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Blast response and failure analysis of pile foundations subjected to surface explosion

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ABSTRACT

This paper presents the response of pile foundations to ground shocks induced by surface explosion using fully coupled and non-linear dynamic computer simulation techniques together with different material models for the explosive, air, soil and pile. It uses the Arbitrary Lagrange Euler coupling formulation with proper state material parameters and equations. Blast wave propagation in soil, horizontal pile deformation and pile damage are presented to facilitate failure evaluation of piles. Effects of end restraint of pile head and the number and spacing of piles within a group on their blast response and potential failure are investigated. The techniques developed and applied in this paper and its findings provide valuable information on the blast response and failure evaluation of piles and will provide guidance in their future analysis and design.

Key words: underground explosion; pile foundation; reinforced concrete; coupled analysis, material models, numerical simulation, pile damage.

1. INTRODUCTION

Significant increase of terrorist attacks against significant and iconic buildings and other infrastructure components highlight the need for the blast resistant design of the structures. Blast loads are short duration dynamic loads and their durations are very much shorter (about 1000 times) than those of earthquakes. Thus, structural response under blast loading could be significantly different from that under a loading with a much longer duration such as an earthquake.

Many studies on blast resistant design have been carried out by the military services and the relevant documents are restricted to official use only. In the open literature, much effort has been spent in investigating the dynamic response and damage of structures to blast loading using different approaches such as analytical methods, experiments and numerical analyses. In analytical methods, the problem is solved using a theoretical model under appropriate assumed conditions. However this method is only applicable to simple problems. Small scale or prototype experiments using explosions are very expensive. They also require the use of large amount of explosives, involving risk and danger. Thus, they are typically not feasible in civilian research. These experiments are mainly carried out by military services. With the recent development of computer hardware technology, increased research in numerical simulation of partial differential equations, finite element (FE) modeling and simulations provide a viable and cost effective method for detailed investigation of blast response and failure analysis of structures under different blast scenarios.

The numerical analysis of structures under blast loading can be divided into three phases such as the detonation of explosive charge to form blast shock waves, the propagation of the blast shock waves in the medium (air, water, soil or rock) and the response of structure to the produced blast shock waves. This analysis can be achieved through an un-coupled or a coupled method. In the uncoupled method of analysis, the main physical procedure is divided into several successive stages. The free field stresses are measured first and then these stresses are applied on the structure to evaluate its response. Many numerical investigations have been carried out using the uncoupled method [1, 2]. In the coupled method all the stages are included in a single model. Anirban De [3] used a fully coupled numerical model to study the effects of a surface explosion on an underground tunnel using a 3D Finite Element model.

The uncoupled method has been used in the analysis of blast loads on structures in different ways in the finite element (FE) computer program LS-DYNA [4]. First, the time histories of blast pressure are computed empirically with ConWep [5] and are directly applied on the Lagrangian elements of the structures. In addition, `LOAD_BLAST` and `LOAD_BLAST_ENHANCED` options available in LS-DYNA can be used to generate blast pressure histories on the structure. As modelling of the detonation process of the explosive charge is not involved, this modelling technique reduces the computational cost. However, this method cannot be applied with confidence for near field problems because of the complexity of the flow processes involved in the formation of a blast wave [6].

The Multi-Material Arbitrary Lagrangian Eulerian (MM-ALE) method can be used for the fully-coupled approach including the explosive in LS-DYNA. In this method, two separate meshes, one for the explosive and another for the surrounding medium (soil or air) are required; and they are modeled using multi-material ALE formulation. Multi-material option enables up to three different materials to be modeled within same mesh [7]. Thus, using this technique, the explosive product is able to expand into the meshes initially occupied by the soil or air.

Previous studies mainly investigated effect of the loads induced on structural components by air propagated blast shock waves. Relatively less attention has been paid towards the blast loading on and response of foundations. Pile foundations transfer the large loads from the superstructure above into deeper, competent soil layers which have adequate capacity to carry these loads. It follows that if these foundations are structurally damaged or fail due to blast loading, the superstructure becomes vulnerable to failure. It is noted that there is little literature reporting on numerical studies of pile and pile groups under lateral loading [8-12].

