrete with a characteristic cube compressive strength of 30MPa and characteristic yield strength of 400MPa for reinforcement was used in modeling.

3.2.1. Loading

Loading on the structure consists of uniform live load of 3kN/m^2 and imposed dead load of 1.5kN/m^2 from partition and finishing. Permanent dead load of the structure was computed by the software using unit weight of concrete.

Earthquake force for a bed rock acceleration that corresponds to very high seismic zone of Ethiopia (Zone 4) was used. Earthquake ground acceleration response function was applied in two horizontal components (X & Y) vs in two horizontal and one vertical component (X, Y & Z).

3.2.2. Load Combination

Load combinations used in the study were listed below:

Combo1 = 1.35DL

Combo 2 = 1.35 DL + 1.5 LL

Combo 3 = DL + 0.3LL + Ex + 0.3Ey

Combo 4 = DL + 0.3LL + Ex - 0.3Ey

Combo 5 = DL + 0.3LL - Ex - 0.3Ey

Combo 6 = DL + 0.3LL - Ex + 0.3Ey

Combo 7 = DL + 0.3LL + Ey + 0.3Ex

Combo 8 = DL + 0.3LL + Ey - 0.3Ex

Combo 9 = DL + 0.3LL - Ey - 0.3Ex

Combo 10 = DL + 0.3LL - Ey + 0.3Ex

Combo 11= Envelop

The above load combination was used for gravity and horizontal seismic loads. For gravity, horizontal and vertical seismic load case; it was added vertical earthquake to each seismic load combination.

3.3. Determination of Design Response Spectrum Function

A response spectrum function is simply a list of period versus spectral acceleration values in the function that is assumed to be normalized. The units are associated with a scale factor that multiplies the function and that is specified when the response spectrum analysis case is defined. Linear dynamic response spectrum analyses were performed on sample building model. The response spectrum is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, which are forced into motion by the same base vibration or shock.

The applied ground acceleration was assumed to be uniform; hence it is the same at the base of each restraint element. The response-spectrum resulting from acceleration loads are always relative to the ground motion.

Development of Horizontal Design Response Spectrum Function

- select ground types according to Table 3.1 on EBCS EN 1998-1-1:2013
- Determination of design spectrum, $S_d(T)$, using the following formula

$$S_d(T) = a_g * s * [\frac{2}{3} + \frac{T}{Tb} * (\frac{2.5}{q} - \frac{2}{3})] \text{ ------------ } 0 \leq T \leq Tb$$

$$S_d(T) = a_g * s * \eta * \frac{2.5}{q} \text{ ---------------------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \begin{Bmatrix} = ag * s * \frac{2.5}{q} \left[\frac{Tc}{T}\right] \\ \geq B * avg \end{Bmatrix} \text{ ----------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \begin{Bmatrix} = ag * s * \frac{2.5}{q} \left[\frac{Tc * Td}{T * T}\right] \\ \geq B * ag \end{Bmatrix} \text{ ---------- ---} Tb \leq T$$

Where:

- Sd(T) = Is the design response spectra
- T= is the vibration period of a linear single-degree-of-freedom system;
- a_g = is the design ground acceleration

$ag = r^* a_{gR}$

- r = importance factor, a_{gR} = reference peak ground acceleration
- B = is found in the National Annex. The recommended value for B is 0.2
- q = is the behaviour factor;

  $q = q_o * K_w \geq 1.5$

  $q_o = 3*au/a1 = 3*1.3 = 3.9$; Kw =1

  $q = 3.9*1 = 3.9 \geq 1.5$ --------------------------ok!
- Tb = is the lower limit of the period of the constant spectral acceleration branch;
- Tc = is the upper limit of the period of the constant spectral acceleration branch;
- Td = is the value defining the beginning of the constant displacement response range of the spectrum
- S = is the soil factor and taken as 1
- η = is the damping correction factor with a reference value of η = 1 for 5% viscous damping

Table 3: For ground type “C”, important parameters horizontal design spectrum vs spectral period

Ground type	S	TB(s)	TC(s)	TD(s)
C	1.15	0.2	0.6	2

Table 4 : Horizontal Design Response Spectrum

Time Period	Acceleration	Time Period	Acceleration	Time Period	Acceleration	Time Period	Acceleration
0	0.153333	1	0.088462	2	0.044231	3	0.04
0.05	0.151859	1.05	0.084249	2.05	0.042099	3.05	0.04
0.1	0.150385	1.1	0.08042	2.1	0.040119	3.1	0.04
0.15	0.14891	1.15	0.076923	2.15	0.04	3.15	0.04
0.2	0.147436	1.2	0.073718	2.2	0.04	3.2	0.04
0.25	0.147436	1.25	0.070769	2.25	0.04	3.25	0.04
0.3	0.147436	1.3	0.068047	2.3	0.04	3.3	0.04
0.35	0.147436	1.35	0.065527	2.35	0.04	3.35	0.04
0.4	0.147436	1.4	0.063187	2.4	0.04	3.4	0.04
0.45	0.147436	1.45	0.061008	2.45	0.04	3.45	0.04
0.5	0.147436	1.5	0.058974	2.5	0.04	3.5	0.04
0.55	0.147436	1.55	0.057072	2.55	0.04	3.55	0.04

0.6	0.147436	1.6	0.055288	2.6	0.04	3.6	0.04
0.65	0.136095	1.65	0.053613	2.65	0.04	3.65	0.04
0.7	0.126374	1.7	0.052036	2.7	0.04	3.7	0.04
0.75	0.117949	1.75	0.050549	2.75	0.04	3.75	0.04
0.8	0.110577	1.8	0.049145	2.8	0.04	3.8	0.04
0.85	0.104072	1.85	0.047817	2.85	0.04	3.85	0.04
0.9	0.098291	1.9	0.046559	2.9	0.04	3.9	0.04
0.95	0.093117	1.95	0.045365	2.95	0.04	3.95	0.04
						4	0.04

Development of Vertical Design Response Spectrum Function

- Determination of vertical design spectrum, $S_d(T)$, using the following formula

$$S_d(T) = a_{gv}*s*[\frac{2}{3} + \frac{T}{Tb} * (\frac{2.5}{q} - \frac{2}{3})] \text{ ------------ } 0 \leq T \leq Tb$$

$$S_d\,(T) = a_{gv}*s*\eta*\frac{2.5}{q} \text{ ---------------------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \begin{cases} = \mathrm{avg} * \mathrm{s} * \frac{2.5}{q}\left[\frac{Tc}{T}\right] \\ \geq B * avg \end{cases} \text{ ----------------- } Tb \leq T \leq Tc$$

$$S_d(T) = \begin{cases} = \mathrm{agv} * \mathrm{s} * \frac{2.5}{q}\left[\frac{Tc*Td}{T*T}\right] \\ \geq B * avg \end{cases} \text{ ---------- ---} Tb \leq T$$

Where:

- Sd(T) = Is the design response spectra
- T= is the vibration period of a linear single-degree-of-freedom system;
- a_{gv} = is the design ground acceleration

$a_{gv} = 0.9 * a_g$, $a_g = r* a_{gR}$

- r = importance factor, a_{gR} = reference peak ground acceleration
- B = is found in the National Annex. The recommended value for B is 0.2
- q = is the behaviour factor; for the vertical component of the seismic action a behaviour factor q up to 1.5 should generally be adopted for all materials and structural systems
- Tb = is the lower limit of the period of the constant spectral acceleration branch;
- Tc = is the upper limit of the period of the constant spectral acceleration branch;

- Td = is the value defining the beginning of the constant displacement response range of the spectrum
- S = is the soil factor and taken as 1
- η = is the damping correction factor with a reference value of η = 1 for 5% viscous damping

Table 5: Recommended values of parameters for vertical elastic response spectra at type -1spectrum

Ground type	S	TB(s)	TC(s)	TD(s)
A, B,C,D,E	1	0.05	0.0.15	1

Table 6: Vertical Design Response Spectrum

Time Period	Acceleration	Time Period	Acceleration	Time Period	Acceleration	Time Period	Acceleration
0	0.12	1	0.045	2	0.036	3	0.036
0.05	0.3	1.05	0.040816	2.05	0.036	3.05	0.036
0.1	0.3	1.1	0.03719	2.1	0.036	3.1	0.036
0.15	0.3	1.15	0.036	2.15	0.036	3.15	0.036
0.2	0.225	1.2	0.036	2.2	0.036	3.2	0.036
0.25	0.18	1.25	0.036	2.25	0.036	3.25	0.036
0.3	0.15	1.3	0.036	2.3	0.036	3.3	0.036
0.35	0.128571	1.35	0.036	2.35	0.036	3.35	0.036
0.4	0.1125	1.4	0.036	2.4	0.036	3.4	0.036
0.45	0.1	1.45	0.036	2.45	0.036	3.45	0.036
0.5	0.09	1.5	0.036	2.5	0.036	3.5	0.036
0.55	0.081818	1.55	0.036	2.55	0.036	3.55	0.036
0.6	0.075	1.6	0.036	2.6	0.036	3.6	0.036
0.65	0.069231	1.65	0.036	2.65	0.036	3.65	0.036
0.7	0.064286	1.7	0.036	2.7	0.036	3.7	0.036
0.75	0.06	1.75	0.036	2.75	0.036	3.75	0.036
0.8	0.05625	1.8	0.036	2.8	0.036	3.8	0.036
0.85	0.052941	1.85	0.036	2.85	0.036	3.85	0.036
0.9	0.05	1.9	0.036	2.9	0.036	3.9	0.036
0.95	0.047368	1.95	0.036	2.95	0.036	3.95	0.036
						4	0.036

3.3.1. Design response spectrum for HEGA and VEGA

Design spectrum in Figure 6 shows visible difference between horizontal and vertical earthquake ground acceleration with variation of source distance and time period.

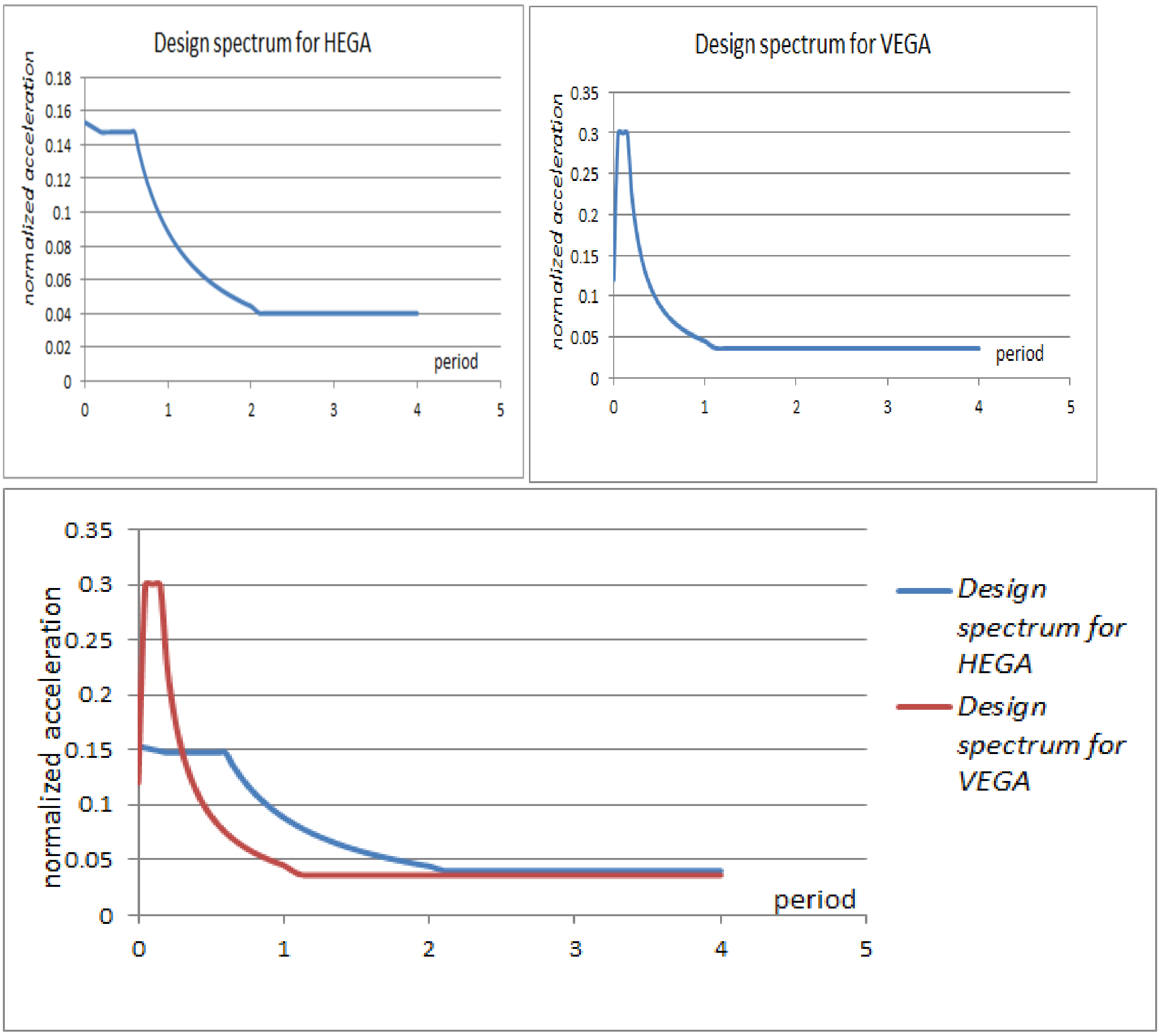


Figure 6: Design response spectrum for horizontal and vertical earthquake ground acceleration

3.4. Earthquake near Source Factor and Seismic Coefficient

For each earthquake hazard level and source distance, the structure is assigned a seismic coefficient according to ATC -40. The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of source on the surface (source projection of the fault plane). The largest value of the near source factor considering all sources shall be used in design according to applied technology council (ATC, 1996).The near source factor may be based on the linear interpolation of values for distance other than shown in Table 9.

