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Pile response subjected to rock blasting induced ground vibration near soil-rock interface

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ABSTRACT

Blasting has been widely used in mining and construction industries for rock breaking. Ground vibration induced by blasting is an inevitable side effect that may cause damage to nearby structures, if not properly controlled. In this study, response and possible damage of rock-socketed pile near soil-rock interface subjected to ground shock excitations are investigated and quantified with coupled SPH-FEM method. Results indicate that the base of the pile is relatively vulnerable and that the soil properties significantly influence on response of pile subjected to a specific blast load. Furthermore, based on the numerical results, ground vibration attenuation equation is proposed.

Keywords: Rock blasting; Numerical simulation; Pile foundation; Wave propagation; Smooth particle hydrodynamics

1. Introduction

Blasting is the widely accepted method to rock breaking in mining industry, and in civil engineering constructions such as constructing underground structures and tunnels. In addition to that, rock blasting method is one of the most practical technique to use for land development projects in land-scarce countries like Singapore to create additional spaces due to its cost effectiveness, higher efficiency and ability to break hard rock. However, ground vibrations from blasting are undesirable and it can cause damage to nearby structures.

Many studies on blast-resistant designs have been carried out by the military services, and the relevant documents are restricted only for official use. However, a considerable amount of

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research on the dynamic response and damage of nearby structures to blast ground motion can be found in the literature [1-5]. Structural responses and damage to blast induced vibration have been extensively studied in the past several decades and those studies concentrated on establishing allowable ground vibration levels in terms of peak particle velocity (PPV) together with the frequency of the ground vibration to limit the structural damage. However, safe vibration limits given by different researchers differ because of those limits were usually obtained based on field observations of low-rise residential buildings. Recently, with the development of technology, numerical simulations have become popular in investigating the dynamic response and damage of structures subjected to blast-induced ground vibration. Chen and Zhang [6] proposed an orthotropic dynamic damage constitutive model to simulate the dynamic responses of typical masonry structures under blasting vibration. The authors have found that the blasting vibration duration influences structural responses and taller structures experience less damage than do smaller structures under the same blasting vibration. Dhakal and Pan [7] performed nonlinear finite element (FE) analyses on a two-storey reinforced concrete (RC) frame to investigate the effects of blasting on structural response. Wu et al. [8] performed a three-dimensional (3D) dynamic response and damage analysis of masonry and masonry infilled RC frame structures to blast ground motions. The authors have concluded that the damage to the structure under blasting vibration is governed by the force-stress rather than the ductility or the interstorey drift. It is also found that the out-of-plane damage of the masonry walls is more severe than the in-plane damage.

In practice, the damage to nearby structures to generated ground vibrations has been controlled by various rules and regulations available. The existing vibration limits are not always applicable, as they depend on the geological conditions of the site and dynamic characteristics of the structure. The scope of the present study is limited to assess the stability and vulnerability of the pile foundation system against ground vibration caused by typical blast loads from nearby rock blasting.

Pile foundations are commonly used as foundations of high-rise buildings and bridges to transfer the heavy loads from the superstructure above through weak compressible soil strata into deeper, competent soil layers which have adequate capacity to carry these loads. Vertical piles are normally designed to carry mainly vertical loads and very little lateral loads. Short duration, high frequency, high amplitude loads such as rock blasting may also have an impact

on the pile foundation system. They can induce lateral and bending stresses in the piles and cause significant damage, resulting in differential settlement and tilting of the superstructure, leading to weakening of the structure. Thus, the lateral response of piled foundation is important in the designing of structures that may be subjected to lateral loads. The lateral response of a pile, however, is a complicated soil-structure interaction problem; because pile deflection depends on the soil resistance and in turn the soil reaction depends on the pile deflection [9].

The performance of pile foundations subjected to dynamic lateral loads is a critical research area, as the foundation plays an important role in the overall structure response. A foundation system can fail even if the piles are not failed by the short-term dynamic load like blast loads, simply due to the combination of secondary action effects such as reduction of effective capacity of the pile due to blast damage, amplification of moments induced by displacements, and amplification of buckling effects [10]. In contrast to an axially loaded pile, the laterally loaded pile is a three-dimensional problem, and soil-pile interaction (SPI) is extremely complex when non-linear conditions and dynamic conditions exist simultaneously and also it plays a significant role in the pile response to ground vibrations [11].

Since a pile foundation is usually buried beneath the ground surface, it can also be considered as an underground structure in some aspects. Thus, it is also relevant to briefly review related experimental and numerical studies on blast response of underground structures. The influences of blasting vibration on underground structures have been studied by researchers using field experiments and numerical simulations. Kutter et al. [12] carried out a series of 1-g to 97-g centrifuge tests to investigate the blast response of shallow tunnels in dry sand. The authors found that the distance from the tunnel to the crater is a more significant factor than the distance from the tunnel to the charge in causing damage to flexible tunnels. Liu and Nezili [13] used a centrifuge test to study the response of transit tunnels in saturated soils under internal blast loading. The test was carried out with a centrifugal acceleration of 50-g on a thin wall Aluminium tube. The study showed that the model tunnel was significantly damaged by the internal blasting and that the large lining deformation that occurred was mainly concentrated in the area close to the explosive. The authors have also concluded that surrounding soil liquefaction is possible for a tunnel in saturated soil subjected to internal blasting.