This paper describes the three dimensional FE modeling and analysis to study the response of Reinforced Concrete (RC) piles (single piles and pile groups) founded in partially saturated soil subjected to blast loads induced by explosion on the ground surface. The present study adopts the fully coupled numerical simulation approach employing nonlinear material models to represent the realistic behavior of the soil-pile system. The paper is organized as follows. Section 2 describes the developed FE model to study the blast wave propagation in soil medium. Two types of boundary conditions at the pile head, fixed head and free head, are considered for the purpose of comparison. Their results are presented and discussed in sections 3 and 4. Section 5 presents some additional results and discussion on the simulated blast response and damage analysis of pile groups. Section 6 presents some concluding remarks.

2. VALIDATION OF MODELLING TECHNIQUES AGAINST EXPERIMENTS

Using the results from experiments [13], the authors validated the modeling techniques and the analysis procedure used to study the blast response of a pile [10]. This validation is briefly discussed in this section. Towards this validation, the authors developed a 3D finite element (FE) model [10] by considering the prototype dimensions in Shim's experiments [13]. Blast wave propagation in the soil and the horizontal pile deformations were studied and the

results were compared with the experimental results in [13]. Figure 1(a) shows the comparison of their blast wave pressures in the soil with Shim's [13] results. Figure 1(b) plots the horizontal residual deformations of the pile along its height for three different standoff distances obtained from the authors' study [10] and those from reference [13]. From these Figures, it is clearly evident that the modeling techniques used by the authors are adequately validated and provide adequate confidence to use these techniques in further studies as done in the present paper. A detailed description of the modeling techniques and the validation can be found in Jayasinghe et al. [10].

Figure 1. Validation of modeling techniques (a) Blast wave propagation in the soil [10] (b) Residual horizontal pile deformation [10]

3. PROPAGATION OF BLAST INDUCED WAVES IN THE SOIL

The study of wave propagation in soils can produce useful information to engineers on the resilient characteristics of a particular site, dynamic soil structure interaction and earthquake analysis. An explosion on the ground surface generates both air-blast pressure and ground shock on structures which are close to the detonation point. However, wave propagation velocities are different for geo-materials and air, and as a consequence, the ground shock excites the structure foundation earlier before the air blast pressure arrives at the structure. Both the ground shock and the air blast pressure might act on the structure simultaneously. This depends on the distance between explosion center and the structure and ground motion properties [14]. But in most scenarios, the ground shock excites the structure before the air blast pressure. Also, surface explosion can induce large stresses and strains in the subsurface soil. Therefore, it is important to investigate the blast wave propagation in soil. Some research on the behavior of soil under blast loading can be found in the bibliography [15-17]. In the present study, effect of explosion on a soil medium is first studied, and the method used and results are described in this section.

3.1. Numerical model

Today, a vast numbers of FE codes that are capable of addressing challenging engineering problems are available. This study was mainly carried out using the explicit nonlinear FE modelling code LS-DYNA. Pre-processing of the FE models, including mesh generation and

application of boundary condition, was performed using MSc Patran. The *.key* file, which is the output of MSc Patran, was modified in LS-DYNA with appropriate parameters. The parameters to be modified in LS-DYNA are material models, equation of states, boundary conditions and controlling parameters such as hour-glassing and termination. LS-PREPOST was used to visualize the complied results.

A FE model was developed to study the blast wave propagation in soil. By making use of symmetry, to save computational time, only a quarter of the system was modeled as shown in Figure 2. It was modeled with the soil 30m high and the explosion occurring on the soil surface. The soil was considered as partially saturated soil as shown in Table 1. The explosive charge used in the tests was 1000 kg TNT and was assumed to have a spherical shape. The same modelling techniques as those described in [10] were adopted in the present study.