To determine the Structural Behavior Type (SBT), this is not independent of the expected ground motion at the site. Normally structural behavior type depends on both engineering judgment and shaking duration expected on the site. Near-fault sites are categorized as short shaking duration while far-field sites are categorized as long shaking duration.

Table 7: Structural behavior type (Table 8-4 in ATC-40)

Shaking duration	Essential new building
Short	Type A
Long	Type B

Determine the soil profile type from Table - 8. Formulas for calculating the average shear wave velocity was provided in ATC-40.

Table 8: Soil profile type (Table 4-3 in ATC-40)

Soil profile type	soil profile near and generic description	shear wave velocity (m/s)
SA	Hard rock	vs > 1524
SB	Rock	762 < vs < 1524
SC	very dense soil and soft rock	366 < vs < 762
SD	Stiff soil	366 < vs < 762
SE	Soft soil	183 < vs < 366

Determine Near-Source Factors from Table - 9
Table 9: Near-source factors, NA and Nv (Table 4-5 in ATC-40)

seismic source type	Distance to known seismic source							
	≤ 2km		5km		10km		≥ 15km	
	NA	Nv	NA	Nv	NA	Nv	NA	Nv
A	1.5	2	1.2	1.6	1	1.2	1	1
B	1.3	1.6	1	1.2	1	1	1	1
C	1	1	1	1	1	1	1	1

Compute seismic coefficients CA and Cv from Table - 8 & table - 9 respectively. The factor ZEN is the multiplication of the seismic zone factor (Z) and the near-source factors (either NA or NV) and the following values for E = 0.5 for the serviceability earthquake, 1.0 for the design earthquake, 1.25 for the maximum earthquake in zone 4 and 1.5 for the maximum earthquake in zone 3.

Table 10: Seismic coefficient CA (Table 4-7 in ATC-40)

Soil profile type	Shaking Intensity, ZEN					
	= 0.075	= 0.15	= 0.2	= 0.3	= 0.4	> 0.4
SB	0.08	0.15	0.2	0.3	0.4	1*ZEN
SC	0.09	0.18	0.24	0.33	0.4	1*ZEN
SD	0.12	0.22	0.28	0.36	0.44	1.1*ZEN
SE	0.19	0.3	0.34	0.36	0.36	0.9* ZEN

Table 11: Seismic coefficient Cv (Table 4-8 in ATC-40)

Soil profile type	Shaking Intensity, ZEN					
	= 0.075	= 0.15	= 0.2	= 0.3	= 0.4	> 0.4
SB	0.08	0.15	0.2	0.3	0.4	1*ZEN
SC	0.13	0.25	0.32	0.45	0.56	1.4*ZEN
SD	0.18	0.32	0.4	0.57	0.64	1.6*ZEN
SE	0.26	0.5	0.64	0.84	0.96	2.4* ZEN

Seismic coefficient (CA and Cv) for dense soil were listed on Table - 12 with different earthquake source distance.

Table 12: Seismic coefficient CA and CV for very dense soil with soft rock

Earthquake source distance	Seismic coefficient (CA)	Seismic coefficient (Cv)
Less than 2km	0.75	1.4
Between 2km and 5km	0.6	1.12
Between 5km and 10km	0.5	0.84
Between 10km and 15km	0.5	0.7

Seismic coefficient (CA and Cv) for soil profile stiff soil are listed on Table -13 with different earthquake source distance.

Table - 13: Seismic coefficient CA and CV for stiff soil profile

Earthquake source distance	Seismic coefficient (CA)	Seismic coefficient (Cv)
Less than 2km	0.825	1.6
Between 2km and 5km	0.66	1.28

Between 5km and 10km	0.55	0.96
Between 10km and 15km	0.55	0.8

The effect of soil profile and source distance has considerable effect on shape and magnitude of response spectra.

3.5. Modeling the Sample Building for Linear Response Spectrum Analysis

This part shows model of sample building considered in response spectrum analysis. In structural analysis software a frame element was modeled as a line element having linearly elastic properties. All three components earthquake response spectrum function was applied in all three nodal displacement direction of the frame system. Finally response spectrum function for vertical earthquake ground acceleration component was applied at different near source distance to know effect of source distance.

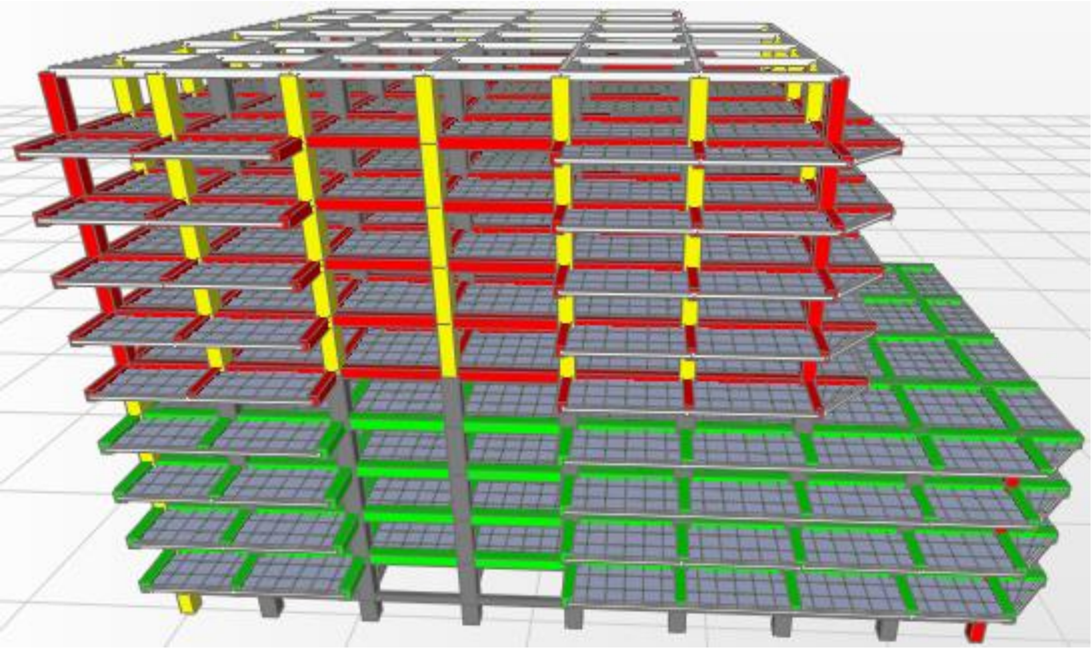


Figure 7: G+10 mixed use sample building model on structural analysis software

4. RESULTS AND COMPARISON

4.1. General

To examine the effect of vertical earthquake component on selected building structures, first the result due to three components X, Y and Z was considered. At next step, the same earthquake result has been considered without considering the effect of vertical earthquake component. In the model, responses of building frame elements caused by horizontal component vs vertical and horizontal components have been studied. The response at the end of selected columns, beams and slabs were considered. For selected near source distance, effect of vertical component earthquake ground acceleration on two type of soil profile was assessed.

4.2. Comparison of Building Response due to effect of HEGA Vs HEGA and VEGA

4.2.1. Effect of HEGA Vs HEGA and VEGA on Vertical Member (Columns)

In this section, the results due to horizontal vs horizontal and vertical component earthquake ground acceleration has been presented. The effects of vertical component earthquake ground acceleration on (exterior, interior and column under cantilever slab) were presented in Table 14-16 and Figure 8-10. Each internal response comparison due to both horizontal and vertical earthquake ground acceleration was presented in Appendix A. Columns used in the comparison are selected based on their position in building (exterior, interior, or column under cantilever slab)

Table 14 : Average internal response variation due to HEGA Vs HEGA and VEGA on columns at axis - A3 and C7

	average difference due to VEGA on exterior column			Average percentage difference due to VEGA on exterior column		
story	Axial (kN)	Shear (kN)	Bending(kN-m)	Axial (%)	Shear (%)	Bending (%)
0	0	1.640	2.460	0	0.608	0.503
1	0	0.895	1.530	0	0.384	0.470
2	0	1.030	1.680	0	0.488	0.541
3	0	1.160	1.945	0	0.708	0.705
4	0	1.190	2.700	0	0.586	0.626
5	0	1.150	2.090	0	0.619	0.715

6	0	0.990	1.870	0	0.610	0.885
7	0	1.025	2.345	0	0.767	1.628
8	0	1.225	2.165	0	1.319	2.513
9	0	0.435	1.090	0	1.722	2.875

Table 15: Average internal response variation due to HEGA Vs HEGA and VEGA on columns at axis - A1, F7 and I2

	average difference due to VEGA on column under cantilever slab			Average percentage difference due to VEGA on column under cantilever slab		
story	Axial (kN)	Shear (kN)	Bending(kN-m)	Axial (%)	Shear (%)	Bending (%)
0	1.418	1.531	1.788	0.044	0.849	0.604
1	1.652	0.690	1.270	0.058	0.374	0.459
2	1.356	1.112	2.331	0.056	0.660	0.985
3	0.816	1.063	1.632	0.041	0.670	0.693
4	0.563	0.840	2.025	0.035	0.517	0.608
5	0.043	0.883	1.670	0.003	0.632	0.761
6	0	0.808	1.275	0	0.610	0.741
7	0	0.671	1.537	0	0.654	1.276
8	0	0.990	1.718	0	1.215	2.102
9	0	0.345	0.710	0	2.371	2.070

Table 16: Average internal response variation due to HEGA vs HEGA and VEGA on columns at axis - B2 and F3

	average difference due to VEGA on interior column			Average percentage difference due to VEGA on interior column		
story	Axial (kN)	Shear (kN)	Bending(kN-m)	Axial (%)	Shear (%)	Bending (%)
0	0	2.666	3.442	0	0.823	0.622
1	0	1.391	2.487	0	0.454	0.553
2	0	2.086	3.708	0	0.750	1.076
3	0	1.909	3.410	0	0.864	0.943
4	0	1.668	3.592	0	0.624	0.640
5	0	2.012	3.959	0	0.785	0.980
6	0	2.016	3.185	0	0.895	1.089
7	0	1.576	4.645	0	0.873	2.479
8	0	2.123	4.104	0	1.868	4.414
9	0	1.359	2.565	0	3.948	4.824

