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Effect of Soil Properties on the Response of Pile to Underground Explosion

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Abstract

This paper develops and presents a fully coupled nonlinear finite element (FE) procedure to treat the response of piles to ground shocks induced by underground explosions. The Arbitrary Lagrange Euler coupling formulation is used in the study with proper state material parameters and equations. Pile responses in four different soil types, viz, saturated soil, partially saturated soil, and loose and dense dry soils are investigated and the results compared. Numerical results are validated by comparing them with those from a standard design manual. Blast wave propagation in soils, horizontal pile deformations, and damages in the pile are presented. Pile damage presented through plastic strain diagrams will enable vulnerability assessment of the piles under the blast scenarios considered. The numerical results indicate that the blast performance of single pile foundations embedded in saturated soil and loose dry soil are more severe than those embedded in partially saturated soil and dense dry soil. The present findings can serve as a benchmark reference for future analysis and design.

Keywords: underground explosion; pile foundation; reinforced concrete; numerical simulation; soil properties.

Introduction

Above ground and underground explosion blast waves have been of great interest in civil engineering design with the increasing occurrence of terrorist attacks. Many studies have been carried out on the propagation of blast waves in the air, soil, and rocks.¹⁻³ An explosion provides a sudden release of energy that produces a pressure pulse or shock wave. When a shock wave impacts a structural surface, the shock wave can cause severe structural and equipment damage, as well as human casualties. According to past records, terrorists mainly targeted significant and iconic buildings. Therefore, it is important that these types of buildings and other infrastructure components are shock-hardened to provide safety to both occupants and equipment (or designed to provide safety to both occupants and equipment under credible blast events).

Events such as the truck bomb explosion in the World Trade Center in New

York on 26 February 1993 and Alfred P. Murrah Federal Building bombing incident in Oklahoma City on 19 April 1995 have caused considerable concern, prompting researchers to investigate aspects of design of structures to protect the integrity of structures and their occupants from the adverse effects of bombings.

Many research projects on blast-resisting designs have been carried out by the military services and the relevant documents are restricted for 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 appropriately assumed conditions. However, this method is only applicable to simple problems. Small-scale or prototype experiments on explosion are very expensive. They also require the use of large amounts of explosives, involving risk and danger. These experiments have been mainly carried out by military services. Thus, they are typically not feasible in civilian research. With the recent development of computer hardware technology, increased research in numerical simu-

lation of partial differential equations, FE modeling, and simulations provide a viable and cost-effective method for the detailed investigation of blast response of structures for different blast scenarios. The authors of Refs. [4, 5] carried out numerical simulations using the non linear dynamic FE software LS-DYNA⁶ to study the dynamic response of reinforced concrete columns subjected to blast loads. In Ref. [5] the numerical model is verified through experimental studies and a formula is proposed to estimate the residual axial capacity on the basis of the mid-height displacement. Reference [7] presents a numerical study on the concrete spallation in reinforced concrete slabs under various blast loading conditions. Reference [8] also reports a numerical simulation study conducted on the blast response of reinforced concrete slabs and the investigation of the influence of reinforcement ratio, explosive weight, and standoff distance. The authors of Ref. [9] proposed a method to assess the vulnerability and residual capacity of building frames and components when subjected to near-field blast events.

However, the above-mentioned studies mainly investigated the effect of the loads induced on structural components by air-propagated blast shock waves. Relatively less attention has been paid toward 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 because of blast loading, the superstructure becomes vulnerable to failure. A foundation system can fail even if the piles are not damaged by the blast simply because of 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. The potential damage due to blast load has not received

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proper attention in the current practice of pile design. Thus, the design of pile foundation under dynamic lateral loads induced by the blast remains a challenging issue. This is due to the lack of knowledge on assessing the response of the pile to blast load. This emphasizes the need for a study to determine the blast response and the vulnerability of pile foundations.

