Geotech Geol Eng (2016) 34:1013–1028
DOI 10.1007/s10706-016-0021-y

ORIGINAL PAPER

Response of Strata and Buildings to Blast Induced Vibrations in the Presence and Absence of a Tunnel

Nalini E. Rebello · R. Shivashankar · V. R. Sastry

Received: 24 July 2015 / Accepted: 26 April 2016 / Published online: 3 May 2016

Abstract Blast induced vibrations form an inevitable and major part of modern day construction. The changes that happen to the strata or buildings surrounding the blast are evident in a fraction of a second. Effect of damage is more pronounced in the absence/presence of the tunnel. The vibration produced due to blast may be induced due to a deep underground explosion, a surface explosion or even an in-tunnel explosion. In this study the above three situations are numerically modeled by a Distinct Element software 3DEC (3.0). Soil properties are varied representing soft and stiff strata. Further, three velocity time histories of 2, 45 and 85 Hz are used as an input in the model and are applied at three different boundaries of the model. Results of the analysis reveal that the response of building in softer strata and lower frequencies led to greater magnification of velocities and displacements compared to response of buildings in stiff strata. Presence of the tunnel led to reduction of peak velocity (PV's) and displacements at the building top due to damping effect. PV's at the top floor were greater than the PV's at the bottom floor and there was an upliftment of the soil mass at the ground level. However, the upliftment in the presence of the building was lower than the upliftment in the absence of the building. Stress in the tunnel lining increased in the presence of the building, however percentage reduction of stress depends on the number of building stories.

Keywords Numerical modelling · 3DEC · Velocity time history · PV

N. E. Rebello (✉) · R. Shivashankar
Department of Civil Engineering, NITK, Surathkal, India
e-mail: nalinirebello@rediffmail.com

V. R. Sastry
Department of Mining Engineering, NITK, Surathkal, India

1 Introduction

Blast induced vibrations usually characterized by predominant vertical velocities rather than horizontal velocities may lead to damage in the surrounding soil mass specially in and around buildings. The effect of vibrations is more significant in the presence of tunnels. Although a few researchers have carried out studies on dynamic impact loads (Liu 2009; Tian and Li 2008; Wei et al. 2011) they are very few as compared to research carried due to earthquake loads (Dowding and Rozen 1978; Hashash et al. 2001; John St and Zahrah 1987; Wang et al. 2001; Liu and Song 2005).

Dynamic impact loads are different from earthquake loads as they last only for a few seconds/milliseconds and are subjected to very high frequencies unlike earthquake loads which last for several seconds and have low frequencies. Further blast PV's are characterized by dominant vertical velocities compared to horizontal velocities and therefore these vertical vibrations may induce damage to buildings which are different than those produced due to earthquake loads. Displacements generated due to earthquake loads is characterized by inter-storey drift, between floor to floor, whereas displacements generated due blast waves produce a degradation of the different components of structure, Guowei et al. (2011). Also, if we notice the characteristics of a blast wave generated due to the explosion of known quantity of explosives, it is triangular in shape as illustrated in Fig. 1.

Previous researchers using impact loads varied the quantity of explosive used in a blast and the burial depths of tunnel in different strata conditions. Liu (2009) carried out studies on small sized tunnels, with rocks, sandy soils and saturated soft soil as the surrounding strata of subway structures, subjected to pressure at an appropriate location. Effects of varying quantity of explosives and different subway burial depths were also studied. High quantities of explosive increased the damage in lining of the tunnel and surrounding strata. The increase was more predominant in softer strata than in stiffer strata as stiffer strata like rocks are capable of withstanding higher stresses and causing less damage. With a modest internal explosion, increase in burial depths of the substructure led to lesser damage, as at greater depths from the surface, strata surrounding the subway confines the structure, hence lining stresses are less and damage is not severe. Similar studies were conducted by Wei et al. (2011) who subjected a subway station to blast loads. Parameters used in the analysis were depth of overburden, soil stiffness and quantity of explosives.

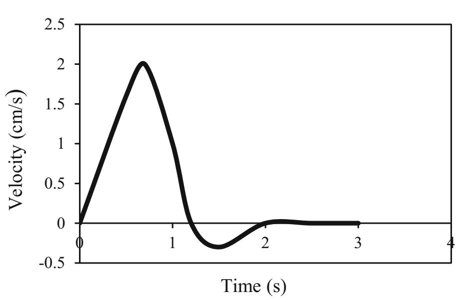


Fig. 1 Typical velocity history generated due to a blast

Conclusions drawn from the analysis were similar to the conclusions drawn by Liu (2009).

In the past few years, damage assessment in various parts of the structure was based on the PPV's (Peak Particle Velocity) of the input blast wave motion/-ground wave motion. Based on field observations by several researchers, structural damage was related to either the PPV or to the principal ground vibration frequency.

Masonry structures are more vulnerable to damage in terms of cracking or settlements as compared to a rigid reinforced concrete structure. Edwards and Northwood (1960), Nicholls et al. (1971), Odello (1976); described three levels of cracking in masonry structures subjected to experimental blast vibrations. Hairline cracks which did not affect the strength of masonry appeared at 114 mm/s. Threshold cracking appeared at 176 mm/s, featured opening of old cracks and formulation of new plaster cracks. Major cracking resulting in serious weakening of the structure appeared at 203 mm/s. Although the field observation of the blast related damage, conducted by several researchers were personal observations, the database of damage vs PPV is still a major reference for engineers in blast related activities. The U.S. Department of Defense Explosive Safety Board and NATO blast safety code which specifies safety code on underground ammunition storage 1996, states that for protection of residential buildings against significant structural damage by ground shock, the maximum particle velocity induced in the ground at the building site may not exceed 60 mm/s in soil, 114 mm/s in soft rock, and 230 mm/s in hard rock. Jiang and Zhou (2012) simulated relationship between stresses in the rock mass and tunnel structure, under different blasting conditions and thus obtained a blasting safety criterion based on PPV. An overview of the vibration standards adopted by a few countries like USSR indicate that vibration limits in single storey structures are permitted to have PPV's of 240 mm/s (Table 1). However, it is not just the PPV of the input wave motion that has a major impact but also the vibration frequency of both the structure and the input frequency of the waves play a significant role.

The focus of this study is to estimate displacements generated due to vertical component of blast wave. Three actual velocity histories (of different frequencies) generated due to explosion at a site in South

Table 1 USSR standard

Type of structures	Allowable PPV (mm/s)	
	Repeated	One-fold
Hospitals	8	30
Large panel residential buildings and children's institutions	15	30
Residential and public buildings of all types except large panels, office and industrial buildings having deformations, boiler rooms and high brick chimneys	30	60
Office and industrial buildings, high reinforced concrete pipes, railway and water tunnels, traffic flyovers	60	120
Single storey skeleton type industrial buildings, metal and block reinforced concrete structures. Soil slopes which are part of primary structures, primary mine openings (service life up to 10 years) pit bottoms, mine entries, drifts	120	240

India, are applied at three different locations of the model, and Peak Velocities (PV's), displacements and stress concentration, during the event, is noted at different target points in the model. As only a linear elastic model is considered, the actual damage pattern in different components of structure is not simulated. Results of the analysis reveal that the response of building in softer strata and lower frequencies led to greater amplification of velocities and displacements compared to response of buildings in stiff strata. PV's and displacements at different locations reduced due to the damping effect created by the opening. PV's at the top floor were greater than the PV's at the bottom floor and there is an upliftment of the soil mass at the ground level. Vertical stress in the tunnel lining increased on inclusion of building loads which is mainly because of elongation of the building. However, with an increase in storey there was decrease in stress concentration in the tunnel lining.