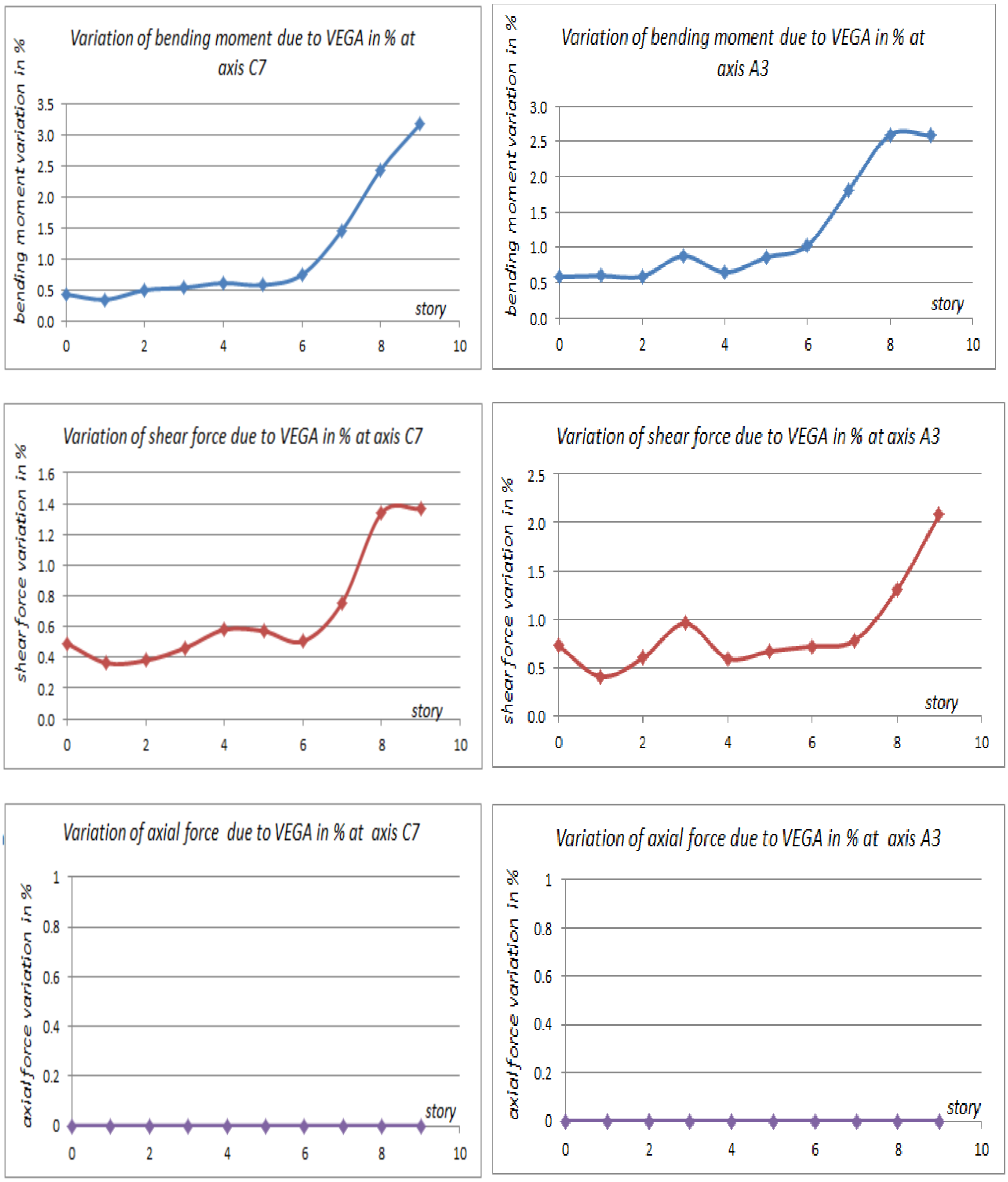
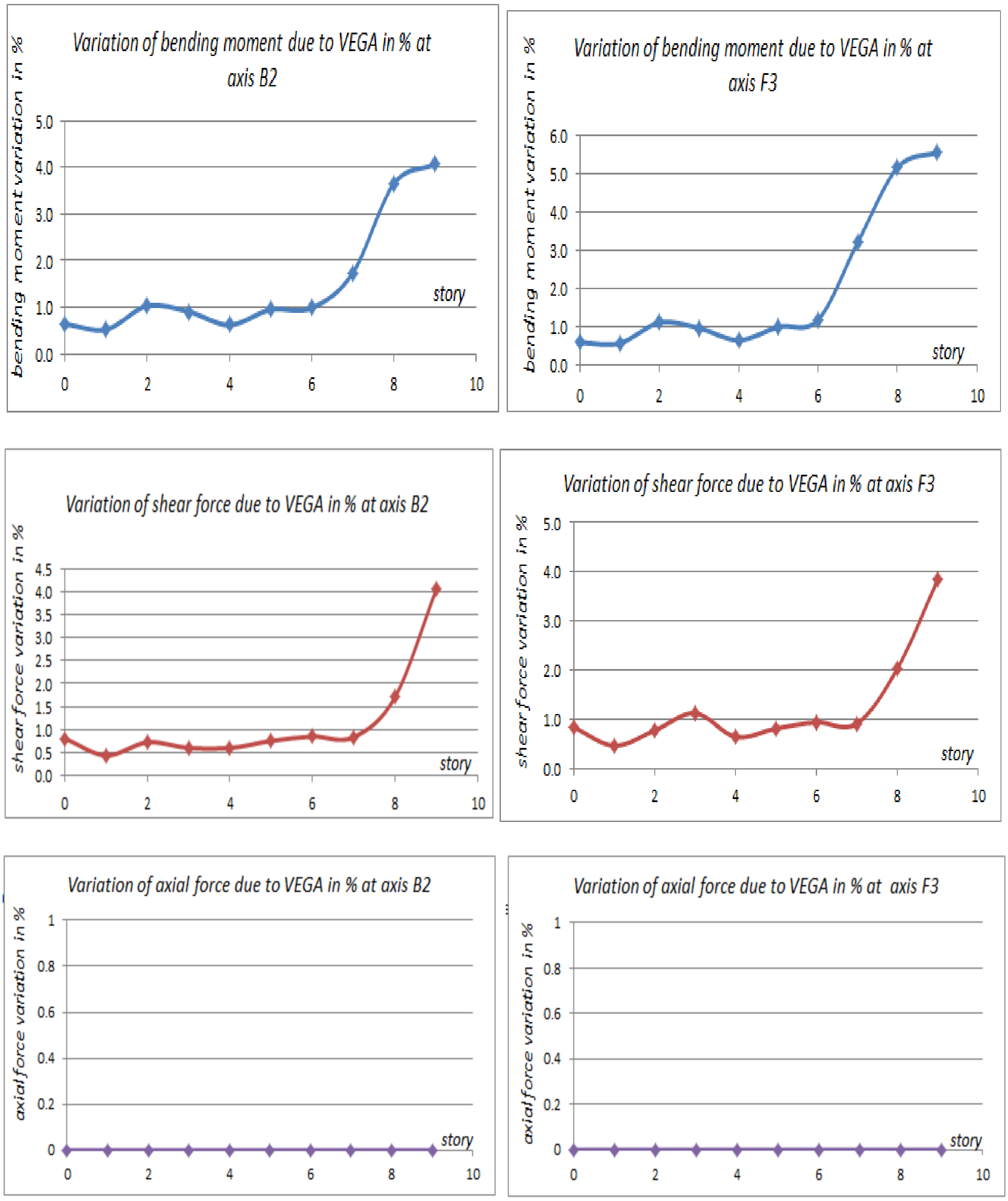


Figure 8: Variation of internal response due to VEGA at exterior column (axis C7 and A3)



Variation of bending moment due to VEGA in % at axis B2

Variation of bending moment due to VEGA in % at axis F3

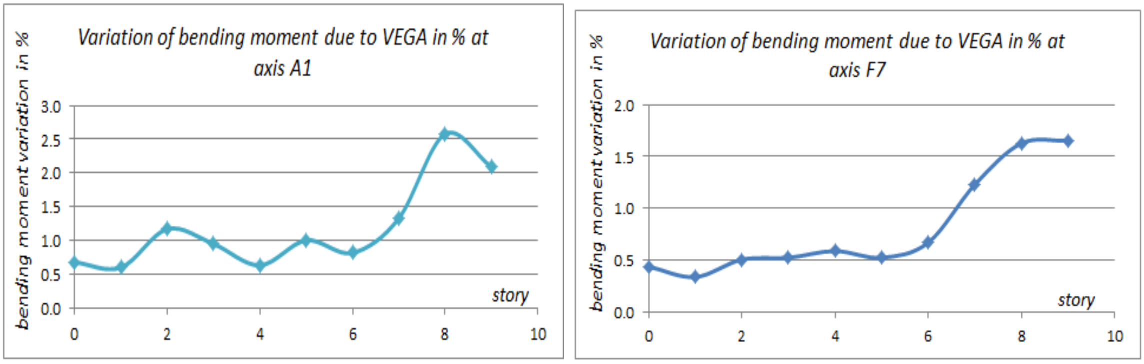
Variation of shear force due to VEGA in % at axis B2

Variation of shear force due to VEGA in % at axis F3

Variation of axial force due to VEGA in % at axis B2

Variation of axial force due to VEGA in % at axis F3

Figure 9: Variation of internal response due to VEGA at interior column (axis B2 and F3)



Variation of bending moment due to VEGA in % at axis A1

Variation of bending moment due to VEGA in % at axis F7

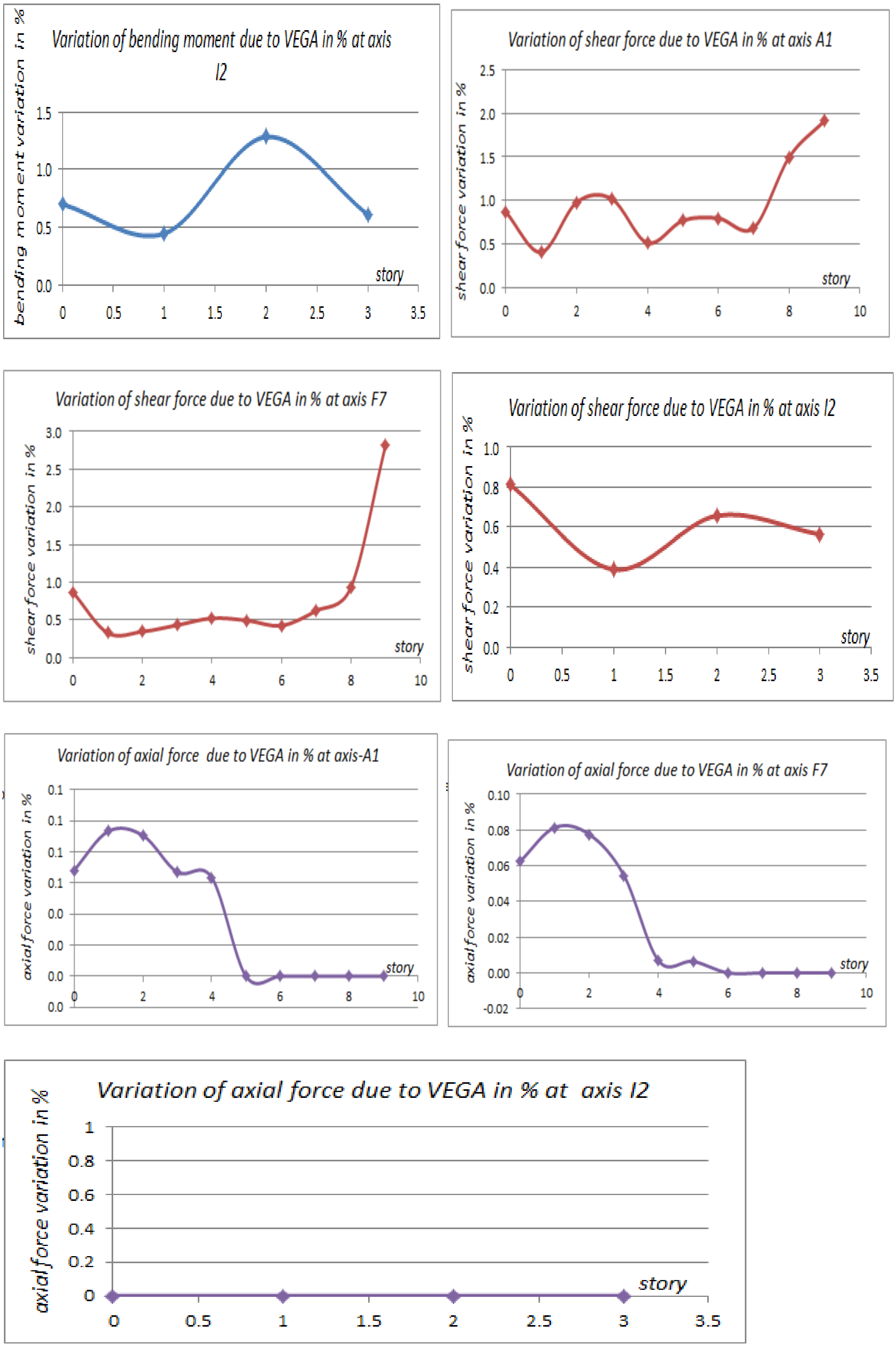


Figure 10: Variation of internal response due to VEGA at column under cantilever slab (axis F7, A1 and I2)

4.2.2. Effect of HEGA Vs HEGA and VEGA on Slabs

The values of stress in slabs due to horizontal vs horizontal and vertical component earthquake ground acceleration were presented in Table 17 and Figure 11.

Table 17: maximum positive and negative stress in longitudinal and transversal direction

Stress	H (kN/m^2)	H+ V (kN /m^2)	Variation due to in %
Maximum positive S11	5978.88	5978.88	0
Maximum negative S11	2761.38	2745.55	0.57
Maximum positive S22	5675.04	5675.04	0
Maximum negative S22	2884.63	2878.48	0.21
Maximum positive S12	1429.4	1429.4	0
Maximum negative S12	609.44	603.18	1.03

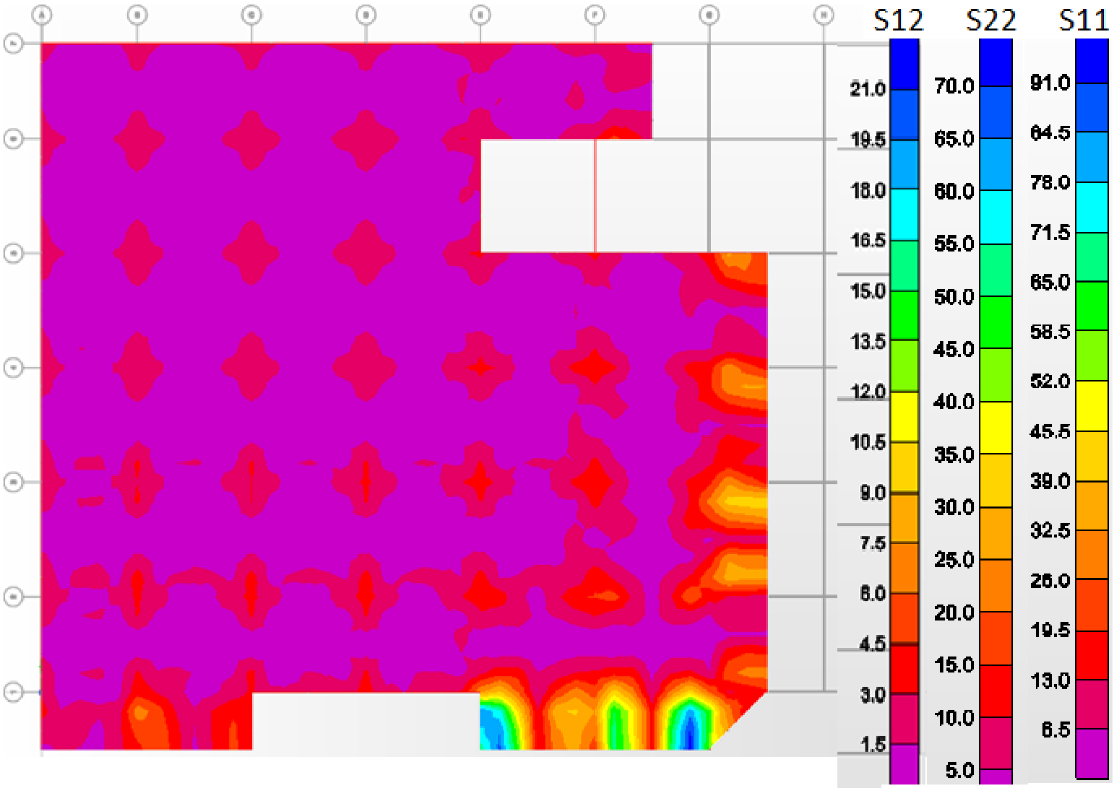


Figure 11 : Stress on slab due to vertical earthquake ground acceleration in selected slab

4.3. Building Response at near Source due to VEGA

The comparison of internal response at different source distance was displayed below to evaluate vertical component earthquake ground acceleration. The effect of source distance to magnify vertical earthquake component had tangible effect on response of structural element as shown in Table 18-20 and Figure 10. To summarize effect of VEGA at near source, results of internal response due to horizontal vs horizontal and vertical EQGA were presented in Appendix C.