Finite element method (FEM) is a useful tool in the analysis of soil-structure interaction problems. In certain situations, FEM provides a relatively simpler tool for the analysis. Yang et al. [14] discussed blast resistant analysis for Shanghai metro tunnel using explicit dynamic nonlinear FE software LS-DYNA [15]. The arbitrary Lagrangian-Eulerian (ALE) method was employed in their model and the overall analysis evaluated the safety of the tunnel lining based on the failure criterion. Since there have not been any established common standards governing the design of such a structure, a series of parametric studies have been carried out in order to evaluate the significance of several parameters such as shear modulus and bulk modulus of soil, on the lining thrust. Nagy et al. [16] investigated the response of a buried concrete structure to various factors affecting structural performance by carrying out a parametric study using a FE model. Depths of the structure and charge were considered as parameters. It was shown that buried explosions result in significant effects on the buried structure than surface explosions under the same conditions. De [17] used fully coupled numerical model to study the effects of a surface explosion on an underground tunnel. The model was validated using the centrifuge test results. Koneshwaran et al. [18] investigated the performance of buried tunnels subjected to surface blasts. The authors found that the response of the tunnel buried in saturated sand is more severe than that in dry soil for a given blast event. Moreover, some researchers have tried to investigate the response of tunnels subjected to internal blasting with different numerical methods. Liu [19] carried out extensive numerical simulation to investigate the soil-structure interaction and failure of cast-iron tunnels in saturated soil subjected to internal blast loading and found that the ground-tunnel interaction was one of the governing factors determining the damage of tunnel lining. It also found that lining damage was mainly caused by the tensile hoop stress because of large inertia and dynamic forces in the radial direction of the tunnel. Khan et al. [20] performed FE analysis of underground tunnels with cast-iron lining embedded both in soil and rock subjected to an internal explosion at the center of the tunnel. It was found that the thickness of tunnel lining, peak blast pressure in the tunnel, and the elastic moduli of soil and rock affect the blast response of tunnel more significantly.

Unfortunately, only a few studies can be found in the literature and discuss the damage and failures of piles under blasting vibration. Thus, it is necessary to study SPI under blasting load, because of the wide application of blasting and to highlight their impact on the surrounding structures. Kamijo et al. [21] conducted vibration tests at a large-scale mining

site to investigate liquefaction phenomena and dynamic responses of pile foundations. The test structure included RC top slab and pile cap, 4 steel tubular piles and 4 steel H-section columns. Ground vibrations from large-scale blasting operations were used as excitation forces for the vibration tests. Nonlinear responses of the soil-pile system at different levels were obtained for various levels of liquefaction in the test pit. It is found that bending moments were maximum at the pile heads, regardless of input motion levels. However, the moment distribution shapes differed with the degree of the liquefaction in the test pit. Ashford et al. [22] conducted full-scale experiments using controlled blasting to assess the dynamic responses of piles during lateral spreading. The test piles included a single pile, a 4-pile group and a 9-pile group. The single pile had the free end condition while the pile groups had RC pile caps, and steel pipes were used for all the piles. The test results indicated that the pile head displacement and moment in the single pile were significantly higher than those observed in the pile groups. It is also found that the degree of fixity at pile tips had a great influence on the moments of individual piles in the group. Large bending moments are developed in the pile when the larger degree of fixity into the dense soil layer.

With the rapid development of explosion theory and computer technology, numerical simulation has become a promising approach to analyzing the SPI problems. Hao et al. [23] presented a numerical method to calculate the elastic and inelastic single pile responses to blast loads. The pile-soil system was modelled as beam-column elements supported by both vertical soil springs of Winkler foundation. However, this method cannot incorporate the radial and three-dimensional components of interaction. The shear stress which is acting along the side of the pile is ignored by this method. Since a 3D FE analysis requires a considerable amount of computational cost for generating input and interpretation results, it has not been used frequently until recent for the SPI analyses. Huang et al. [24] studied the dynamic response of pile-soil-structure interaction (PSSI) system under blasting load. Solid elements were used to simulate piles, soil and pile cap, while beam elements were used to simulate columns and beams of the superstructure. In this study, they applied a velocity-time history curve of blasting seismic wave on the tip of the pile. The authors have concluded that because of the maximum shear stress at the top of the pile, the connection of piles and pile cap are easily damaged and pile-soil contact pressure increases at the pile ends. Jayasinghe et al. [10, 25] carried out FE analyses of RC piles subjected to underground explosions in soils. Jayasinghe et al. [25] developed a fully coupled method to treat the blast response of a pile

foundation in saturated soil and the effects of end restraint of pile head and the number and spacing of piles within a group were investigated later [10]. Chakraborty [26] performed parametric sensitivity studies on hollow steel piles subjected to buried blast loading in sandy soil.

There are many FE codes available that are capable of analyzing challenging engineering problems. This paper treats the response of pile foundations of nearby structures subjected to ground-borne vibrations from rock blasting using numerical simulations through the commercial software package LS-DYNA. FE modeling code LS-DYNA [15] offers different numerical techniques such as Lagrangian, Eulerian, Arbitrary Lagrangian-Eulerian (ALE) and Smooth Particle Hydrodynamics (SPH). SPH is a mesh-free Lagrangian method efficiently used for simulation of problems involving large deformation. The SPH method was developed by Gingold and Monaghan [27] and Lucy [28] to study astrophysics. Subsequently, due to the advantages of the SPH method, it has been used in many fields of research. Liu et al. [29] successfully used the SPH technique to simulate the explosion of high explosive. Hence, this study used coupled SPH-FEM method for the simulation. A brief description of the background on modelling is presented at the beginning of this paper. Then, the study on blast wave propagation in soil and the comparison of numerical results with the field test results are presented. Finally, the blast responses of piles to different blast scenarios are presented.