Figure 2. Finite element model to study the blast wave propagation in soil

Table 1. Soil properties for numerical simulation

The FE model consists three different parts representing the soil, air and the explosive material. One of the most difficult tasks associated with FE modeling is the selection of appropriate material properties to accurately model the physical behavior. Upon evaluation of available soil material models in LS-DYNA, soil was modeled using FHWA soil material model. This material model accounts for geometrical non linearity, material non linearity, and pore water pressure development. For most soil mechanics problems, it is sufficient to use Mohr-Coulomb failure criteria. However, the standard Mohr-Coulomb surface has some deficiencies. The surface comes to a point at the intersection with the stress axis (zero shear strength). This type of singularity can cause problems in numerical computations. To avoid such an angularity, a modified Mohr-Coulomb failure criterion as described in [18] was adopted in this material model.

The explosive charge was modeled using the high explosive burn material model and the Jones-Wilkin-Lee (JWL) equation of state (EOS). The JWL equation of state defines the pressure as a function of the relative volume, V and initial energy per volume, E , such that [4]

$$P = A \left(1 - \frac{\omega}{R_1 V} \right) e^{-R_1 V} + B \left(1 - \frac{\omega}{R_2 V} \right) e^{-R_2 V} + \frac{\omega E}{V} \quad \text{Eq. 1}$$

In the above equation, A , B , R_1 , R_2 and ω are constants pertaining to the explosive. Material parameters and JWL constants for TNT explosive are available [10] and were used in these simulations.

Air was modeled using null material model with a linear polynomial EOS, which is linear in internal energy per unit initial volume, E and the pressure P , and is given by [4]

$$P = C_0 + C_1 \mu + C_2 \mu^2 + C_3 \mu^3 + (C_4 + C_5 \mu + C_6 \mu^2) E \quad \text{Eq. 2}$$

In the above equation, C_0 , C_1 , C_2 , C_3 , C_4 , C_5 , and C_6 are constant and $\mu = \frac{\rho}{\rho_0} - 1$. Where $\frac{\rho}{\rho_0}$ is the ratio of current density to initial density. Material parameters and EOS constants for air are available [10] and were used in these simulations.

3.2. Numerical results and discussion

Figure 3 shows the progressive wave propagation in the soil at different time incidents. It demonstrates that the pressure waves propagate in the soil in the form of hemispherical waves, with the area of wave front increasing with the wave propagation.

Figure 3. Pressure contours in the soil at different times after the detonation

The peak pressures measured in the soil are plotted against the scaled distance in Figure 4. The results show that attenuation of the peak pressure in the soil occurs with increasing distance from the charge.

Figure 4. Variation of peak pressure with scaled distance

TM5-855-1 Manual [19] has been widely used for prediction of ground shock parameters. It provides the following equation to predict the peak pressure in the soil.

$$P_0 = 160.f.\rho c.\left(\frac{R}{W^{1/3}}\right)^{-n} \quad \text{Eq. 3}$$

In these equations, P_0 is the peak pressure in psi, f is a coupling factor and is dependent on the scaled depth of the explosion, ρc is acoustic impedance in psi/fps, R is distance from the explosive source in ft, W is the charge weight in lb, c is the seismic velocity in fps, and n is an attenuation factor which is dependent on the soil type as shown in Table 2 [19]. Coupling factor, f , is different for bursts in air, soil and concrete and depends on the scaled depth of the explosive source. The coupling factor for air is a constant and it is equal to 0.14. This value is also recommended for the surface explosions [19].

Table 2. Soil properties for calculating ground shock parameters [19]

Figure 5 compares the peak pressures obtained from the numerical simulation with the predicted peak pressures from the manual TM5-855-1. As shown in the figure, the numerical results for the peak pressure attenuation agree reasonably well with the empirical results from the manual. The slightly lesser value might be caused by the complicated properties of the soils, which are simplified in the numerical model.