Table 18: Average internal response variation due to HEGA vs HEGA and VEGA on selected exterior and interior columns at 2km from source

Story	Average difference at exterior column under cantilever slab on axis A1, F7 and I2			Average difference at interior column on axis B2 and F3		
	Axial (kN)	Shear (kN)	Bending (kN-m)	Axial(kN)	Shear (kN)	Bending (kN-m)
0	54.08	26.15	35.11	3.17	38.19	59.82
1	49.60	19.77	27.98	3.64	30.29	41.78
2	41.11	17.56	27.91	3.38	26.63	42.38
3	30.16	19.06	27.92	3.01	25.04	40.94
4	20.50	13.13	29.87	2.87	25.37	57.00
5	17.44	12.54	22.24	2.64	27.61	51.37
6	14.46	12.16	19.46	2.51	28.05	46.85
7	11.43	11.39	18.63	2.43	26.73	43.56
8	6.74	11.33	15.95	0	24.05	36.67
9	1.15	3.38	6.15	0	10.72	22.30

Table 19: Average internal response variation due to HEGA vs HEGA and VEGA on selected exterior and interior columns within 15km from source

Story	Average difference at exterior column under cantilever slab on axis - A1, F7 and I2			Average difference at interior column on axis - B2 and F3		
	Axial (kN)	Shear (kN)	Bending(kN-m)	Axial(kN)	Shear (kN)	Bending (kN-m)
0	22.50	14.22	21.89	0	11.18	17.59
1	21.27	9.51	13.20	0	8.55	11.27
2	18.37	8.55	14.27	0	7.14	11.44
3	12.38	8.58	14.34	0	6.70	11.53
4	7.69	5.59	13.67	0	7.28	16.81
5	6.96	5.46	10.36	0	8.02	15.35
6	6.45	5.46	9.07	0	8.30	14.54
7	5.85	5.14	8.77	0	8.25	13.93
8	3.65	5.24	7.63	0	7.27	10.95
9	0.68	1.54	2.75	0	3.35	7.06

Table 20: Average internal response variation due HEGA vs HEGA and VEGA on selected columns above 15km from source

Story	Average difference at exterior column under cantilever slab on axis - A1, F7 and I2			Average difference at interior column on axis - B2 and F3		
	Axial(kN)	Shear (kN)	Bending(kN-m)	Axial(kN)	Shear (kN)	Bending (kN-m)
0	15.27	9.43	14.69	0	12.16	16.13
1	16.08	5.91	8.62	0	11.46	10.39
2	13.22	5.72	10.51	0	10.65	12.66
3	8.39	6.21	10.42	0	11.19	12.30
4	4.85	3.89	9.21	0	10.91	15.96
5	4.56	3.68	6.89	0	11.66	14.65
6	4.59	3.56	6.10	0	11.79	13.85
7	4.39	3.44	6.44	0	11.75	14.49
8	2.84	3.85	5.93	0	11.67	12.68
9	0.53	1.20	1.51	0	5.48	7.74

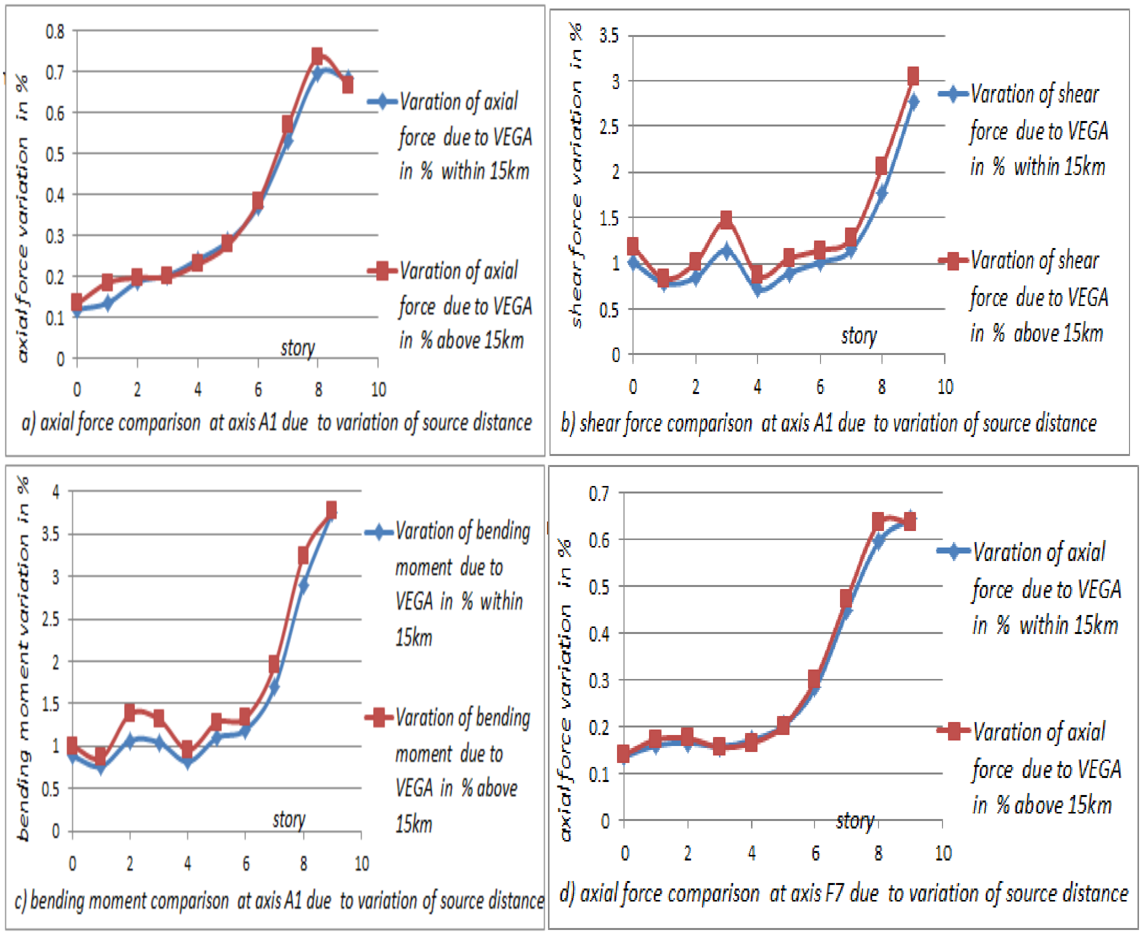
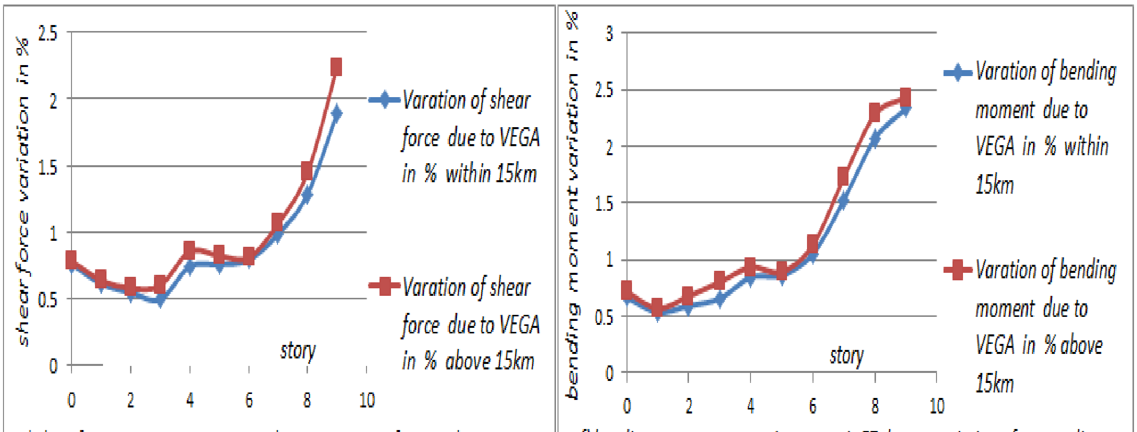
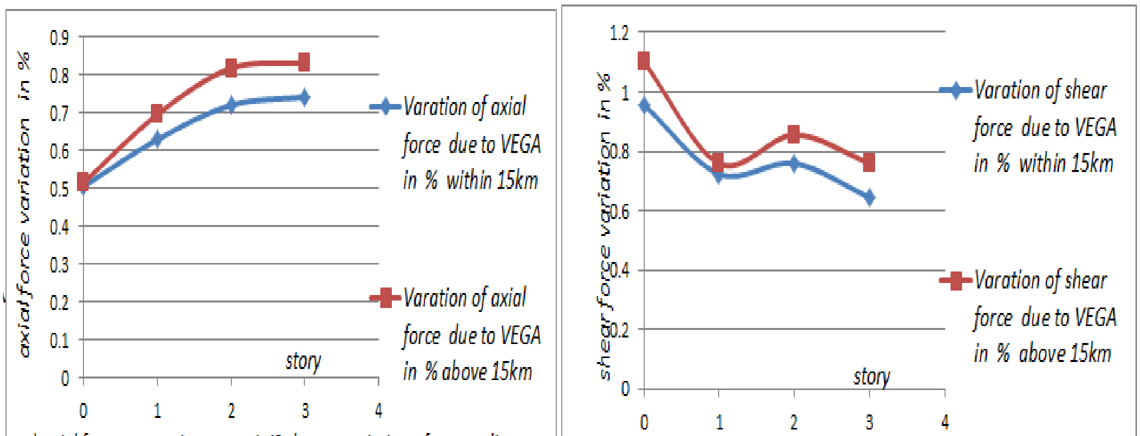


Figure 12 : Effect of VEGA on building response due to difference source distance



e) shear force comparison at axis F7 due to variation of source distance

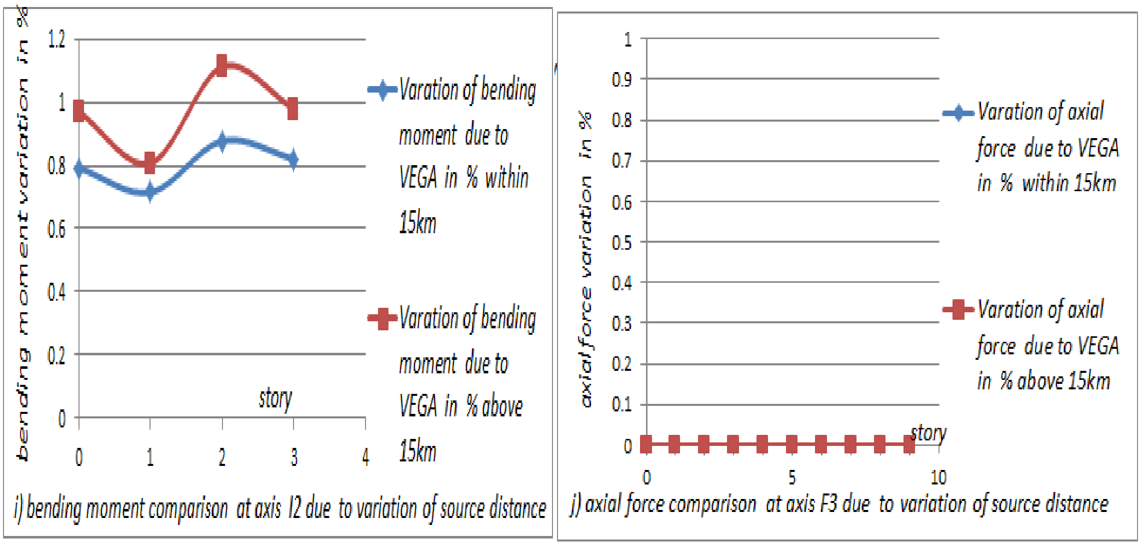
f) bending moment comparison at axis F7 due to variation of source distance



g) axial force comparison at axis I2 due to variation of source distance

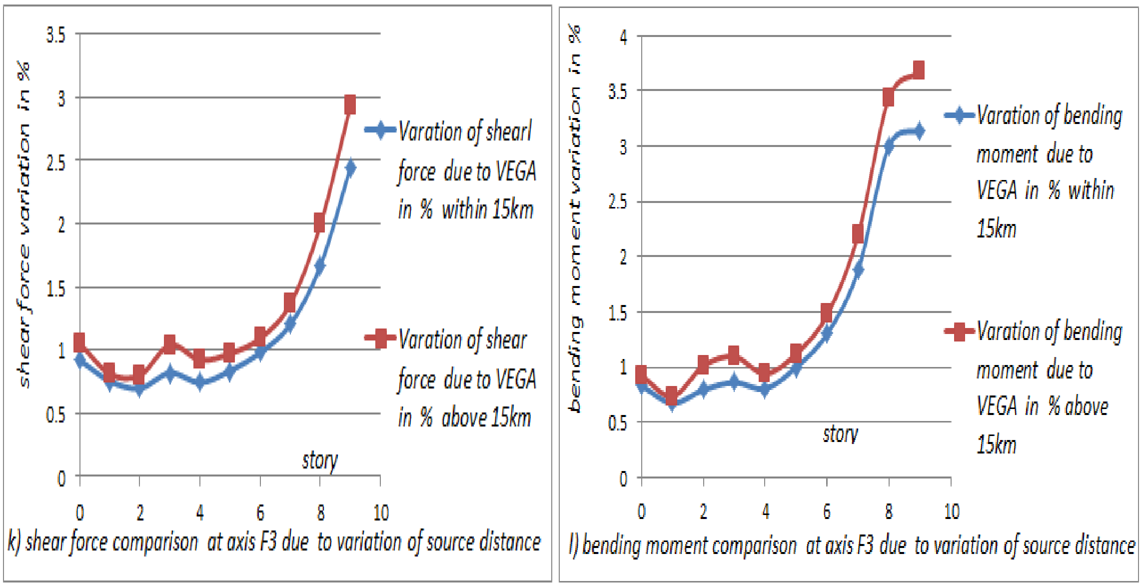
h) shear force comparison at axis I2 due to variation of source distance

Figure 12: Effect of VEGA on building response due to difference source distance cont……



i) bending moment comparison at axis I2 due to variation of source distance

j) axial force comparison at axis F3 due to variation of source distance



k) shear force comparison at axis F3 due to variation of source distance

l) bending moment comparison at axis F3 due to variation of source distance

Figure 12: Effect of VEGA on building response due to difference source distance cont.…

4.4. Internal Response Comparison between Dense and Stiff Soil Profile

In this part the effect of vertical earthquake ground acceleration on dense and stiff soils profile were presented graphically. As shown in Appendix D and Annex B the effect of soil profile had considerable amount of vertical earthquake ground acceleration magnification on response of structural element.