2. Simulation of blast loading

Blast loads are short duration dynamic loads. There are two methods to apply the blast load in numerical models. They can be classified as either uncoupled or coupled method. In the uncoupled method, first the blast pressure curve of the borehole are computed typically using empirical equations, and then are applied directly on the borehole wall. In the coupled method, the explosive charge is modelled in the FE model to form shock waves, hence the detonation of explosive, blast wave propagation in the medium and response of the structure to the produced blast shock waves are combined in a single model. An explosive charge can be simulated using Equation of states (EOS) in LS-DYNA. The Jone-Wilkin-Lee (JWL) EOS is widely used to simulate the relationship between pressure and specific volume in the explosion process since it is easiest to calibrate. This model is suitable to model high explosives like Trinitrotoluene (TNT). However, this model is not suitable to simulate

Ammonium Nitrate Fuel Oil (ANFO) like non-ideal explosives. This is because JWL EOS is not sufficient to capture the detonation of non-ideal explosives [30]. Campbell and Engeike [31] have concluded that velocity of detonation of ANFO like non-ideal explosives depends on the radius of the explosive charge. Thus, a careful attention is required in the simulation of the size effect of ANFO.

The detonation of 5 ANFO cylinders with radii ranging from 45 mm to 150 mm was simulated in this study, and the length to radius ratio was kept as 10 in all the cases. ANFO cylinders were modelled with SPH particles. In SPH method, the entire domain is represented by a finite number of particles that carry individual mass and occupy individual space. Smoothing length is an important parameter in SPH method because it determines the influence domain of each particle. Smoothing length coefficient was set to be 1.1 in all the cases in this study.

The ignition and growth (I & G) reactive flow model was selected to simulate the detonation of ANFO. I & G reactive flow model requires two EOS, one for the unreacted explosive and one for its reaction products, and also a reaction rate law for the conversion of the explosive to products [15].

Two JWL EOS define the pressure in the unreacted explosive, P_e , and the pressure in the reaction products, P_p , as [15]:

$$P_e = r_1 e^{-r_5 V_e} + r_2 e^{-r_6 V_e} + r_3 \frac{T_e}{V_e} \quad (1)$$

$$P_p = A e^{-xp_1 V_p} + B e^{-xp_2 V_p} + g \frac{T_p}{V_p} \quad (2)$$

where V and T are the relative volume and temperature, respectively, with the index e for the unreacted explosive and index p for the reaction products. $r_1, r_2, r_3, r_5, r_6, A, B, xp_1, xp_2$ and g are material constants.

The chemical reaction rate for the conversion of unreacted explosive to reaction products is defined as [15]:

$$\frac{dF}{dt} = I(1-F)^b (V_e^{-1} - 1 - a)^x + G_1(1-F)^c F^d P^y + G_2(1-F)^e F^f P^z \quad (3)$$

where F is the fraction reacted (ratio of the detonation products mass to total explosive mass), P is the pressure, t is the time, I , G_1 , G_2 , a , b , c , d , e , f , x , y and z are constants.

The density of the ANFO was set to be 800 kg/m^3 and Table 1 lists the I & G model parameters used to simulate the ANFO cylinders in this study.

Table 1. I & G model parameters for ANFO

Detonation of ANFO at different time slots is shown in Fig. 1. As can be seen in the figure, the explosive product gradually expands as the detonation front propagates from bottom to top.

Fig. 1. Detonation of ANFO at (a) $t = 15\mu\text{s}$ (b) $t = 25\mu\text{s}$ (c) $t = 50\mu\text{s}$ (d) $t = 100\mu\text{s}$

The characteristics of non-ideal explosives have been studied many researchers through experiments and analytical studies [31-34]. The detonation velocity measurements were mostly made with the pin technique as described by Campbell and Engeike [31] and Bdzil et al. [33]. They have conducted rate stick tests on various non-ideal explosives at ambient temperature to measure the VoD and detonation front curvature as a function of radius of the explosive charge. The experimental results available in the literature [34], for the detonation velocity of unconfined prilled ANFO, were used to calibrate the parameters in the I & G reactive flow model to the detonation of ANFO. The comparison of the present simulation results and the experimental results is illustrated in Fig. 2. In the figure, the VoD in the axial direction at a point just below the top of the charge is plotted against the charge radius. The blue diamonds and red squares represent the experimental results and numerical results, respectively, can be found in the literature [34], and green triangles represent the present numerical results. As can be seen in the figure, the present numerical results are in good agreement with the experimental results. It is also clear that VoD is dependent on the charge radius.

Fig. 2. Velocity of detonation (VoD) vs. radius of explosive charge

3. Blast wave propagation in rock/soil

During rock blasting, only a portion of the total energy of the explosive is consumed in the rock breakage, while the rest is transmitted through the ground as vibrations and through the air as air blast. Various types of waves that originated from a blast source propagate through the rock/soil media and is transmitted to the nearby structures through their foundations. The study of blast wave propagation in rock/soil media can provide useful insights to engineers on the resilient characteristics of a particular site, and the propagation of blast waves through the ground have been discussed in the past by many researchers [35-38].