Figure 5. Comparison of peak pressures from numerical analysis and predictions from manual (TM5-855-1)

4. BLAST RESPONSE AND FAILURE ANALYSIS OF FIXED-HEAD RC PILE

A Reinforced Concrete (RC) pile in a partially saturated soil subjected to blast loading may fail in several modes such as bending failure, compressive failure of concrete, spalling of concrete, shear failure and excessive settlement. Blast loads usually have a short duration and high amplitude. Thus, shear stress develops quickly to a high value, while the flexural deflection does not have adequate time to develop. As a consequence, shear damage is likely to occur. The shear failure is most likely to occur at the top and/or bottom of the pile because of the fixity at these locations. Sometimes, the failure of the pile can be due to a combination of the shear and flexural damages. However, the damage modes also depend on the properties of the pile. This study aims to investigate the response and damage of the RC pile when subjected blast loads for a standoff distance of 7.5m in partially saturated soil.

4.1. Numerical model

A finite element model was developed for a 10m length pile with 600mm diameter circular cross section using explicit dynamic nonlinear finite element software LS-DYNA [4]. The modeling technique used in the present study are the same as in Jayasinghe et al. [10]. The overall FE model has different regions representing the soil, air, pile and explosive charge as shown in Figure 6. By making use of symmetry, only a quarter of the system was modeled. The bottom of the mesh was constrained in the all the directions to represent the bed rock. To form the symmetry in the FE model, the translational displacements of nodes normal to symmetry planes were constrained. The nodes along the interfaces between the air and soil were merged. Fixed boundary conditions were considered at the top and bottom of pile

Figure 6. Finite element model to study blast response of pile

Except for the reinforcing cage, the eight-node solid elements were adopted for the 3D explicit analysis. The TNT explosive, the air and part of soil close to the explosive were modeled with ALE multi-material meshes. This is to prevent the element distortion in large deformations. On the other hand Lagrangian meshes were used to model the pile and the soil region away from the explosive charge. 25mm long beam elements with 2x2 Gauss integration were used for both the vertical reinforcements and ties. The vertical reinforcements were defined as Hughes-Liu beam elements with cross integration and ties were defined as truss elements.

LS-DYNA contains several material models that can be used to represent concrete. The material model Concrete_Damage_REL3 was used in this investigation for the concrete. It is a plasticity-based model with three shear failure surfaces and includes damage and strain rate effects [20]. The literature has shown that material concrete_damage_rel3 model can successfully incorporate non-linear concrete properties [21, 22]. The advantage of this model is that unconfined compressive strength and density of concrete are the two parameters that are required in the calibration process. This concrete material model uses three failure surfaces; namely an initial yield surface, a maximum failure surface and a residual surface with consideration of all the three stress invariants [20]. Hence it can effectively simulate tri-axial state of stress conditions.

Pile reinforcement is normally required to resist the bending and tensile stresses, but may be used to carry a portion of the compression load. 20mm diameter 16 number of bars were used as the pile vertical reinforcement in this study. 10mm diameter steel bars were used for the transverse reinforcement. Transverse reinforcement ratio of 0.24% was used in piles provided at a spacing of 200mm. Reinforcements were modeled as elastic perfectly-plastic materials using the plastic kinematic model available in the LS-DYNA. Kinematic hardening with strain rate effects was implemented for the reinforcement.

The erosion algorithm was used to simulate the crushing of concrete in the finite element model. When the material response in an element reaches a certain critical value, the element is immediately deleted. This provides an efficient means to imitate concrete spalling phenomena and produce graphical plots which are more realistic representations of the actual events. There may be a variety of criteria governing the material erosion. In this study, the concrete elements in the pile were allowed to erode when the principle tensile strain reached 0.01 [23].

4.2. Numerical results and discussion

The horizontal deformation and damage of the pile were obtained at 7 monitoring points on the pile at different heights from the pile tip (bottom): 0.5m (point A), 2m (point B), 4m (point C), 5m (point D), 6m (point E), 8m (point F) and 9.25m (point G).