5. DISCUSSIONS

5.1. General

In this study, the influence of vertical component earthquake ground acceleration on selected reinforced concrete frame elements has been assessed within the framework of linear response spectrum analysis (RSA). As presented in result parts, seismic design of the structure with neglecting vertical component earthquake ground acceleration have not considerable effect in main skeleton of building within zone 4 of Ethiopia. Generally, analysis results presented in comparison part with near fault distance graphically in Figure 10 and in the Appendix C, collectively point towards the importance of including vertical component earthquake ground acceleration in analysis and design of reinforce concrete building structure where building was located around fault. Due to this, seismic design of the structure without considering vertical component earthquake component may have risk in main skeleton of building at near faults (highly suspected area to earthquake).

5.2. Comparison of Building Response due to HEGA Vs HEGA and VEGA

5.2.1. Effect of HEGA Vs HEGA and VEGA on Interior Column - C3 (at axis B2 and axis F3)

Vertical ground motion has little magnifying effects on shear force and bending moment demand on the vertical load carrying members. Totally response due to vertical component earthquake ground acceleration had very small effect compare to horizontal earthquake ground acceleration where building was located within zone 4 of Ethiopia. Influences of vertical component earthquake ground acceleration on axial force demand in selected interior columns were almost zero. From this, VEGA doesn’t have any magnification on axial force in interior columns. To some extent, vertical ground motion has considerable effects on shear force and bending moment in interior columns. Form this; ignoring vertical earthquake ground acceleration component causes an average of 3.948% for shear force and 4.824% for bending moment demand in interior column as shown in Table16. The result led to the confirmation to the occurrence of very small shear and bending moment in the interior columns, thus dispelling neglecting of vertical component earthquake ground motion in analysis and design of reinforced concrete buildings at zone 4 is almost tolerable.

5.2.2. Effect of HEGA Vs HEGA and VEGA on Exterior Columns C2 (axis - A3 and axis - C7)

Vertical ground motion has minor magnifying effects on internal response demand on the vertical load carrying members. Ignoring vertical earthquake ground acceleration component causes an average of 1.319% for shear force and 2.513% for bending moment demand in exterior columns as shown in Table 14. In view of this study, Influences of vertical component earthquake ground acceleration on selected exterior columns were very small compared to response of lateral earthquake ground acceleration.

5.2.3. Effect of HEGA Vs HEGA and VEGA on Exterior Columns under cantilever slab C1 & C2' (at axis A1, F7 and I2)

As shown in Appendix A, vertical ground motion has magnifying effects on axial force; shear force and bending moment demand on column under cantilever slab. Neglecting vertical earthquake ground acceleration component causes an average of 0.056% for axial force, 2.371% for shear force and 2.102 for bending moment demand in column as shown in Table 15. This result led to the confirmation of the occurrence of tensile and compression forces in columns under cantilever slab. Columns subjected to cantilever slab was vulnerable in this respect compared to other columns, since there is net axial force, shear force and bending moment. This may cause very small defects in vertical bearing elements such columns during high magnitude earthquake. Generally, the effect of vertical earthquake ground acceleration can be omitted if the building is built far from active faults. Average percentage magnification of internal response due to vertical component earthquake acceleration is high at top story compared to bottom story results as shown in Appendix A.

5.2.4. Effect of HEGA Vs HEGA and VEGA on Beams IB1 at axis -1 and IB2 at axis - 1 and axis - E

The effect of the vertical earthquake ground acceleration on beam was to increase shear force and bending moment demand. It was observed that the bending moment and shear due to vertical component earthquake ground acceleration had insignificant effects in beam. In this selected case, the effect of ignoring vertical earthquake ground acceleration component causes a maximum of 1.06kN in shear force and 2.55kN-m in bending moment demand at beam support as shown in Appendix B. The result was gathered from beams around support and beams subjected for cantilever slab. Since the effect of vertical earthquake ground acceleration is very small and it can be omitted in beams.

5.2.5. Effect of HEGA Vs HEGA and VEGA on Slabs

The values of stress due to vertical earthquake ground acceleration were shown in Table 17 and Figure-11. Effect of ignoring vertical earthquake ground acceleration component has benefit to reduce maximum negative stress in longitudinal and transversal direction.

5.3. Effect of VEGA at near fault on C1, C2’ & C3 (at axis A1, F7, B2, F3 & I2)

There are two types of seismic body waves, i.e. primary (p-waves) and secondary (S-wave). As the wavelength of P-waves was shorter than that of S-waves, the vertical component of ground motion has much higher frequency content than the horizontal component in the near field earthquake. Since velocity of P-waves is faster than that of S-wave, vertical component of seismic waves reaches earlier than that of lateral component of seismic waves which is adopted in analysis and design of buildings. Also vertical component of earthquake ground motion has much higher frequency content than the horizontal component in the near fault earthquakes. This parameters cause axial force, shear force and bending moment demand in columns due to ignoring vertical component of earthquake ground acceleration especially at near faults as shown in Figure-12 and Table 18-20.

As the distance from the fault increases, the difference in the frequency content between horizontal and vertical components becomes much smaller as a result of faster reduction of high frequencies with distance, and mixing of horizontal and vertical motions due to non-homogeneities along the wave path. Because of this response of structural element are almost similar at far distance from faults.

However, it should be kept in mind that near-field ground motions from relatively large magnitude earthquakes can produce significant ground motions in both the horizontal and vertical directions, to the extent that the vertical motion may significantly exceed the horizontal one at near source. When sample building was sited at distance less than 15km average magnification of internal response reach up to 22.5kN in axial force, 14.22kN in shear force and 21.99kN-m in bending moment demand as shown in Table 19. But when sample building was located at far distance magnification of internal response is very small compared to the near source results. In this model, when sample building was located at distance of above 15km average magnification of internal response reach up to 15.27kN in axial force, 12.16kN in shear force and 16.13kN-m in bending moment demand as shown in Table 20. From this, seismic design of the structure without considering vertical component earthquake component may have risk in main skeleton of building at around faults.

5.4. Response Comparison between Dense Soil and Stiff Soil Profile on C1, C2’ & C3 (axis A1, F7, F3 & I2)

In this portion the effect of vertical earthquake ground acceleration (VEGA) on dense and stiff soils profile were discussed. Thus, for very dense soils and soft rocks at near faults vertical acceleration alone can affect both sides of the equilibrium equation: by increasing the driving forces and, at the same time, by reducing the shearing resistance of the material through an increase in the pore water pressures. Because of this identification of soil type was a critical issue to consider vertical earthquake ground acceleration in analysis of building structure at near fault area. Effects of vertical earthquake ground acceleration (VEGA) on dense and stiff soils profile were presented in Appendix D and Annex B.

In this section, the results due to horizontal vs horizontal and vertical component of earthquake ground acceleration at zone 4 and near source has been presented. The effects of vertical component earthquake ground acceleration have very small effect on response of buildings (axial force, shear force, bending moment and stress) where the building is sited in zone 4 of Ethiopia.

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

In this thesis effect of vertical component earthquake ground acceleration at zone 4 of Ethiopia and at near source were assessed. The assessment was done by comparing structural responses like axial force, shear force, bending moment and stress in building skeleton due to horizontal vs both horizontal and vertical earthquake ground acceleration. After careful inspection and comparison of the output on selected columns, beams and slabs, the following conclusions were drawn:

- ⊗ Responses like axial force, shear force and bending moment have small variation due to addition of vertical earthquake ground acceleration as shown in the results and discussion part. Its effect was high in columns compared to other building skeleton. Average percentage magnification of internal response due to vertical component earthquake acceleration is high at top story compared to bottom story. Influences of vertical component earthquake ground acceleration on selected interior and exterior columns have almost negligible effect. Generally, ignoring the effect of vertical component earthquake ground acceleration for location, away from active fault is almost tolerable.
- ⊗ When sample building was sited at near source distance, average magnification of internal response like axial force, shear force and bending moment demand has considerable effect. From this, if the proposed building structure is sited around near faults, vertical component earthquake ground acceleration should be considered or appropriate safety factor should be considered.
- ⊗ Effect of vertical component earthquake ground acceleration at near fault has effect when the soil condition becomes dense and stiff clay. Especially for near source distances soil profile has considerable effects on the value of axial force, shear force and bending moment demands in columns.

To concluding this, the local site effect and source distance were a significant contributors to peak response of vertical earthquake component. Generally, the analysis results and consecutives discussion presented above collectively point towards about ignoring the effect of vertical component earthquake ground acceleration for location away from active fault and including high magnitude vertical component earthquake ground acceleration in design of reinforce concrete building structure around near faults.

6.2. Recommendation

This thesis work is an inch towards the phenomena of effect of vertical component earthquake ground acceleration on building structure. From this research the following recommendations are outlined for further study:

→ A combination of horizontal, vertical, and rotational components of earthquake ground acceleration with soil structure interaction has been left for future study.

→ In this study the effect of vertical component earthquake ground acceleration for fixed base building structure was studied. But the base isolated has been left for future study.

→ Effect of vertical earthquake ground acceleration on horizontal or nearly horizontal structural members spanning 20m or more, beam supported columns horizontal or nearly horizontal pre-stressed components are left for future study.

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Appendix A

Effect of HEGA Vs HEGA and VEGA on Exterior, Column under Cantilever, Interior Columns on C1, C2, C2' & C3 (axis A1, C3, F7, A3, B2, F3 & I2)

Table A1: Internal response at column – C2 and axis - A3 due to horizontal vs horizontal and vertical EQGA (exterior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	3250.89	3250.89	256.42	258.28	478.92	481.71	0	0.725	0.583
1	2886.9	2886.9	195.41	196.2	279.14	280.81	0	0.404	0.598
2	2518.86	2518.86	185.93	187.04	273.52	275.13	0	0.597	0.589
3	2147.68	2147.68	137.22	138.53	237.23	239.31	0	0.955	0.877
4	1775.67	1775.67	181.2	182.27	402.46	405.06	0	0.591	0.646
5	1443.02	1443.02	173.88	175.04	280.37	282.77	0	0.667	0.856
6	1120.27	1120.27	149.44	150.51	192.51	194.49	0	0.716	1.029
7	797.05	797.05	123.67	124.63	132.34	134.74	0	0.776	1.814
8	472.89	472.89	85.44	86.55	87.52	89.79	0	1.299	2.594
9	149.76	149.76	26.95	27.51	50.32	51.62	0	2.078	2.583

Table A2: Internal response at column – C2 and axis - C7 due to horizontal vs horizontal and vertical EQGA (exterior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	3253.98	3253.98	289.82	291.24	503.56	505.69	0	0.490	0.423
1	2892.41	2892.41	275.08	276.08	407.2	408.59	0	0.364	0.341
2	2526.39	2526.39	250.33	251.28	355.22	356.97	0	0.379	0.493
3	2157.87	2157.87	219.34	220.35	339.26	341.07	0	0.460	0.534
4	1787.34	1787.34	225.67	226.98	461.5	464.3	0	0.580	0.607
5	1454.02	1454.02	199.67	200.81	310.19	311.97	0	0.571	0.574
6	1129.02	1129.02	180.21	181.12	237.35	239.11	0	0.505	0.742
7	803.22	803.22	143.89	144.98	158.7	160.99	0	0.758	1.443
8	476.4	476.4	100.1	101.44	84.71	86.77	0	1.339	2.432
9	150.3	150.3	22.7	23.01	27.79	28.67	0	1.366	3.167

Table A3: Internal response at column - C1 and axis - A1 due to horizontal vs horizontal and vertical EQGA (Column under Cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	3247.046	3249.263	213.981	215.841	363.11	365.561	0.068	0.869	0.675
1	2830.288	2832.933	202.813	203.64	297.654	299.435	0.093	0.408	0.598
2	2416.272	2418.463	186.246	188.07	259.734	262.767	0.091	0.979	1.168

3	2012.532	2013.88	147.591	149.091	238.462	240.718	0.067	1.016	0.946
4	1592.88	1593.894	190.799	191.779	385.006	387.431	0.064	0.514	0.630
5	1272.477	1272.477	157.592	158.806	243.127	245.559	0	0.770	1.000
6	978.674	978.674	148.129	149.31	201.71	203.352	0	0.797	0.814
7	684.826	684.826	120.62	121.448	145.228	147.153	0	0.686	1.326
8	390.766	390.766	93.038	94.431	97.362	99.871	0	1.497	2.577
9	97.373	97.373	27.006	27.523	51.808	52.89	0	1.914	2.088

Table A4: Internal response at column – C2 and axis - F7 due to horizontal vs horizontal and vertical EQGA (Column under Cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	3263.247	3265.284	175.438	176.957	314.053	315.423	0.062	0.866	0.436
1	2838.831	2841.142	156.327	156.837	230.016	230.785	0.081	0.326	0.334
2	2420.3	2422.177	142.808	143.299	200.757	201.768	0.078	0.344	0.504
3	2013.441	2014.54	120.303	120.82	190.159	191.15	0.055	0.430	0.521
4	1657.143	1657.255	134.154	134.853	277.227	278.852	0.007	0.521	0.586
5	1332.454	1332.54	111.84	112.391	174.018	174.926	0.006	0.493	0.522
6	1023.506	1023.506	102.712	103.147	136.202	137.111	0	0.424	0.667
7	715.1	715.1	82.967	83.482	93.619	94.768	0	0.621	1.227
8	406.951	406.951	62.95	63.537	57.006	57.933	0	0.932	1.626
9	99.926	99.926	6.155	6.329	20.456	20.794	0	2.827	1.652