2 Problem Definition and Details about Numerical Simulation

Numerical Modelling is conducted using the 3 Dimensional Distinct Element Code (3DEC) software to predict displacements. While Finite element codes are more applicable to continuum based approaches, Distinct element codes are suitable for strata with a number of discontinuities (geological or geometrical) which is applicable to the current case.

The tunnel under consideration is a tunnel at a metro site in South India with 6.1 m diameter and located at a depth of 8 m from surface (Fig. 2). The thickness of the lining of the tunnel is 0.25 m. Based on Indian Standard Code IS 456 (2000), M30 grade of concrete is considered for the building and M25 grade is considered for the lining of the tunnel and details of the tunnel liner are illustrated in Sect. 2.1. The material model considered for the strata was an elasto-plastic model with Mohr-Coloumb failure criterion. A linear-elastic constitutive model was assigned for the tunnel liner and building. Although a linear elastic model might have its limitations of not capturing the plastic behavior of the soil/structure, it definitely provides a tool for assessing the amplification in PV's, displacements and the like. The joints between soil-concrete and concrete–concrete interfaces in the model are assigned normal and shear stiffness values. The joints have a linear stress-displacement relationship and thus the joint was modeled to have high enough shear and normal strength not to crack.

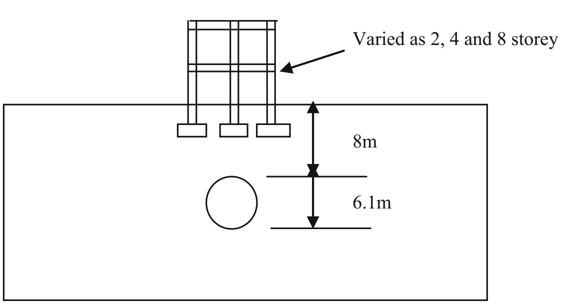


Fig. 2 A sketch of the model

For the static analysis, fixed boundaries were provided at the bottom and at the sides roller boundaries are provided. To reduce the effect of artificial boundaries, a distance of 8 D was provided (where 'D' is the diameter of tunnel) at the sides in the transverse direction and a distance of 17 m was provided from tunnel bottom to the bottom of the model. The entire domain was divided into deformable blocks with each block further discretized into tetrahedrons with predominant geometrical/geological features forming boundaries of different blocks. The average length of each element was 1.5 m. Finer mesh refinement was provided for the building with an average element size of 0.5 m.

Material properties, boundary conditions and gravity loads are assigned to the model, and model is brought to a state of static equilibrium prior to excavation. The tunnel is excavated, model is run for a few more cycles such that the unbalanced forces existing in the model is reduced and once again brought to a state of static equilibrium. For the dynamic analysis, viscous boundary developed by Lysmer and Kuhlemeyer (1969) which also act as absorbent boundaries, are used. Viscous boundaries are applied to the side and bottom boundaries of the model. Thus the blast waves are effectively transmitted through the strata without reflecting back into the model. In order to ensure that load is not suddenly applied to the model, then a time step lesser than the time interval between each event are applied. Hence, time step used for the analysis was $\Delta t = 0.0096$ s and total duration of the blast is1 s.

Further, natural frequencies of the soil mass are initially found out. Natural frequency of strata in the presence of tunnel, natural frequency of strata in presence of building and natural frequency of strata in the presence of both tunnel and building are evaluated. Amplification of ground vibration is evident at frequencies close to the natural vibration frequencies of the structure and therefore it is essential to investigate natural frequency of the building prior to actual application of dynamic loads.

For the dynamic analysis damping is taken into consideration. Damping is a function of type of building construction. Measurements have revealed a wide range of damping for residential structures with an average of 5 % (Dowding et al. 1980). Several researchers have used varied damping values for different components of structures. Tian and Li (2008) and Liu (2009) have used 2 and 5 % viscous damping for the soil mass and tunnel respectively. Although damping in soil mass and structure may be different, damping including mass proportional damping and stiffness proportional damping of 5 % is assigned for both the strata and tunnel in this study.

To evaluate the most significant factors affecting displacements, parametric study is conducted in four stages

2.1 Effect of Two Soil-Structure Types

The study focuses on predicting the effect of blast loads of a tunnel in a specific soil type. Therefore, the properties of soil strata which surrounded the tunnel site are assigned for the entire depth and width of the model. Two types of soil-structure were selected, representing a soft soil-stiff structure (SS-1) and stiff soil-stiff structure (SS-2), in order to understand the effect of tunnel-soil-structure interaction on the PV response, concentration of stress and displacements during the event (Table 2). Material properties assigned for the tunnel, structure and soil are illustrated in Table 2. Similarly in Distinct Element codes like 3DEC the material properties have an important bearing in deformations and therefore material properties of different joints are used in Table 3. The joint stress properties described by Kulhawy (1975) suitable for various types of joints are assigned ensuring the joints have a linear stress-displacement relationship, thus having enough normal and shear strength not to crack.

2.2 Varied Frequency of Blast Wave

Structures with varying stiffness respond differently to varied ground motions induced from blasts. Hence, three different velocity waves of frequency 2, 45 and 85 Hz are introduced in the model and the effect of their response was studied. A velocity wave propagating in the vertical direction was taken into consideration. Thus in the first case a blast wave of PV 21.5 mm/s and frequency of 45 Hz (Fig. 3) are applied. Second case a blast wave of PV 45 mm/s and frequency of 2 Hz are applied (Fig. 4). In the third case a blast wave of frequency 85 Hz and PV of 110 mm/s are applied (Fig. 5).

Table 2 Properties assigned to two different soil-structure types

Type of soil	Material	Bulk modulus (MPa)	Shear modulus (MPa)	Density (kN/m^3)	Cohesion (kPa)	Poisson's ratio
SS1	Soil	1300	285	21	9	0.4
	Structure	14,200	11,000	25		0.18
	Tunnel liner	13,000	10,500	24		0.18
SS2	Soil	3300	1100	22	12	0.35
	Structure	14,200	11,000	25		0.18
	Tunnel liner	13,000	10,500	24		0.18

Table 3 Properties assigned to different joints in the model

Interface	kn (MPa/m)	ks (MPa/m)	Ø
Concrete and concrete	9000	900	45
Soil and concrete	600	80	35

2.3 Varied Positions of Input Blast Wave

Variation of point of application of blast wave has an important bearing on the velocities developed. Waves with the particle motion in the vertical plane are applied coincident with three different boundaries of the model. The blast waves generated due to actual explosion during the construction of cut-cover tunnel in hard rock formations is taken up as input. Hence, effect of blast waves due