Figure 7 shows the time histories of the horizontal deformations of the pile. The horizontal deformations have a significant influence on pile damage and failure. This Figure demonstrates that the pile has residual deflections along its height. These residual deflections indicate the occurrence of plastic deformation of the pile, which means that the pile has suffered permanent deformation and local failure under the buried blast event. Figure 8 presents the residual horizontal deformations of the pile along its height. The maximum residual deformation of 25.3mm occurs at the monitoring point E located 6m above from the pile tip. This could mean that point E is a potential failure region of this pile under the blast loading.

Figure 7. Fixed-head pile deformations

Figure 8. Residual horizontal deformations of fixed-head pile along its height

The damage to the reinforced concrete is also observed from the present numerical simulation. Figure 9 depicts the concrete effective plastic strain variation of the pile with the element erosions that were observed on the pile. Effective plastic strain is the damage parameter in concrete_damage_rel3 material model which range from 0 to 2. The colors in the Figure indicate the fringe level which represents the level of damage in the concrete. The blue color represents the fringe level 0 which indicates elastic state of the concrete, while the red color represents the fringe level 2 which indicates the complete yielding of the concrete. The other colors which are associated with fringe levels between 0 to 2 represent the different damage levels of the concrete. Figure 10 shows the effective plastic strain diagrams of the concrete cross sections taken at the pile ends and the mid-height. As can be seen, it is evident that the pile was considerably damaged at it ends due to shear force generated by the blast loading. Also, the strain diagram in Figure 10(b) indicates that the pile was subjected to damage in about 50% of its section at its mid-height. Potential failure locations of this pile are therefore at the two ends and near mid height where the horizontal deflection was large (as shown in Figure 8).

Figure 9. Blast damage on fixed-head pile

Figure 10. Effective plastic strain diagram of the concrete cross sections at (a) pile top end (b) mid-height of pile and (c) pile bottom end

5. BLAST RESPONSE AND FAILURE ANALYSIS OF FREE-HEAD RC PILE

Using the FE simulation techniques discussed above, further studies were carried out to evaluate the dynamic response of free-head RC pile subjected to a surface blast. All translations and rotations were restrained at the bottom end of the pile, as before to depict fixed end conditions. At the top end of the pile both translations and rotations were allowed in all the directions. However, an axial load on the pile was considered in this study. It was added to the pile by placing 3.75m x 3.75m x 1.875m concrete cube on the top of the pile. The concrete cube simulated an axial load of 600kN on the pile. Figure 11 shows an isometric view of the developed model for the free-head pile with the concrete cube at the top of the pile.

Figure 11. Numerical model of free-head pile

5.1. Numerical results and discussion

In Figure 12, the time histories of the horizontal deformations of the pile at different heights from the pile tip: 0.5m (point A), 2m (point B), 4m (point C), 5m (point D), 6m (point E), 8m (point F), 9.25m (point G), and 10m (point H) are presented. It demonstrates that the pile has suffered permanent deformation and possible (local) failure under the blast load induced by surface explosion. Figure 13 presents the residual horizontal deformations of the pile along its height. The maximum residual deformation of 38mm occurs, as expected at the monitoring point H which is the pile head.

Figure 12. Free-head pile deformations

Figure 13. Residual horizontal deformations of free-head pile along its height

Figure 14 compares the numerical results for the pile deflection obtained in the fixed-head and free-head piles. It is evident that the pile end conditions have significant effect on the pile response under the blast load with respect to the maximum pile deflection and deflected shape of the pile.

Figure 14. Comparison of pile deformations in fixed-head and free-end pile models

Moreover, in order to investigate the effect of axial load on the blast response of pile, further studies were carried out by increasing concrete cube size to 4.45m x 4.45m x 2.225m. It simulated an axial load of 1000kN on the pile. Figure 15 illustrates the effect of axial load on the displacement response of the pile under blast loading. As the figure demonstrates, horizontal pile head displacement of the pile with larger axial load is slightly larger. However, further studies are needed to enable firm conclusions to be made on the effect of axial load on the response of piles under different blast scenarios.