Table A5: Internal response at column - C2’ and axis – I2 due to horizontal vs horizontal and vertical EQGA (Column under Cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	2146.117	2146.117	149.348	150.562	220.313	221.856	0	0.813	0.700
1	1609.186	1609.186	188.096	188.828	283.259	284.519	0	0.389	0.445
2	1073.558	1073.558	155.455	156.477	229.576	232.524	0	0.657	1.284
3	538.209	538.209	207.927	209.098	270.01	271.66	0	0.563	0.611

Table A6: Internal response at column - C3 and axis - B2 due to horizontal vs horizontal and vertical EQGA (Interior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5154.5	5154.5	334.261	336.972	561.234	564.845	0	0.811	0.643
1	4570.967	4570.967	325.33	326.776	480.496	483.01	0	0.444	0.523
2	3989.018	3989.018	294.941	297.086	416.326	420.604	0	0.727	1.028
3	3409.592	3409.592	242.921	244.395	387.111	390.617	0	0.607	0.906
4	2831.481	2831.481	283.245	284.956	586.718	590.364	0	0.604	0.621
5	2292.759	2292.759	269.118	271.159	422.135	426.133	0	0.758	0.947

6	1762.883	1762.883	239.364	241.409	314.972	318.067	0	0.854	0.983
7	1233.624	1233.624	192.646	194.26	212.364	216.045	0	0.838	1.733
8	704.716	704.716	127.84	130.023	114.056	118.219	0	1.708	3.650
9	176.773	176.773	38.314	39.864	61.567	64.07	0	4.046	4.065

Table A7: Internal response at column - C3 and axis – F3 due to horizontal vs horizontal and vertical EQGA (Interior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation internal response due to VEGA in %		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5226.322	5226.322	313.816	316.437	544.164	547.437	0	0.835	0.601
1	4906.498	4906.498	287.64	288.975	421.593	424.054	0	0.464	0.584
2	4282.587	4282.587	262.193	264.22	367.834	371.973	0	0.773	1.125
3	3658.265	3658.265	208.906	211.25	338.299	341.613	0	1.122	0.980
4	3029.104	3029.104	252.069	253.693	537.252	540.789	0	0.644	0.658
5	2453.528	2453.528	244.378	246.361	387.165	391.085	0	0.811	1.012
6	1886.14	1886.14	212.361	214.348	274.073	277.348	0	0.936	1.195
7	1318.37	1318.37	169.243	170.78	173.924	179.533	0	0.908	3.225
8	749.641	749.641	101.795	103.859	78.127	82.172	0	2.028	5.177
9	183.509	183.509	30.338	31.506	47.044	49.67	0	3.850	5.582

Appendix B

Effect of HEGA Vs HEGA and VEGA on Beams

Table B1: Internal response of intermediate beam IB1 at (G+1 & axis -1) due to horizontal vs horizontal and vertical EQGA

	at A	at B	at C	at D	at E	at F	at G	at H	at I
Shear due to H (kN)	256.81	226.987	192.557	191.166	224.328	222.409	224.73	237.042	119.42
Shear due to H+V (kN)	257.87	227.722	193.287	191.89	225.067	223.144	225.44	237.958	119.72
Bending due to H(kN-m)	526.12	519.945	488.186	466.849	503.37	501.554	501.776	499.341	491.081
Bending due to H+V(kN-m)	528.65	522.472	490.277	468.99	505.513	503.696	503.822	501.662	493.456
variation in shear	1.061	0.735	0.73	0.724	0.739	0.735	0.71	0.916	0.3
variation in bending	2.535	2.527	2.091	2.141	2.143	2.142	2.046	2.321	2.375

Table B2: Internal response of intermediate beam IB2 at (G+5 & axis -1) due to horizontal vs horizontal and vertical EQGA

	at A	at B	at C	at D	at E	at F	at G
Shear due to H (kN)	192.807	189.952	148.84	144.545	182.5	176.863	101.616
Shear due to H+V (kN)	193.552	190.494	149.338	145.042	183.046	177.359	101.616
Bending due to H(kN-m)	372.827	396.885	367.062	341.89	383.08	387.948	364.928
Bending due to H+V(kN-m)	374.536	398.359	368.489	343.326	384.577	389.406	366.402
variation in shear	0.745	0.542	0.498	0.497	0.546	0.496	0
variation in bending	1.709	1.474	1.427	1.436	1.497	1.458	1.474

Table B3: Internal response of intermediate beam IB1 at (G+2 & axis -E) due to horizontal vs horizontal and vertical EQGA

	at 7	at 6	at 5	4	at 3	at 2	at 1
Shear due to H (kN)	218.373	199.84	199.822	203.159	169.077	226.835	89.822
Shear due to H+V (kN)	219.017	200.3	200.283	203.183	169.527	227.467	90.455

Bending due to H(kN-m)	436.941	487.337	461.34	460.174	457.734	457.626	467.094
Bending due to H+V(kN-m)	438.491	488.927	462.688	461.527	459.09	459.219	468.643
variation in shear	0.644	0.46	0.461	0.024	0.45	0.632	0.633
variation in bending	1.55	1.59	1.348	1.353	1.356	1.593	1.549

Appendix C

Building Response at Variation of near Source VEGA on C1, C2' & C3 (axis A1, F7, B2, F3 & I2)

Table C1: Internal response at axis - A1 and column - C1 within 2km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	23300.66	23326.39	3049.489	3075.523	5449.489	5468.738	25.731	26.034	19.249
1	19899.65	19928.14	2695.896	2714.314	3999.464	4025.908	28.485	18.418	26.444
2	16541.15	16568.05	2481.186	2498.962	3500.579	3530.909	26.898	17.776	30.33
3	13367.2	13390.86	1851.139	1874.664	3097.898	3124.622	23.661	23.525	26.724
4	9984.129	10005.37	2558.659	2574.287	5364.41	5401.288	21.243	15.628	36.878
5	7477.543	7496.627	2125.052	2141.023	3265.977	3296.298	19.084	15.971	30.321
6	5023.967	5040.965	1886.873	1903.112	2447.821	2473.376	16.998	16.239	25.555
7	2919.94	2934.56	1446.624	1461.117	1574.964	1599.401	14.62	14.493	24.437
8	1310.377	1319.313	975.643	991.157	836.923	860.587	8.936	15.514	23.664
9	213.024	214.589	175.257	180.096	230.214	239.418	1.565	4.839	9.204

Table C2: Internal response at axis - F7 and column - C1 within 2km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	18876.39	18952.63	4504.318	4532.225	7921.447	7967.435	76.245	27.907	45.988
1	16266.41	16334.12	4378.376	4402.274	6615.017	6647.847	67.708	23.898	32.83

2	13706.8	13765.48	4012.862	4032.84	5773.424	5802.274	58.683	19.978	28.85
3	11282.51	11332.26	3539.785	3555.261	5484.676	5513.472	49.744	15.476	28.796
4	8633.69	8673.96	3618.075	3641.825	7301.391	7354.137	40.27	23.75	52.746
5	6588.184	6621.42	3142.422	3164.077	4770.139	4806.523	33.236	21.655	36.384
6	4524.81	4551.185	2779.539	2799.775	3560.859	3593.67	26.375	20.236	32.811
7	2717.658	2737.323	2172.12	2191.8	2326.822	2358.26	19.665	19.68	31.438
8	1298.549	1309.824	1505.199	1523.666	1255.484	1279.68	11.275	18.467	24.196
9	248.558	250.439	333.156	338.461	418.938	428.18	1.881	5.305	9.242

Table C3: Internal response at axis - I2 and column C2’ within 2km distance from source (column under cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	12445.94	12506.21	3028.3	3052.806	5476.229	5516.319	60.276	24.506	40.09
1	9081.186	9133.804	2674.139	2691.142	3946.336	3970.998	52.618	17.003	24.662
2	5783.417	5821.162	2268.858	2283.78	3348.757	3373.298	37.745	14.922	24.541
3	2520.304	2537.376	3172.119	3190.297	4037.196	4065.431	17.072	18.178	28.235

Table C4: Internal response at axis - B2 and column - C3 within 2km distance from source (interior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	8814.142	8820.475	5026.114	5066.874	8616.664	8679.337	6.333	40.76	62.673
1	7434.113	7441.4	4870.536	4902.909	7305.019	7350.242	7.287	32.373	45.223
2	6162.448	6169.209	4455.733	4484.797	6361.799	6408.526	6.761	29.064	46.727
3	5029.835	5035.845	3629.345	3657.154	5856.211	5900.257	6.01	27.809	44.046
4	3863.816	3869.562	4309.546	4336.058	8970.223	9029.697	5.746	26.512	59.474
5	2952.903	2958.181	4077.273	4106.561	6304.712	6359.327	5.278	29.288	54.615
6	2051.384	2056.403	3515.676	3545.68	4459.326	4508.579	5.019	30.004	49.253

7	1273.85	1278.715	2744.976	2773.032	2848.488	2893.559	4.865	28.056	45.071
8	704.716	704.716	1715.184	1741.241	1377.321	1417.778	0	26.057	40.457
9	176.773	176.773	495.815	507.177	764.906	788.285	0	11.362	23.379

Table C5: Internal response at axis - F3 and column - C3 within 2km distance from source (interior column)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5932.365	5932.365	4580.46	4616.087	8040.744	8097.714	0	35.627	56.97
1	5260.951	5260.951	4337.774	4365.981	6492.05	6530.383	0	28.207	38.333
2	4589.501	4589.501	4013.045	4037.239	5715.34	5753.368	0	24.194	38.028
3	3917.77	3917.77	3239.757	3262.036	5274.734	5312.573	0	22.279	37.839
4	3244.624	3244.624	3865.733	3889.953	8048.182	8102.704	0	24.22	54.522
5	2626.278	2626.278	3696.34	3722.266	5707.873	5755.992	0	25.926	48.119
6	2016.239	2016.239	3156.896	3182.986	3943.805	3988.259	0	26.09	44.454
7	1406.319	1406.319	2475.418	2500.832	2496.959	2539.005	0	25.414	42.046
8	796.193	796.193	1530.038	1552.087	1164.05	1196.924	0	22.049	32.874
9	186.914	186.914	464.297	474.375	736.058	757.286	0	10.078	21.228

Table C6: Internal response at axis - A1 and column – C1 within 15km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	12612.1	12627.3	1629.123	1645.571	2864.869	2890.231	15.193	16.448	25.362
1	10844.02	10858.79	1430.394	1441.605	2080.622	2096.486	14.771	11.211	15.864
2	9114.286	9131.381	1284.964	1295.819	1784.964	1803.956	17.095	10.855	18.992
3	7482.745	7497.944	958.837	969.799	1594.953	1611.597	15.199	10.962	16.644
4	5723.231	5737.066	1319.754	1329.201	2821.906	2844.958	13.835	9.447	23.052
5	4376.201	4388.734	1123.664	1133.715	1786.304	1805.904	12.533	10.051	19.6

6	3025.281	3036.529	1031.035	1041.454	1398.3	1414.995	11.248	10.419	16.695
7	1822.02	1831.738	822.451	831.889	954.222	970.412	9.718	9.438	16.19
8	854.821	860.77	581.488	591.757	543.83	559.592	5.949	10.269	15.762
9	153.031	154.074	116.202	119.424	163.217	169.354	1.043	3.222	6.137

Table C7: Internal response at axis - F7 and column - C1 within 15km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	12231.63	12248.15	1415.266	1426.059	2636.017	2653.519	16.521	10.793	17.502
1	10475.63	10492.3	1157.08	1164.18	1688.197	1697.261	16.671	7.1	9.064
2	8761.693	8776.093	1058.981	1064.788	1484.426	1493.215	14.4	5.807	8.789
3	7137.445	7148.648	871.154	875.485	1397.005	1406.166	11.203	4.331	9.161
4	5425.165	5434.403	983.364	990.672	2128.438	2146.396	9.238	7.308	17.958
5	4143.236	4151.583	832.559	838.876	1331.682	1343.174	8.347	6.317	11.492
6	2874.594	2882.688	753.506	759.479	1002.213	1012.737	8.094	5.973	10.524
7	1745.76	1753.577	604.488	610.459	663.842	673.976	7.817	5.971	10.134
8	836.632	841.62	423.424	428.877	345.366	352.506	4.988	5.453	7.14
9	153.273	154.261	74.367	75.777	90.965	93.087	0.988	1.41	2.122

Table C8: Internal response at axis - I2 and column C2' within 15km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	7070.87	7106.644	1611.206	1626.627	2878.487	2901.29	35.774	15.421	22.803
1	5142.18	5174.55	1413.147	1423.378	2051.76	2066.43	32.37	10.231	14.67
2	3274.954	3298.555	1184.822	1193.823	1711.363	1726.385	23.6015	9.001	15.022
3	1448.878	1459.624	1623.311	1633.771	2097.312	2114.518	10.746	10.46	17.206

Table C9: Internal response at axis - F3 and column C3 within 15km distance from source (interior column)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5932.365	5932.365	2421.904	2444.254	4217.444	4252.618	0	22.35	35.174
1	5260.951	5260.951	2259.884	2276.978	3327.035	3349.566	0	17.094	22.531
2	4589.501	4589.501	2045.745	2060.035	2874.303	2897.18	0	14.29	22.877
3	3917.77	3917.77	1632.354	1645.749	2666	2689.064	0	13.395	23.064
4	3244.624	3244.624	1956.476	1971.033	4160.783	4194.398	0	14.557	33.615
5	2626.278	2626.278	1916.158	1932.194	3064.736	3095.442	0	16.036	30.706
6	2016.239	2016.239	1690.081	1706.674	2228.712	2257.783	0	16.593	29.071
7	1406.319	1406.319	1369.275	1385.779	1482.603	1510.459	0	16.504	27.856
8	796.193	796.193	869.861	884.395	728.98	750.871	0	14.534	21.891
9	186.914	186.914	272.923	279.613	448.73	462.858	0	6.69	14.128