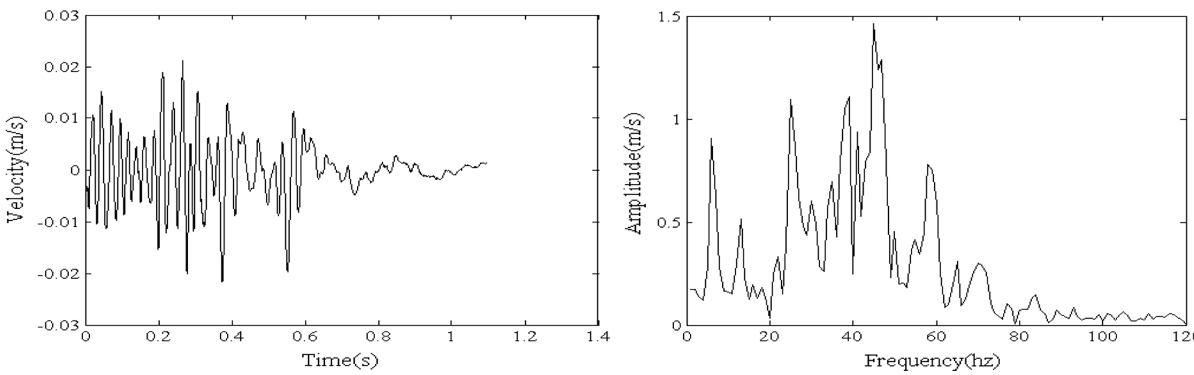


Fig. 3 Velocity time history of wave of PV 21.5 mm/s (frequency 45 Hz)

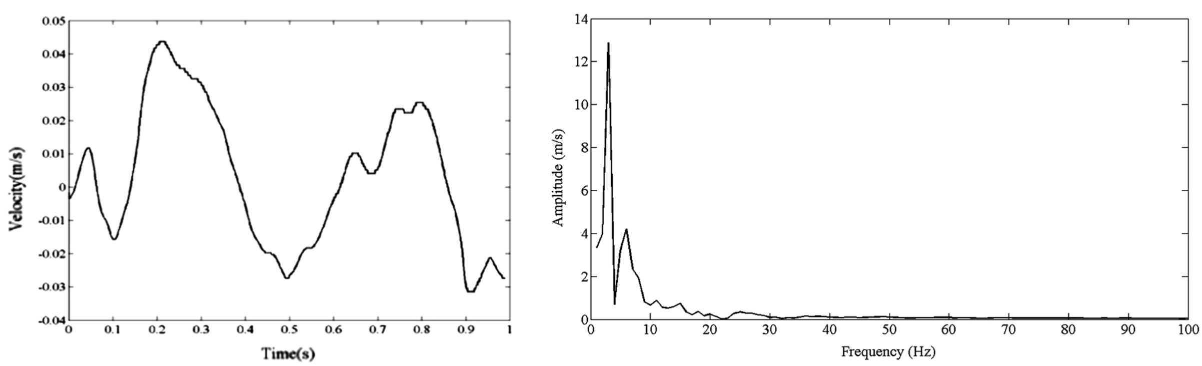


Fig. 4 Velocity time history of wave of PV 45 mm/s (frequency 2 Hz)

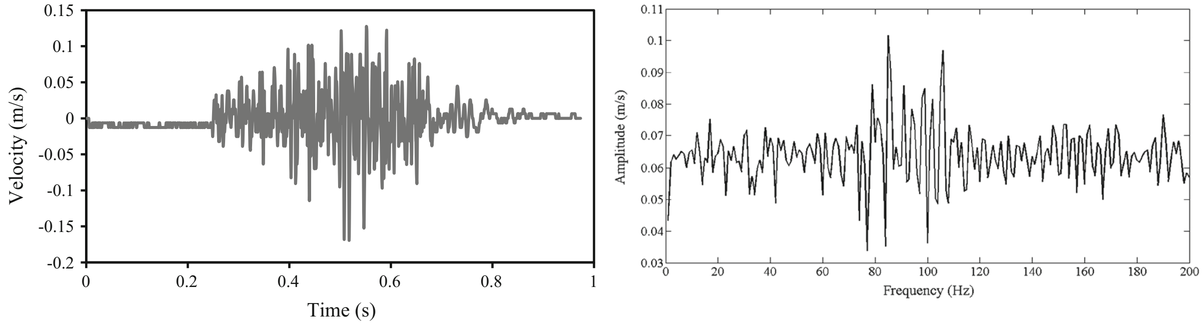


Fig. 5 Velocity time history of wave of PV 110 mm/s (frequency 85 Hz)

to application at three boundaries of the model is evaluated. In the first case, blast wave is applied at the bottom boundary of the model assuming a deep underground explosion to have taken place. In the second case blast wave is applied at the crown of the tunnel assuming an in-tunnel explosion. In the third case the blast wave is applied at surface just below the structure to investigate the response of the structure, tunnel and strata surrounding it.

2.4 Varied Building Stories

A framed 2, 4 and 8 storey building without brick-infill walls was considered for the analysis. Columns were of size 0.35 m × 0.45 m with an axial stiffness of 128MN. Slab is assigned a thickness of 0.15 m. Beams have cross-sectional dimension of 0.3 m × 0.35 m with axial stiffness of 85.8 MN and bending stiffness 0.876 MN-m^2. Even though the above mentioned dimensions are characteristics of structures with more number of floors, slightly oversized beams and columns were provided to facilitate ease in modeling. The footings were of dimensions 2 m × 2 m with a thickness of 0.5 m. A distance of 4 m was provided from the center line of one footing to the other, both in the transverse as well as longitudinal direction.

3 Results and Discussion

3.1 Influence of Varied Soil Types and Building Stories

3.1.1 Variation in PPV and Displacements in the Beams of Building Considered as Base Case

Natural frequency of 2, 4 and 8 storey building is 0.75, 0.6 and 0.45 Hz on interaction with SS-1 and 1, 0.75 and 0.6 Hz in presence of SS-2. Effect of soil stiffness on the PV response of the building in the absence of the tunnel is studied. PV and displacement response of the building to a vertically propagating blast vibration load of frequency 45 Hz is investigated. Since blast wave do not produce horizontal drifts compared to vibrations produced by seismic excitations, only vertical displacement at various floor of the building were monitored. Further, the horizontal displacements is relatively small compared with the vertical displacements and hence ignored. The blast wave is applied at the base of the model. In order to understand the PV's generated, different target points were selected for the analysis (Fig. 6). It can be seen that calculated PV's at the top of the two, four and eight storey building are 356 mm/s (at B2), 367 mm/s (at B4) and 401 mm/s (at B8) in presence of SS-1 (Fig. 6). The peak velocity is 328.4 mm/s at B2 of 2 storey building which increased to 337 mm/s for the four storey structure (at B4) and further increased to 340.3 mm/s for the eight storey structure when building interacts with SS-2. The relationship between the input blast wave and subsequent PV's and displacements are influenced by the interaction between soil and building. As a linear elastic model without brick-infill walls was considered, the PV's generated are of higher magnitude as compared to practical considerations. The Peak Velocities were higher in the top floors than the bottom floors which is predominantly due to greater cantilever action at the top floors than the bottom floors. Reduction in PV's in SS-2, indicate that structures in stiff soils are less vulnerable to damage than structures in soft soil conditions. The corresponding displacements at the same points of observation are 15.72 (at B2), 17.88 (at B4) and 24.36 mm (at B8) respectively for SS-1 and