Figure 15. Effect of axial load on the displacement response of pile

6. BLAST RESPONSE AND FAILURE ANALYSIS OF PILE GROUPS

Piles are usually constructed in groups and tied together by a pile cap at the ground surface. The response of a pile within a group of closely spaced piles could be different from that of a single pile because of the pile-soil-pile interactions that take place in the group. LS-DYNA models were developed to study the blast response of pile groups. Three analysis cases were considered for the purpose of comparison as shown in Table 3. Cases 1 and 2 pertain to groups of 2 piles that are closely and widely spaced respectively while case 3 pertains to a group of 4 closely spaced piles. Figure 16 (a) and (b) show the FE models developed for the 2-pile (cases 1 and 2) and 4-pile (case 3) groups, respectively. The most important factor is the pile spacing in a pile group. Therefore, two FE models were created with pile spacing of 2.5 times and 5 times the pile diameter for the blast analysis of the pile groups with closely-spaced and widely-spaced piles respectively. Only half of each model is meshed using symmetry conditions. The rotational restraint at the pile cap connection was considered in all cases.

Table 3. Selected analysis cases

Figure 16. FE models of (a) 2-pile group (b) 4-pile group

6.1. Numerical results and discussion

In this subsection, results related to the behavior of pile groups in partially saturated soil under blast loads induced by surface explosion are presented. The residual horizontal displacements along the height of each pile in the 2-pile group in analysis case-1 are plotted in Figure 17. As can be seen, it is clear that front pile of the 2-pile group was more deformed than the rear pile. These residual horizontal deformations have a significant influence on pile damage and failure.

Figure 17. Comparison of residual horizontal deformations in front and rear piles for case 1

Figure 18 compares the numerical results for the pile deflections obtained in all three cases. Figure 18(a) presents the residual horizontal displacements of the front piles in the pile groups. They have maximum lateral residual deflections of 37.5 mm, 40 mm and 37mm in cases, 1, 2 and 3 respectively. Figure 18(b) shows that the residual lateral deflections of the

rear piles in the pile groups. Maximum lateral residual deflections of approximately 35.7mm, 36 mm and 31.5mm are observed in cases 1, 2 and 3 respectively. Piles in the widely-spaced pile group (case 2) have slightly higher pile head displacements compared to piles in the closely-spaced pile group (case 1). This is due to the lower stiffness and the reduced pile-soil-pile interaction in the widely-spaced pile foundation system. In the closely-spaced 2 pile and 4 pile systems (case 1 and case 3 respectively) the rear piles have maximum pile head displacements of 35.7 mm and 31.5 respectively. Hence, the number of piles in a group and the spacing between piles have an influence on the pile head displacement under surface blasts.

Figure 18. Comparison of pile responses for case 1 to 3 (a) front pile of pile groups (b) rear pile of pile groups

To further study the behavior of each pile, the effective plastic strain diagrams and blast damage in each pile in the pile groups were examined. Figure 19 shows the concrete effective plastic strain variation of each individual pile in the pile groups. As described earlier, the effective plastic strain in Mat72_rel3 concrete material model is the damage parameter which ranges between 0 and 2, with fringe levels 0 and 2 indicating no yielding and maximum yielding of the concrete, respectively. As can be seen in the figure, a significant portion of each pile has suffered damage and local failure, as the effective plastic strains are greater than 0. Piles in the pile group in case-1 suffered more damage than the piles in the other two cases. However, the results for each case did not show any signs of catastrophic failure.