The study of blast wave propagation in rock/soil media and validation of the developed FE model are described in this section. The blast wave propagation was simulated in a 3D FE model using LS-DYNA. By considering the symmetry of the model, only a half symmetry-geometrical FE model was developed to minimise the computational time. Fig. 3 shows the developed 3D FE model to study the blast wave propagation in rock/soil media. The model size was set to be 32 m long, 10 m wide and 10 m high. The explosive charge, stemming materials and the part of the rock experiencing large deformations were modelled with SPH particles while the rest of the model (rock and soil) was based on Lagrange FEM element. The symmetry face is fixed against translational displacements normal to the symmetry plane. Non-reflecting boundaries are applied to the other surfaces, except the top surface which has the free boundary condition.

Fig. 3. A half symmetrical model

Nonlinear effects near the explosive source were considered to accurately model the physical behaviour. There are several material models for rock implemented in LS-DYNA. For simplicity, the constitutive characteristics of rock are assumed to be homogeneous isotropic elastic-plastic and the elastic-plastic material model *MAT_PLASTIC_KINEMATIC in LS-DYNA was used to simulate the constitutive characteristics of rock under blast loading. The physical and mechanical parameters of the Bukit Timah granite are described in Wei et al [39] and were used in the simulations. The corresponding parameters for the rock are described in Table 2.

Table 2. Material parameters for rock

The constitutive stress-strain behaviour of soil is also an important aspect in wave propagation in soil and SPI analysis. Generally, there are two types of soil models that are used in finite element analyses of soils such as elastic models and elastic-plastic models. Elastic-plastic constitutive models such as Mohr-Coulomb and Drucker-Prager can provide a reasonable representation of a typical wave propagation problem. Thus, the *MAT_MOHR_COULOMB material model in LS-DYNA library was used to simulate the soil and the basic inputs required in this material model are presented in Table 3.

Table 3. Material parameters for soil

Stemming was modelled as gravel and the *MAT_SOIL_AND_FOAM material model in LS-DYNA was used. The density, elastic modulus and Poisson's ratio for the stemming material were considered as 1750 kg/m³, 5 kPa and 0.2, respectively.

ANFO explosive charge was modelled using the I & G reactive flow model as described in the above section. Blast hole diameter and length were taken as 64 mm and 6 m, respectively. 50 mm diameter of explosive charge with 4 m length was modelled and the height of the stemming column was considered as 2m. Charge weight was changed from 4 kg to 50 kg of ANFO and the peak particle velocity (PPV) values were obtained at the monitoring points on the soil surface level at 10 m, 20 m and 30 m distance from the blast hole. Table 4 shows the results obtained from the above analyses. The relationship between the PPV and scaled distance (SD) is shown in Fig. 4. Scaled distance is a parameter that relates similar blast effects from various charge weights of the same explosive detonated at various distances to the structure or point of concern. It is calculated by dividing the distance between the charge and the monitoring point by a fractional power of the weight of the explosive charge. The square root scaled distance is the most commonly used term in general blasting situations where the charge is distributed in a long cylinder. All the obtained numerical results were used to derive the ground vibration attenuation equation relationship between the PPV and SD, determined from Fig. 4, are shown in the following equation:

$$\text{PPV (mm/s)} = 708.06 (\text{SD})^{-1.606} \quad (4)$$

Table 4. PPV values – numerical results

Fig. 4. Relationship between PPV and SD

The numerical model was calibrated by comparing the field tests results obtained from a field rock blasting carried out at a site in the northern part of Singapore in late January 2016. For the field rock blasting, the blast holes in 6m depth with 21 kg and 23 kg ANFO charges per delay were used. During the test, velocity sensors were employed on the soil surface to record the ground vibration and PPV values were recorded at the blasting ranges of 30 m to 80 m for each blasting. Fig. 5 illustrates the test location (a hilly area in western Singapore) and the monitoring points at the ground surface while the on-site measured PPV values are listed in Table 5.

Fig. 5. Illustration of the location of blasting and monitoring points

Table 5. Ground vibration monitoring data

The field monitoring PPV values and the numerically simulated ground surface PPV attenuation are shown in Fig. 6. Even though the numerical results are slightly higher than the measured results from the field rock blastings carried out at the site, it can be seen that the simulated PPV values are in good agreement with the field monitoring results. The differences in results are possibly caused by the differences in actual ground profile at the field and simplified ground profile used in the numerical model. It is too complicated to account for all the factors influencing the vibration induced by rock blasting, such as the detailed blasting design and the geological condition between the blasting and the location of concern. A sloping down soil-rock interface was considered in the present simulation, and equivalent properties were used to simulate the rock and soil. Moreover, the rock was modelled without considering the presence of joints and fractures due to insufficient geological site investigation data. Even though they greatly affect on the accuracy of the numerical result, the present simulation results give a sufficiently good match against the test results and can therefore be used in the further simulations.