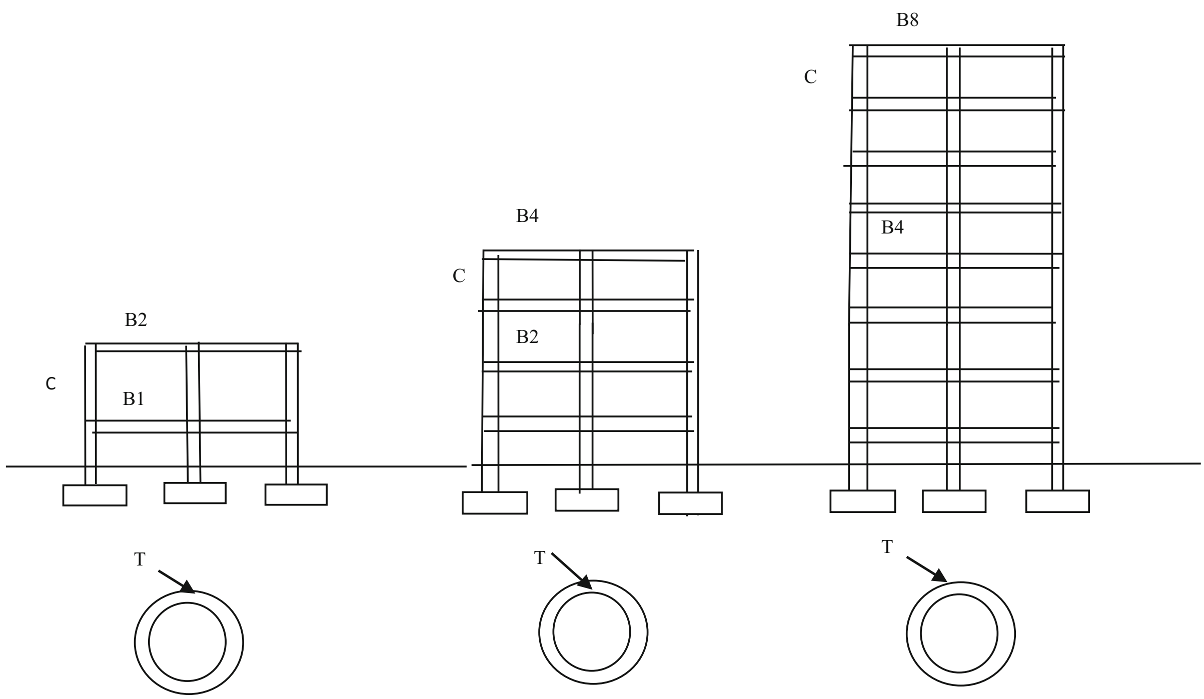


Fig. 6 Various target points in the model

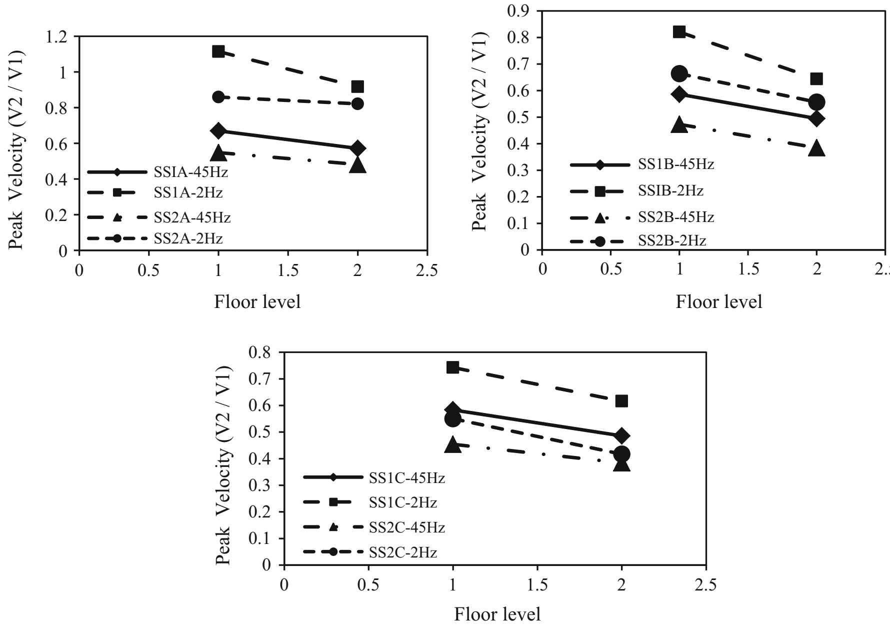
4.6 (at B2), 7.38 (at B4) and 13.45 mm (at B8) respectively for SS-2. With an increase in storey, buildings interacting with SS-1 and SS-2, led to increase in PV's and displacements. Greater cantilever action is noticed at the free end or the top end of building. Magnification of velocities at higher floors as compared to lower floors was consistent with the results obtained by experimental studies conducted by Singh and Roy (2010).

3.1.2 *Variation in PPV in the Beams of the Building in the Presence of Tunnel*

The stiffness of soil-structure in the presence of tunnel on the PV response of the building is evaluated. Response of the building to a vertically propagating wave of frequency 45 Hz, applied at the crown of the tunnel, is studied. Representative graphs for absolute maximum PV's at various locations of the 2, 4 and 8 storey building are shown in Fig. 7.

PV's are of magnitude 167.96 and 173 mm/s in 1st and 2nd floors of the two storey structure, 201.78 and 216.18 mm/s for the 2nd floor and 4th floor of the four storey structure and 218.62 and 245.76 mm/s at the 4th and 8th floor of 8 storey structure. In case of buildings which interact with SS-1, the magnitude of displacements was 6.04, 7.06 and 7.41 mm at the top most beam of the 2, 4 and 8 storied structure. PV's are of magnitude 115.3 and 126.19 mm/s at B1 and B2 of the two storey structure, 128.21 and 146.7 mm/s in B2 and B4 of the 4 storey structure and 131.26 and 154.32 mm/s in B4 and B8 of the 8 storey structure which interact with SS-2. Displacements are of magnitude 1.82, 1.99 and 4.22 mm at the topmost beam of the 2, 4 and 8 storied building which interact with SS-2. Irrespective of the soil type, taller structures led to greater amplification of PV and displacements compared to short structures. PV's in the top beam of the building, in the presence of tunnel, reduced by 32.2, 19.15 and 18.37 % compared to the PV's developed in the absence of tunnel (SS-1). Reduction in PV's at the topmost beam of building, compared to the case of building without tunnel was 61.57, 56.46 and 54.65 % for two, four and eight storey, respectively in SS-2. This indicates that the presence of the tunnel has a marked reduction of PV's and displacements in a building which is mainly due to the damping effect created by the opening or the formation of void in the soil mass. The effect is more pronounced as the tunnel diameter of 6.1 m, which is

Fig. 7 Ratio of absolute maximum PV's at B1, B2 of 2 storey building



almost equal to the total base width from column to column of the building, of 8 m, does not allow the wave to propagate upwards. A smaller diameter of the tunnel or larger base width of the building i.e. from outer columns would have led to insignificant reduction in PV's.