Table C10: Internal response at axis - A1 and column C1 above 15km distance from source (column under cantilever slab)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	8023.311	8034.092	958.813	970.118	1671.429	1688.221	10.781	11.305	16.792
1	6927.262	6940.086	842.638	849.655	1218.84	1229.345	12.824	7.017	10.505
2	5961.531	5973.319	753.305	760.91	1049.251	1063.72	11.788	7.605	14.469
3	4849.627	4859.293	568.403	576.71	948.535	961.016	9.666	8.307	12.481
4	3760.393	3769.042	778.131	784.856	1659.637	1675.491	8.649	6.725	15.854
5	2911.762	2919.84	662.287	669.269	1057.42	1070.954	8.078	6.982	13.534
6	2058.517	2066.374	612.84	619.854	846.858	858.169	7.857	7.014	11.311
7	1279.528	1286.826	499.762	506.127	608.975	620.845	7.298	6.365	11.87
8	625.967	630.574	370.629	378.285	378.411	390.71	4.607	7.656	12.299
9	122.577	123.394	82.796	85.309	128.99	131.854	0.817	2.513	2.864

Table C11: Internal response at axis - F7 and column- C1 above 15km distance from source (column under cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	7836.01	7846.897	835.01	841.545	1540.313	1551.34	10.887	6.535	11.027
1	6742.699	6754.279	682.312	686.674	984.36	989.948	11.58	4.362	5.588
2	5679.02	5688.909	616.602	620.182	861.453	867.279	9.889	3.58	5.826
3	4666.928	4674.173	507.089	510.136	821.818	828.388	7.245	3.047	6.57
4	3601.469	3607.381	576.899	581.835	1261.141	1272.906	5.912	4.936	11.765
5	2786.179	2791.778	492.461	496.516	801.303	808.438	5.599	4.055	7.135
6	1977.11	1983.028	451.656	455.314	620.333	627.329	5.918	3.658	6.996
7	1239.172	1245.04	370.882	374.833	432.335	439.786	5.868	3.951	7.451
8	617.932	621.858	269.159	273.067	240.886	246.376	3.926	3.908	5.49
9	123.429	124.214	47.855	48.927	68.764	70.434	0.785	1.072	1.67

Table C12: Internal response at axis - I2 and column - C2' above 15km distance from source (column under cantilever slab)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	4685.152	4709.293	947.857	958.299	1673.007	1689.245	24.141	10.442	16.238
1	3423.826	3447.659	837.774	844.134	1212.076	1221.85	23.833	6.36	9.774
2	2198.308	2216.3	698.895	704.876	1009.926	1021.159	17.992	5.981	11.233
3	991.827	1000.086	958.759	966.048	1247.929	1260.144	8.259	7.289	12.215

Table C13: Internal response at axis - B2 and column - C3 above 15km distance from source (interior column)

	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
story	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5154.5	5154.5	1561.235	1578.602	2638.096	2663.656	0	17.367	25.56

1	4570.967	4570.967	1484.037	1496.294	2177.91	2194.936	0	12.257	17.026
2	3989.018	3989.018	1319.827	1331.695	1859.03	1880.258	0	11.868	21.228
3	3409.592	3409.592	1066.405	1078.99	1728.453	1748.398	0	12.585	19.945
4	2831.481	2831.481	1274.274	1285.669	2708.958	2734.059	0	11.395	25.101
5	2292.759	2292.759	1239.022	1251.522	1994.213	2018.012	0	12.5	23.799
6	1762.883	1762.883	1113.495	1126.264	1519.392	1541.242	0	12.769	21.85
7	1233.624	1233.624	917.736	930.072	1074.343	1097.299	0	12.336	22.956
8	704.716	704.716	618.257	630.966	600.893	621.904	0	12.709	21.011
9	176.773	176.773	193.148	198.97	323.406	335.591	0	5.822	12.185

Table C14: Internal response at axis - F3 and column - C3 above 15km distance from source (interior column)

story	Axial force (kN)		Shear force (kN)		Bending moment (kN-m)		Variation response due to VEGA		
	H	H+V	H	H+V	H	H+V	Axial	Shear	Bending
0	5932.365	5932.365	1418.479	1433.44	2457.651	2480.473	0	14.961	22.822
1	5260.951	5260.951	1310.631	1321.286	1915.855	1929.991	0	10.655	14.136
2	4589.501	4589.501	1177.775	1187.216	1654.867	1671.631	0	9.441	16.764
3	3917.77	3917.77	940.449	950.243	1544.591	1561.551	0	9.794	16.96
4	3244.624	3244.624	1130.212	1140.64	2415.041	2437.82	0	10.428	22.779
5	2626.278	2626.278	1111.543	1122.365	1793.172	1813.314	0	10.822	20.142
6	2016.239	2016.239	987.628	998.442	1335.347	1355.048	0	10.814	19.701
7	1406.319	1406.319	815.275	826.433	933.141	953.662	0	11.158	20.521
8	796.193	796.193	532.873	543.507	494.662	511.69	0	10.634	17.028
9	186.914	186.914	174.905	180.048	300.469	311.51	0	5.143	11.041

Appendix D

Internal Response comparison due to variation of Soil Profile on effect of VEGA on C1, C2' & C3 (axis A1, F7, F3 & I2)

Table D1: Internal response comparison at axis - A1 and column - C1 due to variation of soil profile (column under cantilever slab)

story	axial force difference on dense soil (kN)	axial force difference stiff soil(kN)	shear force difference on dense soil(kN)	shear force difference on stiff soil(kN)	bending moment difference on dense soil(kN-m)	bending moment difference on stiff soil(kN-m)
0	15.193	16.939	16.448	18.2	25.362	28.13
1	14.771	19.69	11.211	12.45	15.864	17.656
2	17.095	18.9	10.855	12.05	18.992	21.03
3	15.199	16.78	10.962	12.15	16.644	18.44
4	13.835	15.259	9.447	10.51	23.052	25.54
5	12.533	13.807	10.051	11.13	19.6	21.63
6	11.248	12.38	10.419	11.47	16.695	18.4
7	9.718	10.69	9.438	10.41	16.19	17.82
8	5.949	6.54	10.269	11.31	15.762	17.34
9	1.043	1.14	3.222	3.54	6.137	6.75

Table D2: Internal response comparison at axis - F7 and column C1 due to variation of soil profile (column under cantilever slab)

story	axial force difference on dense soil (kN)	axial force difference stiff soil(kN)	shear force difference on dense soil(kN)	shear force difference on stiff soil(kN)	bending moment difference on dense soil(kN-m)	bending moment difference on stiff soil(kN-m)
0	16.521	18.62	10.793	11.27	17.502	19.49
1	16.671	18.64	7.1	7.93	9.064	10.17
2	14.4	16.06	5.807	6.53	8.789	9.85
3	11.203	12.48	4.331	4.82	9.161	10.19
4	9.238	10.26	7.308	8.19	17.958	20.03
5	8.347	9.23	6.317	7.05	11.492	12.76
6	8.094	8.93	5.973	6.65	10.524	11.63
7	7.817	8.38	5.971	6.61	10.134	11.16
8	4.988	5.49	5.453	6.019	7.14	7.86
9	0.988	1.089	1.41	1.55	2.122	2.32

Table D3: Internal response comparison at axis - I2 and column - C2' due to variation of soil profile (column under cantilever slab)

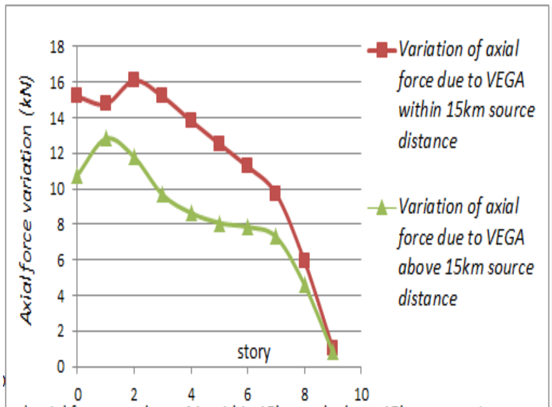
story	axial force difference on dense soil (kN)	axial force difference stiff soil(kN)	shear force difference on dense soil(kN)	shear force difference on stiff soil(kN)	bending moment difference on dense soil(kN-m)	bending moment difference on stiff soil(kN-m)
0	35.774	39.87	15.421	17.07	22.803	27.51
1	32.37	35.92	10.231	11.38	14.67	16.34
2	23.6015	26.15	9.001	10.01	15.022	16.68
3	10.746	11.89	10.46	11.7	17.206	18.62

Table D4: Internal response comparison at axis - F3 and column - C3 due to variation of soil profile (interior column)

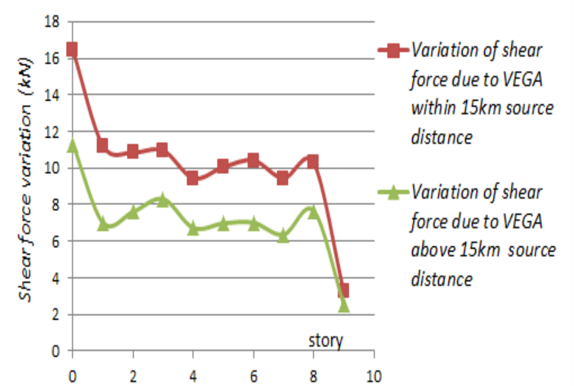
story	axial force difference on dense soil (kN)	axial force difference stiff soil(kN)	shear force difference on dense soil(kN)	shear force difference on stiff soil(kN)	bending moment difference on dense soil(kN-m)	bending moment difference on stiff soil(kN-m)
0	0	0	22.35	23.878	35.174	39.01
1	0	0	17.094	19	22.531	25.15
2	0	0	14.29	15.93	22.877	25.46
3	0	0	13.395	14.91	23.064	25.62
4	0	0	14.557	16.2	33.615	37.29
5	0	0	16.036	17.79	30.706	33.93
6	0	0	16.593	18.35	29.071	32.04
7	0	0	16.504	18.21	27.856	30.66
8	0	0	14.534	16	21.891	24.08
9	0	0	6.69	7.37	14.128	15.54

Annex A

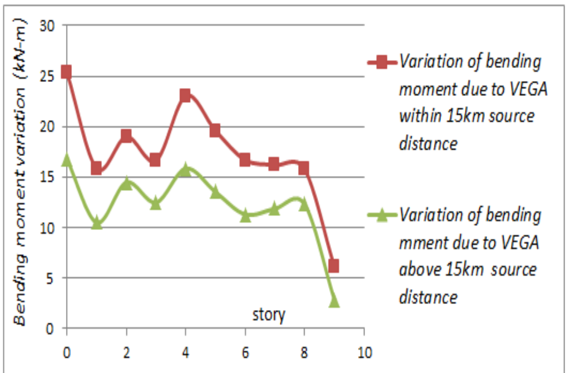
Building Response at near Source due to VEGA



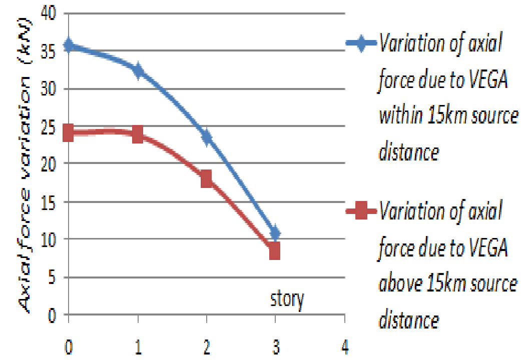
a) axial force at column A1 within 15km and above 15km source distance



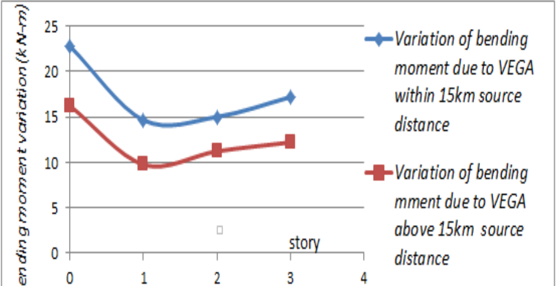
b) shear force at column A1 within 15km and above 15km source distance



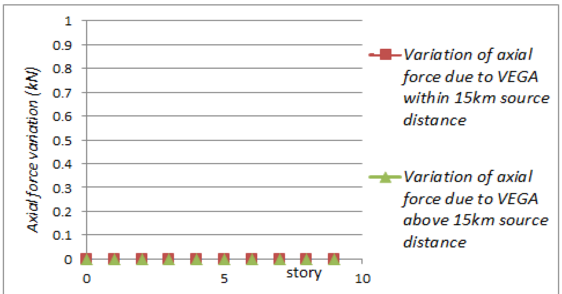
c) bending moment at column A1 within 15km and above 15km source distance



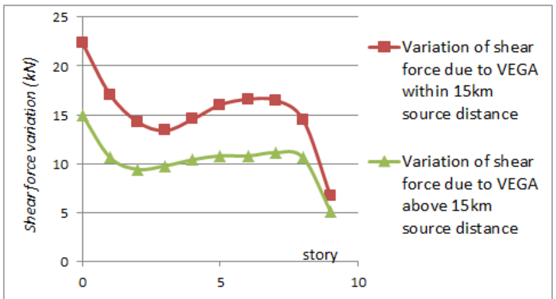
g) axial force at column I2 within 15km and above 15km source distance