Fig.6. Measured PPV from the field test and numerical simulation

4. Numerical simulation of blast response of pile

This paper investigates the blast response of RC pile using dynamic computer simulation technique. In this section, results for the blast response of the pile foundation to different blast scenarios are presented. The influence of standoff distance, charge weight, soil properties, pile ends conditions and slope of rock-soil interface on the dynamic response and damage of piles are evaluated and discussed.

4.1 Effect of standoff distance and charge weight on the blast response of pile

The response of a single pile subjected to ground vibration from rock blasting is simulated in a 3D FE model using LS-DYNA. The same modelling techniques and material models and parameters described above were also adopted in this model. However, a single pile with a pile cap was modelled here as shown in Fig. 7.

Fig. 7. 3D FE model

3D FE model was developed for 8 m long reinforced concrete (RC) pile with 600 mm diameter circular cross section. The pile was reinforced with 8 numbers of 16 mm diameter bars, providing a vertical reinforcement ratio of 0.6% and spiral reinforcement of 10 mm bars at spacing of 200 mm. Eight-node solid elements were used for the 3D explicit analysis, except for the reinforcing cage. Both the vertical reinforcements and the ties were modelled with 25mm long beam elements having 2x2 Gauss integration. The vertical reinforcements were defined as Hughes-Liu beam elements with cross-integration and ties were defined as truss elements. The ANFO explosive, stemming material and part of rock close to the explosive were modelled with SPH particles while Lagrangian meshes were used to model the pile, soil and the rock region away from the explosive charge. The simulations used automatic node to surface contact conditions for the coupling interaction between the SPH particles and Lagrange FEM elements.

LS-DYNA contains several material models that can be used to represent concrete. Since the selection of a model capable of characterising concrete behaviour under high strain rate blast loads is essential, the *MAT_CONCRETE_DAMAGE_REL3 material model was used to

simulate the concrete behaviour. It is a plasticity-based model with three shear failure surfaces and includes damage and strain rate effects [40]. The literature has shown that material concrete_damage_rel3 model can successfully incorporate non-linear concrete properties [38, 41]. The advantage of this model is that unconfined compressive strength and density of concrete are the only two parameters required in the calibration process. However, if the simulated response significantly differs from the observed behaviour of the concrete, then the material parameters can be refined to improve the accuracy of the material model [41]. In this study, concrete density, the compressive strength of concrete, Poisson's ratio and the maximum aggregate size were considered as 2300 kg/m³, 35 MPa, 0.19 and 19 mm, respectively.

In order to account for the increase in strength under high strain rates, a coefficient called the dynamic increased factor (DIF) described by Malvar and Crawford [42] is employed in this analysis. The DIF for the concrete compressive strength is given as:

$$\text{DIF} = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1.026\alpha} & \text{for } \dot{\epsilon} \leq 30 \text{ s}^{-1} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\frac{1}{3}} & \text{for } \dot{\epsilon} > 30 \text{ s}^{-1} \end{cases} \quad (5)$$

where $\dot{\epsilon}$ is the strain rate in the range of 30×10^{-6} to 300 s^{-1} , $\dot{\epsilon}_s$ is the static strain rate and it is equal to $30 \times 10^{-6} \text{ s}^{-1}$, $\log \gamma = 6.156\alpha - 2$, $\alpha = 1/(5 + 9f_c/f_{co})$; $f_{co} = 10 \text{ MPa}$, f_c is the static compressive strength of the concrete. The DIF for concrete in tension is given by:

$$\text{DIF} = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\delta} & \text{for } \dot{\epsilon} \leq 10 \text{ s}^{-1} \\ \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\frac{1}{3}} & \text{for } \dot{\epsilon} > 10 \text{ s}^{-1} \end{cases} \quad (6)$$

where $\dot{\epsilon}$ is the strain rate in the range of 1×10^{-6} to 160 s^{-1} , $\dot{\epsilon}_s$ is the static strain rate and it is equal to $30 \times 10^{-6} \text{ s}^{-1}$, $\log \beta = 6\delta - 2$; $\delta = 1/(1 + 8f_c/f_{co})$; $f_{co} = 10 \text{ MPa}$, f_c is the static compressive strength of the concrete. Thus, different rate enhancements were included in tension and compression in the concrete material model used in this study.

Both vertical and transverse reinforcements were modelled using the elastic-plastic material model *MAT_PLASTIC_KINEMATIC in LS-DYNA. Kinematic hardening with strain rate effects was

implemented for the reinforcement. Strain rate is accounted for using the Cowper-Symonds model given by [15]

$$\frac{\sigma'_d}{\sigma_s} = 1 + \left(\frac{\dot{\varepsilon}}{C} \right)^{1/P} \quad (7)$$

where σ'_d is the dynamic flow stress at a uni-axial plastic strain rate $\dot{\varepsilon}$, and σ_s is the associated static flow stress. C and P represent the material constants. Material model parameters for the steel are described in Table 6.

Table 6. Material parameters for steel

A proper coupling mechanism needs to be used to achieve good interaction between concrete and reinforcement elements. There are various ways to achieve coupling in LS-DYNA such as merging the reinforcing beam elements with solid concrete elements in the form of shared nodes, which most researchers have used in their studies. In this study, the `Constrained_Lagrange_in_Solid` was used to couple concrete solid elements with the reinforcing cage. This method when used with the fluid-structure coupling mechanism of `CTYPE = 2`, couples concrete with reinforcement in an efficient manner and it removes the problem of having to align the beam nodes to the solid element nodes.