Ratio of absolute maximum PV's generated at the different floor levels in the presence and absence of tunnel are presented in Fig. 7. (SS1A in graph indicates, soil type is SS-1 and point of application is at the bottom of the model, SS1B indicates the point of application is at the surface of model in SS-1, similarly SS1C indicates point of application is at the crown of the tunnel in SS-1). Ratio of PV's in the top floor with considering tunnel, to the PV's generated without tunnel, was 0.485(SS-1) and 0.385(SS-2) (Fig. 7). (In Fig. 7, 'V2' indicates Peak velocity in the presence of tunnel and building and 'V1' indicates Peak velocity in presence of only building). Further, for a given soil condition, amplification of PV's in top floors was greater than the amplification of bottom floors with a value of 0.58 and 0.485 in 1st floor and 2nd floor of two storey structure in SS-1 and 0.45 and 0.385 in SS-2. Comparison of peak velocity of buildings indicate that with an increase in storey there is increase in PV's and displacements at the top most beam (B2, B4 and B8) of the structure.

3.2 Influence of Varied Point of Application of Input Wave

Vertical velocity responses under varied building stories and varied points of application of dynamic blast wave are presented in Table 4. Three different cases of application of velocities at different boundaries of the model were studied. Vertical velocity histories generated at different floor levels indicate that they are much higher than the input wave motion when the wave is applied at the bottom of the model, followed by input motions applied at the surface and crown. The main reason is that the base of the entire model is subjected to vertical velocity load history unlike the other two cases where the input wave is applied at specific locations of the model namely, the crown of the tunnel and the surface. Direct application of vertical vibration load at the base of the building or the surface led to magnification which was much more than amplification of displacements when the velocity wave was applied at the crown of the tunnel. Thus, farther the distance of vibration load from the surface the lesser will be the dynamic responses. Further, the percentage change in PV's as compared to the case of PV's generated in the presence of tunnel are evaluated (Table 5).

Table 4 Effect of varied point of application of input wave on PV response (mm/s)

Type of soil	Storey	Point of application	2 Hz	45 Hz	85 Hz
SS-1	2	Base	347.6	203.6	186.07
		Surface	243.8	176.2	173.05
		Crown	233.37	173.5	170
	4	Base	404.9	228.56	221.2
		Surface	276	228.17	217.12
		Crown	271	216.18	215
	8	Base	438.8	261.2	253.44
		Surface	318.3	248.52	242
		Crown	300.9	245.76	219.86
	2	Base	283.4	158.16	111.11
		Surface	192	126.31	109.00
SS-2	4	Crown	143.8	126.19	108.95
		Base	296.8	170	135.4
		Surface	200	159	133.9
	8	Crown	198.3	146.7	134.29
		Base	356.4	186.88	164
		Surface	242.6	154.32	162.8
		Crown	235.2	138.63	162.45

3.2.1 Variation of PV's in the Top Column (SS1) in Presence of Building and Tunnel

Vertical velocities generated under varied building stories and varied points of application of dynamic blast wave are presented in Figs. 8 and 9. The vertical displacement plots presented below are displacements generated on application of blast velocity time history. Velocity waves of frequency 2, 45 and 85 Hz are applied. Vertical velocity histories generated at columns at different floor levels indicate that they are much higher than the input wave motion when the wave is applied at the bottom of the model, followed by velocity history generated with input motions applied at the surface and crown. Thus a velocity of 188.6, 168.4 and 165.25 mm/s is noticed on application of input wave at base, surface and crown of the 2 storey building in SS-1. Reduction in PV's was evident in the presence of tunnel (Tables 6, 7).

Peak Velocity (PV) at two points along the height of the building (column level) is observed. PV's at the top of the columns is more than the PV's generated at the bottom of the column. Greater magnitude of PV's in the top floor columns was evident in structures founded in soft soil conditions, since the building experiences greater cantilever action and hence larger the displacements. Seismic vibrations, characterized by dominant horizontal accelerations, induce greater horizontal movements at the top stories of the building and therefore a similar effect is noted in buildings subjected to impact induced vibrations.

The amplification of PV's is evident in taller structures. Even though, the magnitude of PV's in SS-1 is greater than PV's generated in SS-2, the ratio of vibration in top column to that at mid-height of the building in SS-2, is slightly greater than the ratio of PV's generated in SS-1. Hence, ratio of vibration produced at the second floor as compared to the first floor of two storey building is 1.02-1.06, in SS-1 and 1.07-1.16 in SS-2 respectively. The ratio of amplification of PV at the mid-height of column of 4th floor as compared to the PV in 2nd floor column was 1.08-1.12 in SS-1 and 1.06-1.52 in SS-2. The column of the 8th floor experienced PV's 1.11-1.32 times higher than that experienced by the 4th floor of the structure in SS-1 and 1.16-1.5 times higher than that experienced by the 4th floor of the structure. Further, it is clear that greater amplification of PV is noticed when the input frequency is 2 Hz as compared to input frequencies of 45 and 85 Hz (Tables 8, 9). From Tables 8 and 9 it is

Table 5 Variation (%) of PV's in the absence and presence of tunnel

Type of soil	Storey	Point of application	2 Hz	45 Hz	85 Hz
SS-1	2	Base	−8.2	−42.1	−47.94
		Surface	−35.62	−50.5	−51.86
		Crown	−38.37	−54.26	−52.18
	4	Base	2.44	−37.72	−39.13
		Surface	−30.12	−37.82	−40.25
		Crown	−31.39	−41.095	−40.83
	8	Base	8.34	−34.86	−36.5
		Surface	−21.4	−38.02	−39.5
		Crown	−25.7	−38.7	−44.8
SS-2	2	Base	−17.85	−51.83	−65.23
		Surface	−44.34	−61.53	−65.8
		Crown	−58.30	−61.57	−66.2
	4	Base	−16.39	−49.55	−59.00
		Surface	−43.66	−52.80	−59.5
		Crown	−44.14	−56.46	−59.45
	8	Base	−11.73	−45.00	−51.47
		Surface	−39.90	−54.65	−51.83
		Crown	−41.75	−59.26	−51.93

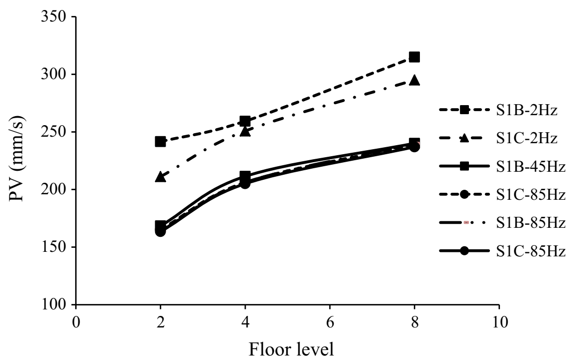


Fig. 8 Absolute maximum PV's at 'C' of 2, 4 and 8 storey building in SS-1

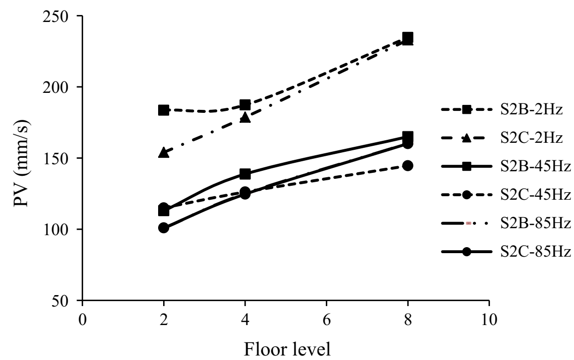


Fig. 9 Absolute maximum PV's at 'C' of 2, 4 and 8 storey building in SS-2

clear with an increase in storey magnification of PV's and displacements occurred due to greater cantilever action in the top floors.