Even though researchers have not considered the response of piles to blast loads, some studies on laterally loaded piles can be found in the literature. The behavior of laterally loaded piles using the continuum theory is analyzed in Ref. [10]. It was found that the major factors influencing pile behavior are pile flexibility and the length to diameter ratio, for both fixed-head and free-head piles. The authors of Ref. [11] presented a numerical analysis of single laterally loaded piles embedded in cohesion-less soil, which was modeled as an elastic material. In Ref. [12] the response of a flexible pile to lateral loading using numerical simulation was studied. The soil was treated as an elastic continuum with a linearly varying soil modulus, and a formula was developed to determine the maximum bending moment induced in a free-headed pile.

Although pile foundation is a surface-buried structure, it can be assumed as an underground structure in some aspects. Thus, by reviewing the studies on blast response of underground structures valuable information such as material behavior, different analytical methods, and soil-structure interaction can be obtained, which are useful for studying the pile foundation response subjected to blast load. The performance of underground structures subjected to blast loads is a critical research area, as these structures play an important role in the overall structure response. Underground explosions usually produce a crater, and blast-induced ground shock propagates in the surrounding soil media. If an explosion occurred near a buried structure, the soil pressure and acceleration could result in severe damage or even the collapse of the structure. The evolution of centrifuge tests had led to some studies on the dynamic response of underground structures to blast loading.^{13,14} Centrifuge models were used in Ref. [15] to study the response of piles in saturated soil under blast loading. A series of 70 g centrifuge tests were carried out to investigate blast wave propagation

and the response of piles embedded in saturated sand. Several tests have been carried out on aluminium piles with a hollow circular section at different standoff distances. Recently, different types of numerical methods have been used to investigate the response of underground structures under blast loads. They can be classified as either uncoupled or coupled methods. In the uncoupled method, the main physical procedure is divided into different successive stages. Free-field stresses are measured first and then these stresses are applied to the structure to evaluate its response. Many numerical investigations have been carried out using the uncoupled method.^{16,17} In the coupled method, all the stages are included in a single model. In Ref. [18] blast resistance analysis for the Shanghai metro tunnel using explicit dynamic nonlinear FE software LS-DYNA is discussed. The overall analysis evaluated the safety of the tunnel lining on the basis of the failure criterion. Since there has not been any established common standards governing the design of such a structure, a series of parametric studies have been carried out to evaluate the significance of several parameters such as shear modulus and bulk modulus of soil, on the lining thrust. Ref. [19] investigated the response of a buried concrete structure to various factors affecting structural performance by carrying out a parametric study using an FE model. Depths of the structure and the charge were considered as parameters. It was shown that buried explosions resulted in more significant effects on the buried structure than surface explosions under the same conditions. In Ref. [20] a fully coupled numerical model was used to study the effects of a surface explosion on an underground tunnel using a three-dimensional (3D) FE model.

This study treats the response of a single pile foundation to a buried blast loading using numerical simulations through the commercial software package LS-DYNA. 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 Arbitrary Lagrange Euler (ALE) formulation²¹ is used in the explosion, air domain, and the soil region near the explosion to eliminate the distortion of the mesh under high deformation. A brief description of the background on modeling and material models is presented in the

following sections. Pile foundation response to blast loads is investigated under different soil conditions. A study of blast wave propagation in different soil types under buried explosion is hence presented at the beginning. The response of a single pile to underground explosion is then presented. This study develops and applies a comprehensive FE technique to study the blast response of a single pile foundation. It also evaluates the influence of soil properties and standoff distance on the response of a reinforced concrete pile. Consequently, outcomes of this study will expand the current knowledge on the blast response of a single pile foundation and could serve as a reference for future analysis and design.