The interaction between the pile and surrounding soil are usually modelled either as a perfectly bonded interface or as a frictional interface where soil-pile slipping and gapping may occur. In reality, the soil-pile interface should be modelled to incorporate slipping and gapping. However, due to the high computational time and convergence problems, a perfect bonding was considered at the soil-pile interface. Moreover, the axial load acting on the pile was not considered in this study.

The distances between the piles and the blast holes were considered as 7.5 m and 11 m, and blast responses of piles were studied when 6 different single-slot, fully coupled charges were detonated, for the purpose of comparison as shown in Table 7.

Table 7. Analysis cases

The horizontal deformations of the pile were obtained at 7 monitoring points along the pile at different heights from the pile tip (bottom): 0 m, 1 m, 2 m, 4 m, 6 m, 7 m and 8 m, in each case. Fig. 8(a) shows the horizontal deformation of the pile along the pile length. It is clearly seen that the pile is subjected to a maximum deformation of 0.7 mm at a height of 6 m above the pile bottom. The damage to the reinforced concrete is also observed from the present numerical simulation. Fig. 8(b) depicts the concrete effective plastic strain variation observed on the pile. Effective plastic strain is the damage parameter in the concrete_damage_rel3 material model which ranges from 0 to 2. The colours in the Figure indicate the fringe level which represents the level of damage in the concrete. The blue colour represents the fringe level 0 which indicates the elastic state of the concrete while the red colour represents the fringe level 2 which indicates the complete yielding of the concrete. The other colours which are associated with fringe levels between 0 and 2 represent the different damage levels of the concrete [38]. Based on the effective plastic strain data, the pile remained elastic under the ground vibration caused by 11.5kg charge weight detonated at 7.5 m standoff distance.

Fig. 8. Results in case 1 (a) Pile horizontal deformation (b) Effective plastic strain diagram of concrete

The obtained results in the cases 2 to 6 are illustrated in Fig. 9 and 10. The residual horizontal displacements along the height of pile in each case are plotted in Fig. 9. As seen in the figure, maximum pile deformations were 1.6 mm, 13.4 mm, 30.3 mm, 1.7 mm and 20 mm at the pile head level in case 2, 3, 4, 5 and 6, respectively. To further study the behaviour of each pile, the effective plastic strain diagrams and blast damage in each pile were examined as illustrated in Fig. 10. Fig. 10(a) and (d) depict that the pile remained in the elastic state in the cases 2 and 5, and no pile damage was observed. In the cases 3 and 6, although the pile remained almost in the elastic state, concrete was damaged at the bottom of the pile as shown in the Fig. 10(b) and (e). Based on the effective plastic strain diagram in Fig 10(c), the pile is subjected to considerable damage in case 4.

Fig. 9. Results for pile horizontal deformation in (a) case 2 (b) case 3 (c) case 4 (d) case 5 (e) case 6

Fig. 10. Results for effective plastic strain diagram of concrete in (a) case 2 (b) case 3 (c) case 4 (d) case 5 (e) case 6

By comparing the results obtained in each case, it can be seen that the piles are not vulnerable to ground vibration generated by rock blasting at a scaled distance greater than $1.1 \text{ m/kg}^{1/2}$ and it becomes significant when blasting occurred close to the pile (in this study, the scaled distance is equal to $0.75 \text{ m/kg}^{1/2}$). When the blast waves travel through the rock/soil interface, a large percentage of energy dissipates. Thus, the blast waves that propagate in the soil are weaker than the waves that travel in the rock. Moreover, blast waves are transmitted to the bottom of the pile first, because of the rock has a higher wave propagation velocity than the soil. Thus, the bottom of the pile is subjected to a larger blast pressure and stresses are highly concentrated at the pile bottom. Because of this, the pile bottom can fail in shear before it reaches its plastic moment capacity.

4.2 Effect of soil properties on the blast response of pile

In the above 6 cases, soil density and seismic velocity of the soil was considered as 1450 kg/m^3 and 175 m/s , respectively. Thus, another case was considered (case 7) by modifying the soil properties used in the case 4 to study the influence of soil properties on the blast response of pile. Corresponding soil parameters were changed by assuming the soil density as 1960 kg/m^3 and the seismic velocity of the soil as 500 m/s .

The results of case 7 are shown in Fig. 11. In Fig. 11(a), it can be clearly seen that the lateral pile head displacement is 82 mm . Also, Fig. 11(b) shows that pile has severe damage, especially at the both ends of the pile.

Fig. 11. Results in case 7 (a) Pile horizontal deformation (b) Effective plastic strain diagram of concrete

Horizontal deformations of the piles obtained in case 4 and case 7 are compared as shown in Fig. 12. The shape of pile deformation for both cases are similar for the specific blast load.

However, amplification in pile deformation can be observed in case 7, due to the larger blast energy transfer into the soil through the rock/soil interface for case 7.