Figure 19. Blast damage on pile groups for analysis (a) case-1 (b) case-2 (c) case-3

7. CONCLUSION

A fully coupled numerical procedure incorporating different material models was developed and applied to study the dynamic response of reinforced concrete pile foundations to blast loads using the commercial computer program LS-DYNA. The modelling techniques used in the present paper have been validated as described in an earlier paper [10]. Single piles and pile groups with different pile numbers and spacing were considered. The soil was assumed

to be partially saturated and was modeled using FHWA soil material model. Blast wave propagation in soil was validated with the predicted pressures using the TM5-855-1. The Concrete_Damage_REL3 material model was used to simulate the concrete with the reinforcement considered as elastic-perfectly plastic material. Both free head and fixed head boundary conditions at the pile head were considered in the analysis of single piles. Numerical results show that pile head boundary conditions have significant effects on the pile response under blast loading. Moreover, the effect of axial load on the blast response of pile was also investigated. It was found that horizontal displacement of the pile with the larger axial load is slightly higher for the considered RC pile and blast load condition. Piles in three different arrangements of pile groups were considered to study the blast response of pile groups. Displacement responses of pile groups indicated that piles in the widely-spaced pile group have slightly larger pile head displacements which are attributed to the lower stiffness and reduced pile-soil-pile interaction of the widely-spaced pile foundation system. It was also evident that the number of piles in a group and the spacing between the piles within a group have an influence on the lateral pile head displacements. Residual lateral deflections and effective plastic strain diagrams of the piles were presented. They can be used to identify local damage in the piles and their potential failure under surface blast loading. The modeling techniques developed and applied in this paper and its research outcomes can be useful in future studies on the blast response and failure analysis of pile foundations.

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(a)

(b)

Figure 1. Validation of modeling techniques (a) Blast wave propagation in the soil [10]
(b) Residual horizontal pile deformation [10]

Figure 2. Finite element model to study the blast wave propagation in soil

Figure 3. Pressure contours in the soil at different times after the detonation

Figure 4. Variation of peak pressures with scaled distance

Figure 5. Comparison of peak pressures from numerical analysis and predictions from manual (TM5-855-1)

Figure 6. Finite element model to study blast response of pile

Figure 7. Fixed-head pile deformation

Figure 8. Residual horizontal deformations of fixed-head pile along its height

Figure 9. Blast damage on Fixed-head pile

(a)

(b)

(c)

Figure 10. Effective plastic strain diagram of the concrete cross sections at (a) pile top end (b) mid-height of pile (c) pile bottom end

Figure 11. Numerical model of free-head pile

Figure 12. Free-head pile deformations

Figure 13. Residual horizontal deformations of free-head pile along its height

Figure 14. Comparison of pile deformation in fixed-head and free-end pile models

Figure 15. Effect of axial load on the displacement response of pile

Figure 16. FE models of (a) 2-pile group (b) 4-pile group

Figure 17. Comparison of residual horizontal deformation in front and rear piles for case 1

(a)

(b)

Figure 18. Comparison of pile responses for cases 1 to 3 (a) front pile of pile groups (b) rear pile of pile groups

(a)

(b)

(c)

Figure 19. Blast damage on pile groups for analysis (a) case-1 (b) case-2 (c) case-3

Table 1. Soil properties for numerical simulation

Soil properties	Partially saturated soil
Composition	Sand & Clay
Density	1960 kg/m ³
Degree of saturation	85% ($V_a > 4\%$)
Seismic velocity	500 m/s

Table 2. Soil properties for calculating ground shock parameters [18]

Soil types	Unit weight, (pcf)	Seismic velocity, c (fps)	Acoustic impedance, ρc (psi/fps0)	Attenuation coefficient, n
Heavy saturated clays and clay shale	120 - 130	> 5000	150 - 180	1.5
Saturated sandy clays and sands with air voids < 1%	110 - 124	5000	130	2.25 - 2.5
Dense sand with high relative density	109	1600	44	2.5
Wey sandy clay with air voids > 4%	120 - 125	1800	48	2.5
Sandy loam, loess, dry sands and backfills	124	1000	22	2.75
Loose, dry sands and gravels with low relative density	90 - 100	600	12	3 - 3.25

Table 3. Selected analysis cases

Analysis case	No. of pile in group	Spacing between piles/Pile diameter
1	2	2.5
2	2	5
3	4	2.5