Ground Motion and Free-Field Stresses

The study of wave propagation due to an explosion in soils can provide information useful to engineers on the resilient characteristics of a particular site, dynamic soil-structure interaction, and earthquake analysis. When an explosion occurs in the soil, an explosive cavity with high pressure and high temperature gas is formed. The explosive cavity immediately begins to expand against the surrounding soil, causing high initial radial displacements and stresses in soil that propagate outward from the explosive. In the vicinity of the explosion, stresses in the soil are extremely high and the result is that the soil will lose its shear resistance. As the explosion cavity expands, stresses in the soil decrease with distance.²²

The study of blast wave propagation in soils and validation of the soil material model are described in this section. An FE model was developed and validated by comparing the blast wave pressures in the soils obtained from numerical simulations with the predicted peak pressures in the manual TM5-855-1.²² The FE model was developed with the soil 10 m high and the explosion occurring at the mid-depth of the soil. The explosive charge used in the tests was 500 kg Trinitrotoluene (TNT). The same modeling techniques that are described in Ref. [23] were adopted in this study.

Soil Material Model for Blast Study

This study aims to investigate the blast response of a single pile embedded in different soil types. The following soil types, saturated soil, partially saturated

	Soil type 1	Soil type 2	Soil type 3	Soil type 4
Soil type	Saturated soil	Partially saturated soil	Loose dry soil	Dense dry soil
Composition	Clay	Sand and clay	Sand	Sand
Density	2065 kg/m ³	1960 kg/m ³	1450 kg/m ³	1800 kg/m ³
Degree of saturation	100%	85% (volume of air >4%)	0%	0%
Seismic velocity	1575 m/s	500 m/s	175 m/s	300 m/s

Table 1: Soil properties for numerical simulation

soil, and loose and dense dry soil as shown in Table 1, were used.

On evaluation of available soil material models in LS-DYNA, the soil was modeled using the FHWA²⁴ soil material model. This material model accounts for geometrical nonlinearity, material nonlinearity, and pore water pressure development. For most soil mechanics problems, it is sufficient to use the Mohr–Coulomb failure criterion. In one dimension, the Mohr–Coulomb yield surface is linear and is defined by a line between shear stress, τ , and normal stress, σ , which is written as

$$f = \tau - (c - \sigma \cdot \tan \phi) = 0 \quad (1)$$

where the constant c and ϕ are cohesion and internal friction angle of the soil, respectively. In three dimensions, the Mohr–Coulomb yield criterion is expressed as

$$f = (\sigma_1 - \sigma_3) - (\sigma_1 + \sigma_3) \cdot \sin \phi - 2c \cdot \cos \phi = 0 \quad (2)$$

where σ_1 , σ_2 , σ_3 are principle stresses, and σ_1 and σ_3 are maximum and minimum principle stresses.

Important advantages of the Mohr–Coulomb failure criterion include its simplicity and the fact that it permits FE solutions to be compared with a wide variety of classical plasticity solutions.²⁵ The Mohr–Coulomb model is the best known model for an isotropic pressure-sensitive soil. However, this model is not mathematically convenient owing to the presence of corners or singularities. The surface becomes a point at the intersection with the stress axis (zero shear strength). This

type of singularity can cause problems in numerical computation. To avoid such angularity, a modified Mohr–Coulomb failure criterion as described in Ref. [24] was adopted in this material model.

Prediction of Free-Field Stresses

Ground shock propagation in soil is a complex function of the dynamic constitutive properties of the soil, the explosive products, and the geometry of the explosion.¹ TM5-855-1²² provides the following equations to predict the peak values of pressure, velocity, and acceleration, respectively.