3.3 Influence of Frequencies

3.3.1 Influence on Beams

As can be expected, for a given point of application of vertical load, input frequency of 2 Hz led to maximum changes in the beams and columns of the building. Even though input vibration load of 85 Hz had a PV of 110 mm/s, it did not lead to greater amplification of displacements in different parts of the structure. Therefore irrespective of PV's the frequency of input wave had a prominent effect in amplifying PV's and displacements. The rate of change of PV's with respect to base case was different for different ground conditions. In dense soils, the PV was smaller but the percentage decrease in PV's was significant with greater decrease in stiff soils than in soft soils.

Table 6 Percentage variation (%) of PV's in column in SS-1 with respect to base case

Storey	Point of application	2 Hz	45 Hz	85 Hz
2	Base	0.91	−37.32	−43.11
	Surface	−20.93	−44.03	−45.5
	Crown	−30.6	−45.08	−45.58
4	Base	7.17	−31.3	−31.6
	Surface	−20.46	−33.5	−32.8
	Crown	−23.06	−35.03	−33.18
8	Base	15.11	−29.5	−24.5
	Surface	−10.86	−31.7	−26.5
	Crown	−16.52	−32	−27.5

Table 7 Percentage variation of PV's in column in SS-2 with respect to base case

Storey	Point of application	2 Hz	45 Hz	85 Hz
2	Base	−16.69	−47.1	−61.7
	Surface	−33.95	−54.3	−62.43
	Crown	−44.60	−54.5	−62.35
4	Base	−15.00	−44.18	−46.8
	Surface	−33.10	−46.22	−46.8
	Crown	−36.18	−51.1	−46.96
8	Base	−2.70	−40.4	−45.3
	Surface	−28.6	−44.5	−45.5
	Crown	−29.2	−51.3	−45.53

Table 8 Ratio of PV in the top floor to the PV at mid-height of building in SS-1

Storey	Point of application	2 Hz	45 Hz	85 Hz
2	Base	1.06	1.05	1.045
	Surface	1.06	1.035	1.02
	Crown	1.04	1.04	1.03
4	Base	1.12	1.12	1.08
	Surface	1.093	1.085	1.08
	Crown	1.12	1.089	1.083
8	Base	1.32	1.12	1.11
	Surface	1.14	1.13	1.12
	Crown	1.14	1.11	1.15

Similarly at low frequency of 2 Hz the percentage decrease in PV was least compared to vertical velocity history developed due to application of PV of 45 Hz and 85 Hz indicating that the response of building without and with tunnel led to nearly equal responses (Tables 6, 7). For a given soil condition the percentage change in PV's was the least in 2 storey structure followed by 4 and 8 storey structure.

Table 9 Ratio of PV in the top floor to the PV at mid-height of building in SS-2

Storey	Point of application	2 Hz	45 Hz	85 Hz
2	Base	1.13	1.075	1.1
	Surface	1.09	1.075	1.07
	Crown	1.16	1.12	1.094
4	Base	1.15	1.13	1.08
	Surface	1.35	1.16	1.14
	Crown	1.18	1.17	1.15
8	Base	1.5	1.23	1.16
	Surface	1.53	1.21	1.16
	Crown	1.29	1.18	1.16

3.3.2 Influence on Final Displacements

Effect of the vertically propagating input wave, on the final displacements is interpreted. The negligible horizontal movements are ignored and only vertical movements are examined. In the absence of the building the maximum displacement at surface is 12.5, 11.5 and 11.41 mm respectively in SS-1 and which reduced to 8.39, 8.24 and 5.79 mm in SS-2 when subjected to PV's at the base, surface and crown of the model/tunnel respectively. In the presence of the building there is a reduction in displacement. Tunnels constructed in softer soils are not capable of resisting upward movement when subjected to vertical vibration loads and hence experience greater upliftment with maximum upward displacement noticed when the point of application of blast load was at the base followed by application at the surface and crown respectively.

Regardless of the type of soil, least downward displacement in the soil is noticed when the input frequency is 85 Hz, followed by greater magnitude of displacement when subjected to a frequency of 45 Hz. Upward displacements were observed when the input wave had a frequency of 2 Hz indicating that upliftment of soil takes place at frequencies very close to the natural frequencies of structure–strata. The displacements are of smaller magnitude compared to displacements produced in the absence of the building. This is due to the inertial effects of the building which tend to restrain the upward movement of the soil mass. Overall there is an upliftment of strata with a maximum displacement of magnitude 1.2, 2.6 and 5.06 mm in SS-1 and 1.45, 1.63 and 3.46 mm noticed in 2, 4 and 8 storey building in SS-2. Response of the soil to frequency 2, 45 and 85 Hz and in varied stories, in SS-1 and SS-2, are illustrated in Figs. 10 and 11. Displacement generated from the output file are presented in Fig. 12 (Points of maximum displacement are indicated by red/pink). The effect of upliftment and elongation of the building as a whole is more dominant in high storey structure and in softer soils. The reason being that soft strata do not provide the necessary rigidity against the elongation of building. Therefore minimum fixity at the base coupled with greater cantilever action led to greater displacements in tall structures and in soft soils.

3.3.3 Changes in the Lining Stress

3.3.3.1 Effect of Stress in the Lining (Considering Only Tunnel) Stress in the lining when subjected to blast wave at the bottom of the model was 0.067 MPa (at 45 Hz), 0.069 MPa (at 2 Hz) and 0.056 MPa (at 85 Hz) in SS-1 and 0.0797, 0.325 and 0.02 MPa in SS-2. Similarly when the blast wave is applied at the surface of the model, the stress in the lining is 0.072, 0.153, 0.06 MPa in SS-1 and 0.059 MPa (45 Hz), 0.325 MPa (2 Hz) and0.031 MPa (85 Hz), for SS-2. Maximum compressive stress concentration is observed in the lining of the tunnel when the blast wave is applied at the crown of the tunnel, due to direct impact of velocity wave on the tunnel lining. Stress in the lining is 0.216 MPa (45 Hz), 0.802 MPa (2 Hz) and 0.096 MPa (85 Hz) in SS-1 and 0.196 MPa (45 Hz), 0.99 MPa (2 Hz) and 0.0468 MPa (85 Hz) on interaction with SS-2 when subjected to a blast wave at the crown of the tunnel. As pointed out in Sect. 3.2, the upliftment of ground movement of the soil mass at the surface of tunnel, in soft soil conditions, causes a reduction in the compressive strength of the tunnel liner. Hence, an intensity of smaller magnitude (0.802 MPa) is noted in soft soil condition and greater magnitude compressive stress is observed in stiff soil condition (0.99 MPa).