Fig. 12. Comparison of pile deformations obtained in the cases 4 and 7

4.3 Effect of pile ends conditions on the blast response of pile

In order to assess the influence of pile ends conditions on the blast response of pile, 4 different analysis cases are considered in this section as shown in Fig. 13. The above-mentioned case 7 was used as a base case of analysis. Thus, in all the analyses, 600 mm diameter and 8 m long RC piles are embedded in a single layered soil with the density = 1960 kg/m³, the seismic velocity of soil = 500 m/s. A single pile (rock-socketed) without a pile cap was considered in the case 8 as shown in Fig. 13(b). Thus, the pile has free boundary conditions at the pile head level in this case. A pile group of 2 piles connected by a grade beam through pile caps as illustrated in Fig. 13 (c) and (d) were considered in cases 9 and 10. However, in case 9 the leading pile (pile A) is socketed in rock and the tailing pile (pile B) stands in soil while both leading and tailing piles (piles A and B) are socketed in rock in case 10.

Fig. 13. Analysis cases for effect of pile ends conditions on blast response of pile (a) case 7 (b) case 8 (c) case 9 (d) case 10

Horizontal deformations of the piles obtained in cases 7 to 10 are compared as shown in Fig. 14. The comparison of the results obtained in cases 7 to 9 is shown in Fig. 14(a) while Fig. 14(b) compares the pile deformations observed in cases 9 and 10. The shape of pile deformation is similar in almost all cases. In figures, it can be clearly seen that the pile with fixed head condition (case 7) moved less than the free head pile (case 8) due to the effect of pile head restraint contributing to resist the moment induced by blasting vibration. Also, pile groups (cases 9 and 10) have smaller pile head displacement when comparing with the cases of the single pile (cases 7 and 8). This is because of the pile-soil-pile interactions that take place in the group.

Fig. 14. Comparison of pile deformations obtained in (a) cases 7 to 9 (b) cases and 9 and 10

To further study the behaviour of each pile, the concrete effective plastic strain variation of the piles were examined as illustrated in Fig. 11(b) and 15. It shows that concrete in the both ends is severely damaged in all the cases except in the tailing pile (pile B) in case 9. It can be noted that free head pile suffered the most damage compared to the other piles. Also, it can be noted that the pile B is being shielded by the pile A and thus subjected to less damage. However, the bottom of the pile B has more damages in case 10 compared to those in case 9. This is because of the bottom of the pile is subjected to a larger blast pressure and stresses are highly concentrated at the pile bottom.

Fig. 15. Results for effective plastic strain diagram of concrete in (a) case 8 (b) case 9 (c) case 10

4.4 Effect of slope of rock-soil interface on the blast response of pile

In all the above cases, a downslope rock-soil interface was considered with the slope angle of 58° to horizontal. Thus, another 2 cases were considered by changing the slope angle as shown in Fig. 16 to study the influence of slope of the rock-soil interface on the blast response of pile.

Fig. 16. Analysis cases for effect of slope of rock-soil interface on blast response of pile (a) case 11
(b) case 12

Horizontal deformations of the piles obtained in cases 9, 11 and 12 are compared as shown in Fig. 17. Figure 17(a) presents the horizontal deformations of the leading piles (pile A) in the pile groups. Figure 17(b) shows that the residual lateral deflections of the tailing piles (pile B) in the pile groups. It can be clearly seen that leading piles of the 2-pile groups are more deformed than the tailing piles. They have pile head displacements of 35 mm, 18.5 mm and 8.8 mm in cases, 9, 11 and 12, respectively. However, it was found that the pile A is subjected to a maximum deformation of 46.1 mm at a height of 6 m above the pile bottom and 40 mm at the mid-height, in cases 11 and 12, respectively.

Fig. 17. Comparison of pile deformations obtained in cases 9, 11 and 12 (a) leading pile of pile groups (b) tailing pile of pile groups

Figure 18 shows the concrete effective plastic strain variations of the piles were observed on the pile cases 11 and 12. It is clear that bottom of the pile A were critically damaged in both two cases as same as in case 9 (Fig. 15(b)). Also as expected, pile B has less damage and deformations due to shielding effect from the pile A. However, deformed shape of the pile is different in each case. It can be noted that the top of the pile suffered the most damage in the analysis case 9 compared to the other two cases while concrete in the middle of the pile suffered most damage in the analysis case 12. Therefore, the slope of the rock-soil interface greatly influences the location of damage and the extent of damage on the pile.

Fig. 18. Results for effective plastic strain diagram of concrete in (a) case 11 (b) case 12

5. Conclusion

The dynamic response of reinforced concrete pile foundation to ground-borne vibration from rock blasting has been evaluated using the commercial computer program LS-DYNA. Since a fully coupled technique has been used in the present numerical simulation, the detonation process of an ANFO explosive was modelled in the FE model and calibrated with the experimental results identified in the literature. 3D Simulation of blast wave propagation in rock/soil media using LS-DYNA has been studied, and the results were compared with the PPV values recorded at a site in the western part of Singapore, from controlled blasting. A downslope soil-rock interface was considered in the present simulation, and equivalent properties were used to simulate the rock and soil. The present simulation results reasonably agree with the test results and can therefore be used in the further simulation. Based on the numerical simulation results, a ground vibration attenuation equation is proposed.

Blast responses of piles were studied for different blast scenarios when fully coupled charges were detonated. The influence of standoff distance, charge weight, soil properties, pile ends conditions and slope of rock-soil interface on the dynamic response and damage of piles were studied. However, due to the high computational time and convergence problems, a perfect bonding was considered at the soil-pile interface. Moreover, the axial load acting on the pile

was not considered in this study. Based on the parameters considered in the study and the presented results, the following main conclusions can be drawn.