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

$$V_0 = 160 \cdot f \cdot \left(\frac{R}{W^{1/3}} \right)^{-n} \quad (4)$$

$$a_0 = \frac{50 \cdot f \cdot c}{W^{1/3}} \cdot \left(\frac{R}{W^{1/3}} \right)^{(-n-1)} \quad (5)$$

In these equations, P_0 is the peak pressure in psi, V_0 is the peak particle velocity in ft/s (fps), a_0 is the peak acceleration in g (acceleration of gravity), f is a coupling factor dependent of the scaled depth of the explosion, ρc is the acoustic impedance in psi/fps, R is the 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 dependent on the soil type as shown in Table 2.²²

Comparison of Numerical Results for Free-Field Stresses with TM5-855-1 Predictions

As described earlier, an FE model was developed with the soil 10 m high,

and 500 kg TNT was detonated at the mid-depth of the soil. The results from the present numerical simulations for peak pressures obtained in each soil type are compared with the predicted pressures from the manual TM5-855-1²² to validate the soil material model for each soil type. To monitor the blast wave propagation in the soil mass, a group of target points is selected along the line horizontal to the explosive charge. The target points are located within the range 5 to 25 m from the detonation point. Figure 1 shows the pressure time histories of the compressive waves at those target points in soil type 1.

The peak pressures obtained in soil type 1 from the numerical simulation are compared with the peak pressures given by TM5-855-1²² as shown in Fig. 2. Peak pressure attenuation has been plotted against the scaled distance in Fig. 2. As soil properties in the TM5-855-1²² are given in a range for the soil type considered, Fig. 2 shows two straight lines representing the upper and the lower empirical limits of the peak pressure in soil type 1. It can be noted that the numerical results for the peak pressure are almost in between the upper and lower limits of the predicted peak values for this soil type. Predicted peak pressures close to the explosion are marginally higher than the numerical results.

Figures 3, 4, and 5 compare the peak pressures obtained from the numerical simulation with the predicted peak pressures using the TM5-855-1 for soil types 2, 3, and 4, respectively. As shown in these figures, the numerical results of the peak pressure attenuation agree reasonably well with empirical results. In addition, the results show that attenuation of the peak pressure in the soil occurs with increasing distance from the charge.

Figure 6 shows a comparison of the numerically obtained results of the peak pressure attenuations plotted against the scaled distance for all the

Soil types	Unit weight (kg/m ³)	Seismic velocity, c (m/s)	Acoustic impedance, ρc (MPa s/m)	Attenuation coefficient, n
Heavy saturated clays and clay shale	1920–2080	>1524	33,9–40,7	1,5
Saturated sandy clays and sands with air voids < 1%	1760–1984	1524	29,4	2,25–2,5
Dense sand with high relative density	1744	488	9,9	2,5
Wey sandy clay with air voids >4%	1920–2000	549	10,8	2,5
Sandy loam, loess, dry sands and backfills	1984	305	5	2,75
Loose, dry sands and gravels with low relative density	1440–1600	183	2,7	3–3,25

Table 2: Soil properties for calculating ground shock parameters²²

Fig. 1: Pressure time histories at different distances in the soil from the charge for soil type 1

Fig. 2: Relationship of peak pressures with scaled distance for soil type 1

soil types. It can be noted that the peak pressures in soil type 3 show smaller values. Soil type 1 has the highest peak

pressures. The small peak pressure in dry soil results from the slow wave velocity.

Modeling the System: Pile–Soil–Explosive–Air

Investigation of the response of the end bearing pile foundation to the ground shock caused by a buried explosion is the focus of this study. An FE model was developed for a 10 m long pile with 600 mm diameter circular cross section using the explicit dynamic non-linear FE software LS-DYNA.⁶ The overall model has different regions representing the soil, air, pile, and explosive charge as shown in Fig. 7. Taking advantage of symmetry, only a quarter of the system was modeled.

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 the soil close to the explosive were modeled with ALE multimaterial meshes. This is to prevent 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. 25 mm long beam elements with 2×2 Gauss integration were used for both the vertical reinforcements and the ties. The vertical reinforcements were defined as Hughes–Liu beam elements with cross integration and the ties were defined as truss elements.