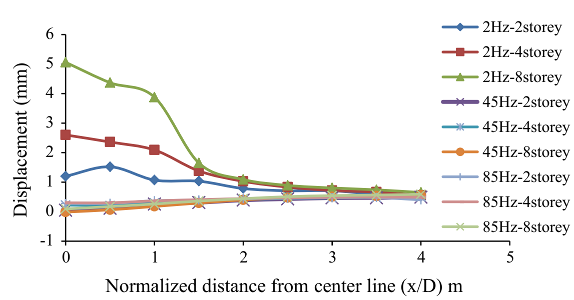


Fig. 10 Displacement on interaction with SS-1

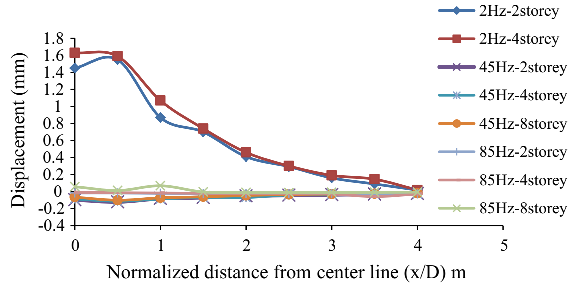


Fig. 11 Displacement on interaction with SS-2

3.3.3.2 Effect of Stress in the Lining (Considering Building and Tunnel) Presence of the building with its inertia has a different effect on the blast induced responses of tunnel. Stress in the lining is evaluated for different combinations of input wave frequencies and two different soil-structure types. Irrespective of the type of soil, maximum concentration of stress was noted when the PV was applied on the inner lining i.e. at the crown due to direct effect of application of blast wave on the lining. Regarding response of the lining to various input frequencies, response of the lining to input frequency of 2 Hz was maximum, followed by input wave frequency of 45 Hz as the input wave motion resonates with the natural frequency of the structure. These observations are similar to response, of only tunnel to blast vibrations. Irrespective of PV's, the frequency of the input wave of 2 Hz created magnification of velocities and concentration of stress. It is evident that least stress was noticed in the tunnel lining subjected to an input wave frequency of 85 Hz. A stress of magnitude 0.078, 0.195 and 0.87 MPa was noticed when 2, 4 and 8 storey structure interacted with SS-1. Maximum stress concentration of magnitude 0.45 MPa, 0.57 MPa and 2.1 MPa was noticed in the 2, 4 and 8 storied structure with SS-2.

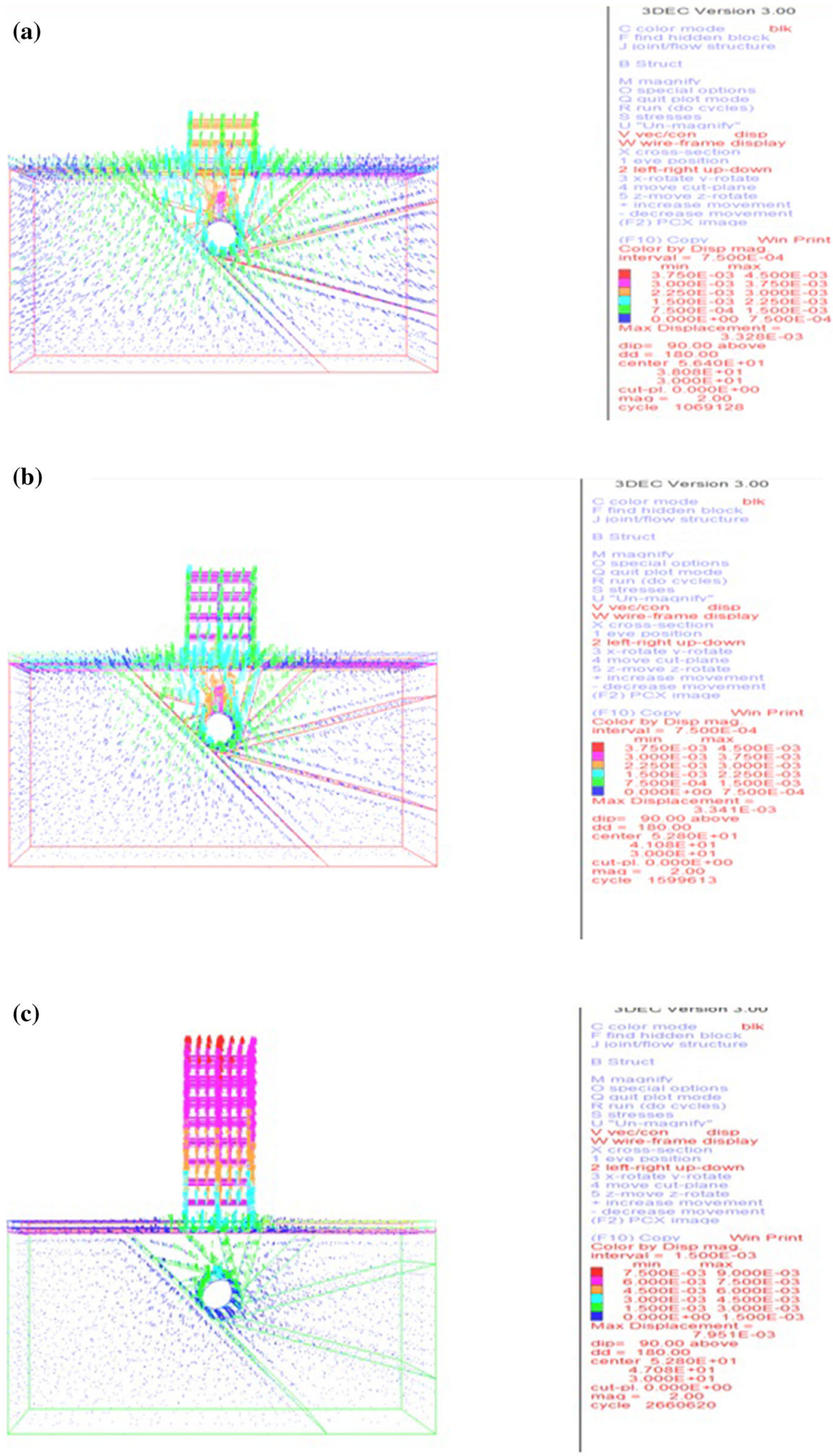


Fig. 12 Typical vertical displacements generated in 2, 4 and 8 storey structure (applied at crown of tunnel)

Table 10 Percentage increase in stress in the tunnel lining compared to no building case (applied at crown) 2 Hz

Type of soil	Stress without building (MPa)	2 Storey (%)	4 Storey (%)	8 Storey (%)
SS-1	0.802	90.2	79	8.47
SS-2	0.99	95.45	94.24	77.87

Change in stress concentration in the absence and presence of building are illustrated in Table 10. Thus Table 10 clearly indicates the presence of the building increased the major principal stress when subjected to dynamic vertical velocity load. With an increase in storey a decreasing trend in percentage increase in major principal stress was observed due to greater magnitude of elongation in taller structures. Although it is evident from Figs. 10 and 11 that there is an upward movement of building with the soil mass, the inertial effects due to the presence of the building tends to reduce the elongation of tunnel liner and therefore the compressive stress does not get dissipated. However, with an increase in storey the elongation of the building which dominates inertia, tends to produce greater displacements accompanied with lesser increase in stress in the liner of the tunnel.