1. Blast waves are transmitted to the bottom of the pile first, because the rock has a higher wave propagation velocity than the soil. In addition, because the wave attenuation in the rock is slower than in the soil, the bottom of the pile is subjected to a larger blast pressure and stresses are highly concentrated at the pile tip. Because of this, the pile tip can fail in shear before it reaches its plastic moment capacity.
2. It was found that the pile deformations and damage to the pile decays dramatically with the stand-off distance and the pile response increases with charge weight. Furthermore, it was found that the soil properties have a significant influence on the blast response of the pile for a specific blast load. Moreover, the slope of the rock-soil interface greatly influences the location of damage and the extent of damage on the pile.
3. The effect of pile head restraint of the pile improved the overall pile performance by decreasing the displacement of the pile and blast damage on the pile. Because of the pile-soil-pile interactions that take place in a pile group, blast damages on individual piles in the group are lesser than compared to the case of a single pile. Also, it can be noted that the tailing pile in the group is shielded by the leading pile in the group and thus subjected to less damage from a blasting vibration.

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Fig. 1. Detonation of ANFO at (a) $t = 15\mu s$ (b) $t = 25\mu s$ (c) $t = 50\mu s$ (d) $t = 100\mu s$

Fig. 2. Velocity of detonation (VoD) vs. radius of explosive charge

Fig. 3. A half symmetrical model

Fig. 4. Relationship between PPV and SD

Fig. 5. Illustration of the location of blasting and monitoring points

Fig.6. Measured PPV from the field test and numerical simulation

Fig. 7. 3D FE model

Fig. 8. Results in case 1 (a) Pile horizontal deformation (b) Effective plastic strain diagram of concrete

(a)

(b)

(c)

(d)

(e)

Fig. 9. Results for pile horizontal defromation in (a) case 2 (b) case 3 (c) case 4 (d) case 5 (e) case 6

(a)

(b)

(c)

(d)

(e)

Fig. 10. Results for effective plastic strain diagram of concrete in (a) case 2 (b) case 3 (c) case 4 (d) case 5 (e) case 6

(a)

(b)

Fig. 11. Results in case 7 (a) Pile horizontal deformation (b) Effective plastic strain diagram of concrete

Fig. 12. Comparison of pile deformations obtained in the cases 4 and 7

Fig. 13. Analysis cases for effect of pile ends conditions on blast response of pile (a) case 7 (b) case 8
(c) case 9 (d) case 10

(a)

(b)

Fig. 14. Comparison of pile deformations obtained in (a) cases 7 to 9 (b) cases and 9 and 10

Fig. 15. Results for effective plastic strain diagram of concrete in (a) case 8 (b) case 9 (c) case 10

Fig. 16. Analysis cases for effect of slope of rock-soil interface on blast response of pile (a) case 11
(b) case 12

(a)

(b)

Fig. 17. Comparison of pile deformations obtained in cases 9, 11 and 12 (a) leading pile of pile groups (b) tailing pile of pile groups

Fig. 18. Results for effective plastic strain diagram of concrete in (a) case 11 (b) case 12

Table 1. I & G model parameters for ANFO

Unreacted explosive parameters		Reaction products parameters		Reaction rate parameters	
r_1 (GPa)	150	A (GPa)	151.8	I (μs^{-1})	40
r_2 (GPa)	-2	B (GPa)	0.715	a	0
r_3 (GPa/K)	0	g (GPa/K)	2.9×10^{-4}	b	0.222
r_5	11	x _{p1}	5	c	1
r_6	15	x _{p2}	1	d	0
				G_1 ($\mu\text{s}^{-1} \text{ GPa}^{-1}$)	0.05
				G_2	0
				x	4
				y	1

Table 2. Material parameters for rock

Density (kg/m³)	Young's modulus (GPa)	Poisson's ratio	Yield stress (MPa)	Tangent modulus (GPa)	Hardening Parameter (β)
2650	61.41	0.27	154.57	24.56	1

Table 3. Material parameters for soil

Density (kg/m³)	Elastic shear modulus (MPa)	Poisson's ratio	Friction angle	Cohesion (kPa)
1450	12.7	0.3	30°	22

Table 4. PPV values – numerical results

W (kg)	PPV (mm/s)		
	D = 10m	D = 20m	D = 30m
4	41	17	7
7	95	32	11
10	118	35	28
11.5	120	40	31
21	185	64	34
23	224	88	36
40	315	107	46
50	370	188	62

Table 5. Ground vibration monitoring data

Distance (m)	Charge weight (kg, ANFO)	Scaled distance (m/kg^{1/2})	Measured PPV (mm/s)
30	23	6.26	25.32
60	21	13.09	7.36
80	23	16.68	6.88
80	21	17.46	6.35

Table 6. Material parameters for steel

	Density (kg/m³)	Young's modulus (Gpa)	Poission's ratio	Yield stress (Mpa)	Tangent modulus (Gpa)	Hardening Parameter (β)	C	P
Vertical R/F	7850	210	0.3	460	2	0	40	5
Ties	7850	210	0.3	250	2	0	40	5

Table 7. Analysis cases

Analysis case	Charge weight (kg)	Distance (m)	Scaled distance (m/kg^{1/2})
1	11.5	7.5	2.21
2	23	7.5	1.56
3	46	7.5	1.11
4	100	7.5	0.75
5	46	11	1.62
6	100	11	1.10