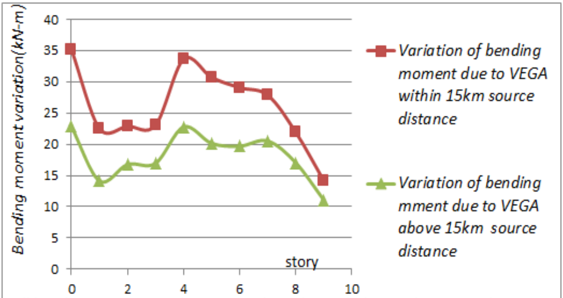
i) bending moment at column I2 within 15km and above 15km source distance



j) axial force at column F3 within 15km and above 15km source distance

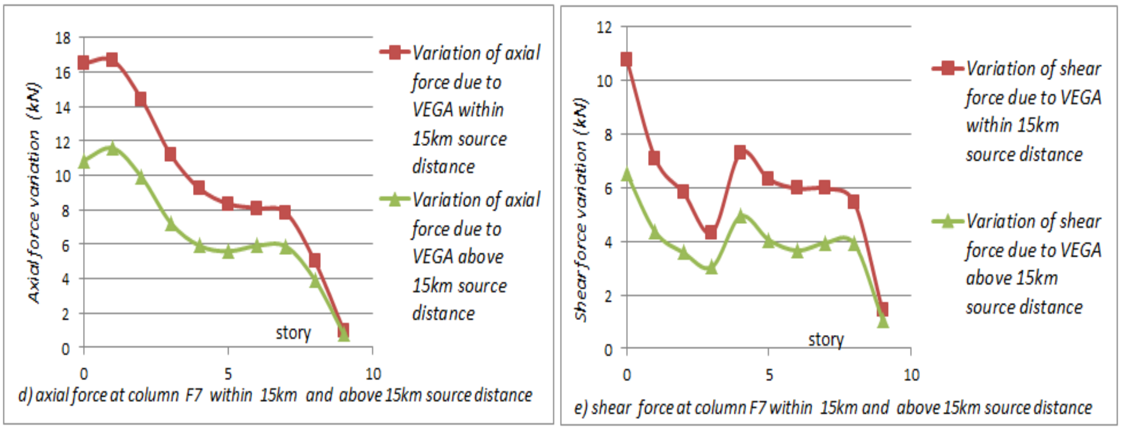


k) shear force at column F3 within 15km and above 15km source distance



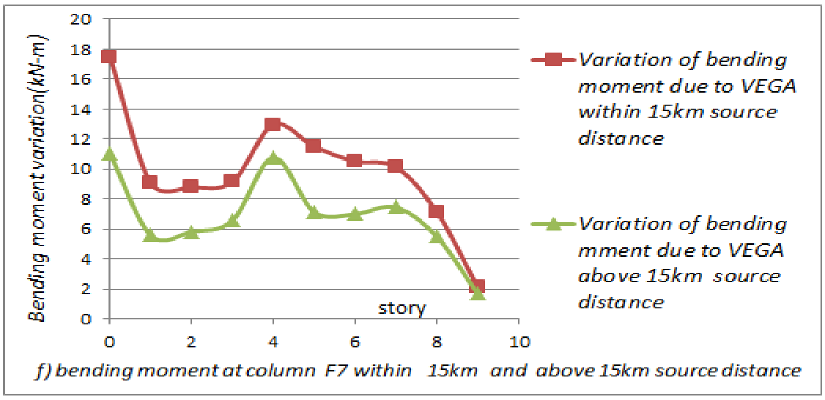
l) bending moment at column F3 within 15km and above 15km source distance

Figure A: Effect of VEGA on building response due to difference source distance



d) axial force at column F7 within 15km and above 15km source distance

e) shear force at column F7 within 15km and above 15km source distance

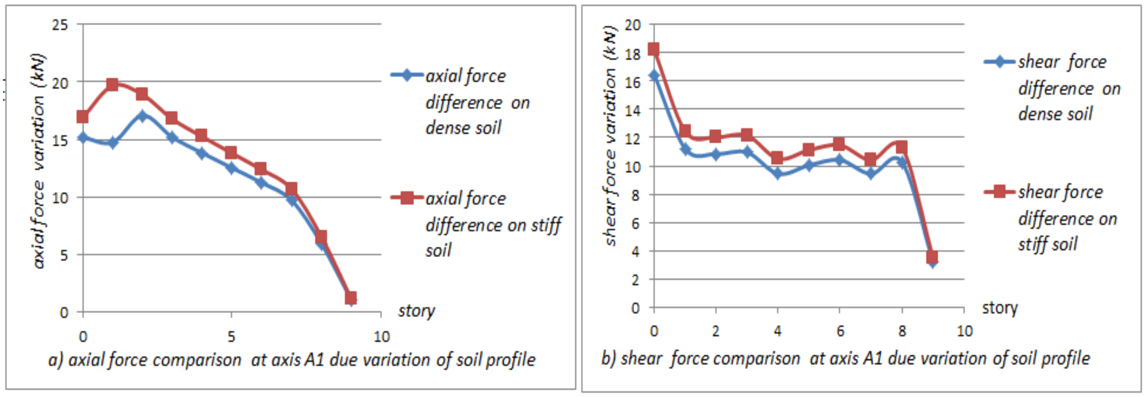


f) bending moment at column F7 within 15km and above 15km source distance

Figure A: Effect of VEGA on building response due to difference source distance cont.…

Annex B

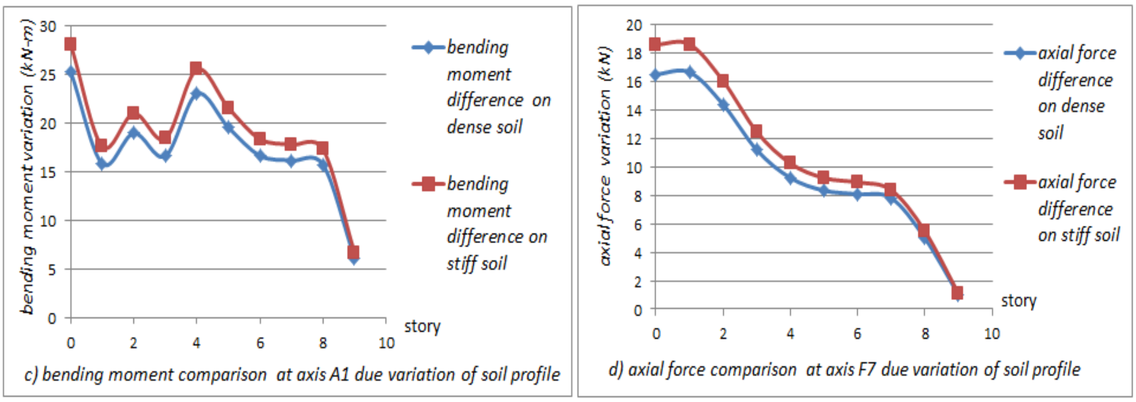
Internal Response Comparison between Dense and Stiff Soil Profile



a) axial force comparison at axis A1 due variation of soil profile

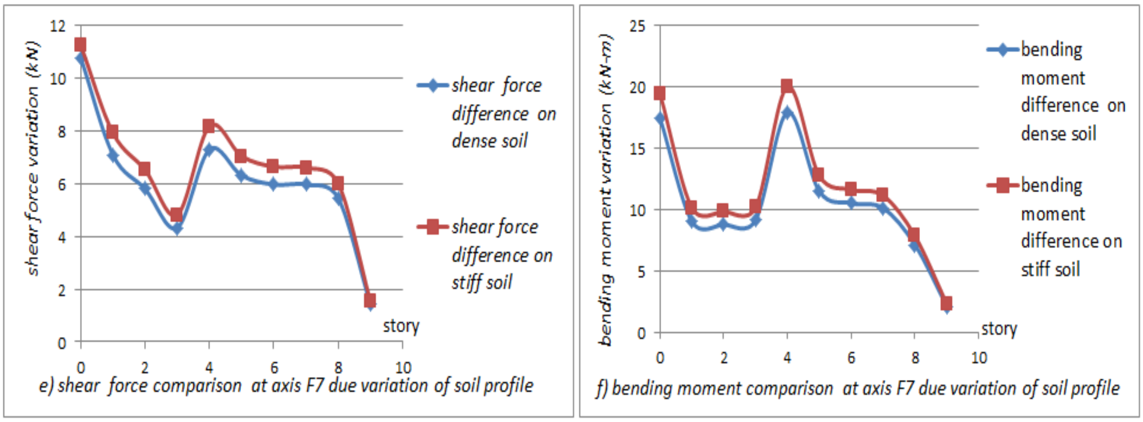
b) shear force comparison at axis A1 due variation of soil profile

Figure B: Structural response comparison between dense and soft rock and stiff soil at source distance within 15km



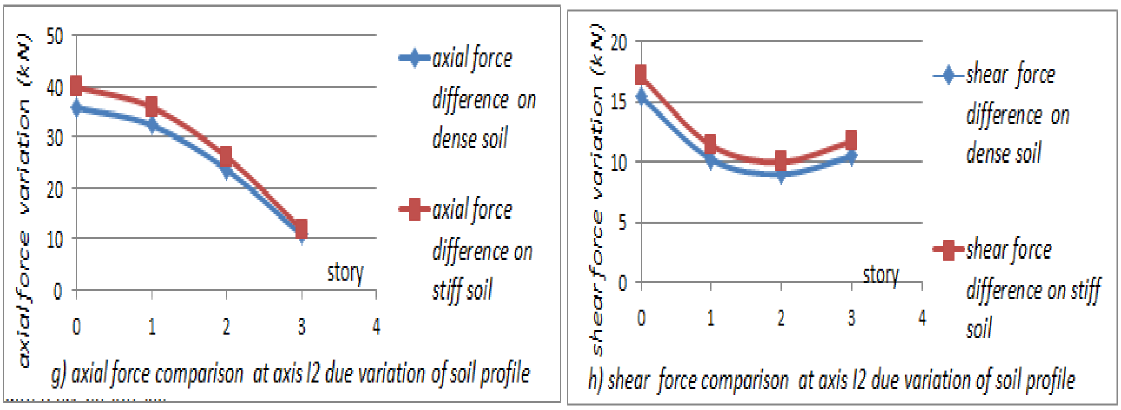
c) bending moment comparison at axis A1 due variation of soil profile

d) axial force comparison at axis F7 due variation of soil profile



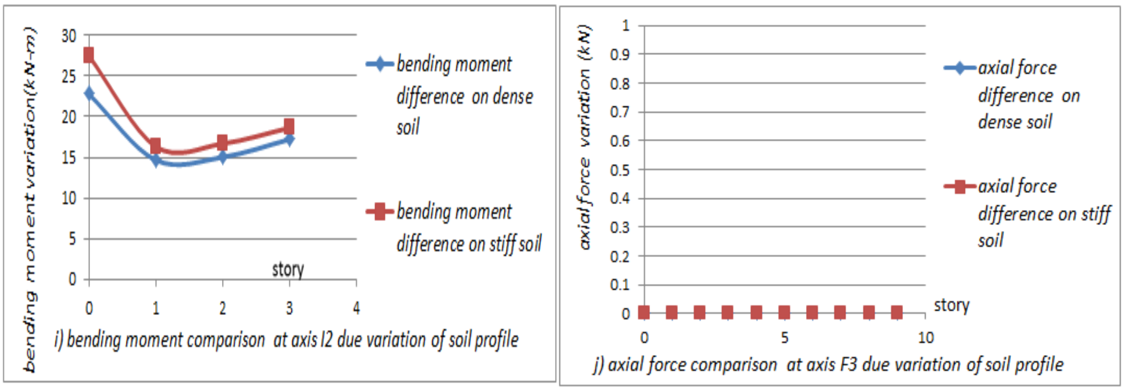
e) shear force comparison at axis F7 due variation of soil profile

f) bending moment comparison at axis F7 due variation of soil profile



g) axial force comparison at axis I2 due variation of soil profile

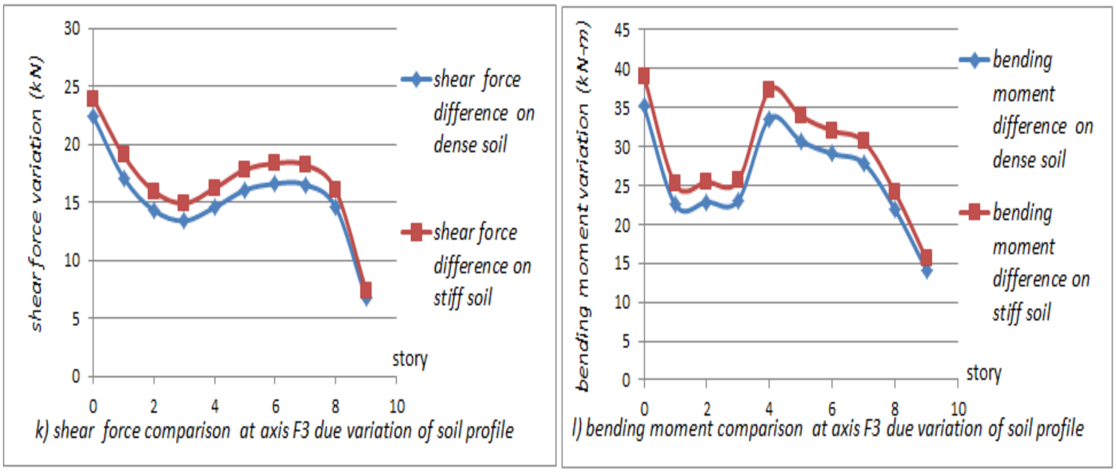
h) shear force comparison at axis I2 due variation of soil profile



i) bending moment comparison at axis I2 due variation of soil profile

j) axial force comparison at axis F3 due variation of soil profile

Figure B: Structural response comparison between dense and soft rock and stiff soil at source distance within 15km cont.…



k) shear force comparison at axis F3 due variation of soil profile

l) bending moment comparison at axis F3 due variation of soil profile

Figure B: Structural response comparison between dense and soft rock and stiff soil at source distance within 15km cont.…