3.4 Time for Processing

With an i5 processor at 3.2 GHz speed, 16 GB RAM and 64 bit-operating system, processing took several days. The dimension along the Z-direction limited to 40 m, in the simplest case of a tunnel with 2-storey building, took 10 h for preprocessing and another day for developing the velocity time histories or post-processing. A combination of tunnel with 4-storey building took 3 days for preprocessing and combination of tunnel with 8-storey building took 5 days for generating results and the output. A total of 130 models were developed for the analysis.

4 Limitations of this Study

Although the process of creating an opening and subsequent application of blast loads has been modeled, few of the limitations of the study are listed below

1. 3DEC 3.0 is mainly suitable for modeling stiff clays, rocks and granular soils of higher density with large geometrical or geological discontinuities.
2. Further the PV values generated are for a typical case of a tunnel with diameter 6.1 m and for a constant depth of overburden of 8 m. Modifications in the tunnel diameter and depth may lead to varied results from the above cases.
3. Since elastic analysis of the liner and the building is considered, the results are not representative of the actual field observations where lesser magnitude of PV's and displacements may be obtained.
4. Also, version 3DEC3.0 does not incorporate superior material models as compared to 3DEC5.0. This limits the extent to which modeling can be carried out to predict the actual behavior of surrounding soil in softer strata.

5 Conclusions

The behavior of building and liner installed in varied soil types and subjected to vertical vibration loads was investigated numerically. Parametric studies of the most influencing factors, such as varied frequency, varied soil types, varied point of application of dynamic wave and varied building stories, on displacements were assessed. Further the stresses developed in the lining of the tunnel were evaluated. Following conclusions are drawn from this study.

1. The investigations indicated that building structures in softer soils subjected to vertical vibrations, led to peak responses of PV values compared to structures in stiffer soils.
2. Presence of the tunnel reduced the PV's at the top of the building. This indicates that the void produced due to the presence of the tunnel leads to damping and restricts the upward propagation of the wave. Further experimental and numerical investigations are needed to find the response under different tunnel diameters and varied building base widths.

3. Regarding columns, PV's and displacement at the top columns increased with an increase in storey. With an increase in storey the ratio of PV's at the top column to that of PV's at the mid-height of building increased. The reason for increase in PV's was due to greater cantilever action at the top floors compared to the bottom floors. These results are consistent with the results carried by Singh and Roy (2010).
4. Although it is evident that taller structures, in softer soils amplify the PV's compared to structures constructed in stiff soils, the ratio of vibration in top column to that at mid-height of the building in SS-2, is slightly greater than the ratio of PV's generated in SS-1.
5. Vertical velocity histories generated at different floor levels indicated that they were much higher than the input wave motion when the wave was applied at the bottom of the model, followed by input motions applied at the surface and crown. The main reason being that in the first case, base of the entire model was subjected to vertical velocity history vibration.
6. Regarding frequencies of the input wave, low frequencies of magnitude 2 Hz led to greatest amplification of PV values compared to other frequencies of 85 and 45 Hz. Further, maximum displacements at the surface, in the presence or absence of the building occurred at 2 Hz. Similarly the stress in the lining was maximum at low frequencies of 2 Hz. The main reason behind this effect is the input frequency of 2 Hz was closer to the natural frequency of the building-tunnel-soil. Further, irrespective of the input peak velocity, the frequency of the vibration load had the most significant effect in affecting the peak velocity and displacements in different parts of the building and tunnel.
7. Presence of the structure led to increase in stress values in tunnel lining which is mainly attributed to elongation/expansion of the tunnel lining. With an increase in storey, the percentage increase of stress concentration in the tunnel lining reduced.

Although numerical investigations are needed to study other aspects of the elasto-plastic behaviour of liner and building, the parametric studies involving varied soil types, varied input frequencies and point of application will provide a reliable guideline in making in advance assessment of peak velocities and displacements when subjected to blast wave vibrations.

Appendix

Code for developing frequencies

```
load Filename.txt
velocity = Filename (:,2);
time = Filename(:,1);
plot(abs(fft(Filename(:,2))))
xlim([0,120])
```

References

Dowding CH, Rozen A (1978) Damage to rock tunnel from earthquake shaking. J Geotech Eng Div ASCE 104(GT2):175–191

Dowding CH, Beck WK, Atmatzidis DK (1980) Blast vibration implications of cyclic shear behavior of model plaster panels. ASTM Geotech Test J 3(2):81–90

Edwards AT, Northwood TD (1960) Experimental studies of the effect of blasting on structures. Eng Lond 210(5662):539–546

Hashash YA, Hook JJ, Schmidt B, Yao JIC (2001) Seismic design and analysis of underground structures. Tunn Undergr Space Technol 16:247–293

Jiang N, Zhou C (2012) Blasting vibration safety criterion for a tunnel liner structure. Tunn Undergr Space Technol 32:52–57

John St CM, Zahrah TF (1987) A seismic design of underground structures. Tunn Undergr Space Technol 2:165–197

Kulhawy FH (1975) Stress deformation properties of rock and rock discontinuities. Eng Geol 9:327–350

Liu H (2009) Dynamic analysis of subway structures under blast loading. Geotech Geol Eng 27:699–711

Liu H, Song E (2005) Seismic response of large underground structures in liquefiable soils subjected to horizontal and vertical earthquake excitations. Comput Geotech 32:223–244

Lysmer J, Kuhlemeyer R (1969) Finite dynamic model for infinite media. J Eng Mech Div ASCE 95(4):859–877

Ma G, Zhou H, Karen C (2011) In-structure shock assessment of underground structures with consideration of rigid body motion. J Eng Mech ASCE 137:797–806

Nicholls HR, Johnson CF, Duvall WI (1971) Blasting vibrations and their effect on structures. Bureau of Mines, Bulletin 656, Washington

Odello RJ (1976) Ground shock effects from accidental explosions. Technical memorandum No.TM M-51-76-22, Department of the U.S. Army Picatinny Arsenal

Singh PK, Roy MP (2010) Damage to surface structures due to blast vibration. Int J Rock Mech Min Sci 47:949–961

Tian L, Li Z (2008) Dynamic response analysis of a building structure subjected to ground shock from a tunnel explosion. Int J Impact Eng 35:1164–1178

Wang WL, Wang TT, Su JJ, Lin CH, Seng CR, Huang TH (2001) Assessment of damage in mountain tunnels due to the Taiwan Chi-Chi earthquake. Tunn Undergr Space Technol 16:133–150

Wei H, Jian-Yun Chen, Jing Guo (2011) Dynamic analysis of subway station subjected to internal blast loading. J Cent South Univ Technol 18:917–924