Five kinds of materials were involved in this FE model such as air, explosive, soil, concrete, and steel. The air is commonly modeled using the null material model with a linear polynomial equation of state (EOS). The TNT explosive charge is modeled using the high explosive material model with the Jones–Wilkins–Lee (JWL) EOS. Material parameters and EOS constants for air and explosive are available in Ref. [23] and were used in the present simulations. FHWA material model was used to model the soil as described earlier.

At the bottom, (the mesh of) the model was constrained in all the directions to represent the bed rock. To form the symmetry in the FE model, the translational displacements of nodes normal to the symmetry planes were constrained. The nodes along the interfaces between the air and the soil were merged. Fixed boundary conditions were considered at the top and the bottom of the pile.

The interaction between the pile and the surrounding soil was modeled by specifying the Automatic_Surface_to_Surface contact option in LS-DYNA.

Fig. 3: Relationship of peak pressures with scaled distance for soil type 2

Fig. 4: Relationship of peak pressures with scaled distance for soil type 3

Fig. 5: Relationship of peak pressures with scaled distance for soil type 4

Fig. 6: Comparison of peak pressure attenuations

This assumes contact at the surface and enables transfer of stresses between the solid materials. Contact nonlinearity was established by assigning the viscous damping coefficient. In addition, static and dynamic friction coefficients were introduced to simulate the frictional forces that were transmitted across the contact interface. Thus, this contact method was used at the soil-pile interface to allow for separation in tension and to ensure compatibility in compression.

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 need to align the beam nodes to the solid element nodes.

Concrete Material Model

The response of the concrete under dynamic loading is a complex non-linear and rate-dependent process. Concrete compressive strength is increased by 200 to 300% at strain rates between 100/s and 1000/s.²⁶ Blast pressures normally produce high strain rates in the range of 100/s to 10 000/s. It is well known that the numerical results are very sensitive to the material properties, and thus the ability to define the material model accurately is one of the most important issues in the numerical simulation. LS-DYNA contains several material models that can be used to represent concrete. The material model `Concrete_Damage_REL3` was used for the concrete in this investigation. It is a plasticity-based model, using three shear failure surfaces and including damage and strain rate effects.²⁷ 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.

In order to account for the increase in strengths under high strain rates, a coefficient called the dynamic increase factor (DIF) is employed in this analysis. Dynamic increase factor

Fig. 7: Finite element model

is the ratio of the strength at a point of interest on the stress strain curve under high strain rate dynamic loading to the strength at the corresponding strain under static loading. The expressions proposed in Ref. [28] are utilized. The DIFs for the concrete compressive strength are given as

$$\text{DIF} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{1.026\alpha} \quad \text{for } \varepsilon \leq 30/s \quad (6)$$

$$\text{DIF} = \gamma \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{\frac{1}{3}} \quad \text{for } \varepsilon > 30/s \quad (7)$$

where $\dot{\varepsilon}$ is the strain rate in the range of 30×10^{-6} to $300/s$; $\dot{\varepsilon}_s$ is $30 \times 10^{-6}/s$; $\log \gamma = 6,156 \alpha - 2$; $\alpha = 1/(5+9f_c/f_{co})$; $f_{co} = 10$ MPa; and f_c is the static compressive strength of the concrete. The DIF for the concrete in tension is given by

$$\text{DIF} = \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{\delta} \quad \text{for } \dot{\varepsilon} \leq 1.0/s \quad (8)$$

$$\text{DIF} = \beta \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_s} \right)^{\frac{1}{3}} \quad \text{for } \dot{\varepsilon} > 1.0/s \quad (9)$$

where $\dot{\varepsilon}$ is the strain rate in the range of 1×10^{-6} to $160/s$; $\dot{\varepsilon}_s$ is $1 \times 10^{-6}/s$; $\log \beta = 6\delta - 2$; $\delta = 1/(1+8f_c/f_{co})$; $f_{co} = 10$ MPa; and 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.

The erosion algorithm was used to simulate the crushing of concrete in the FE model. When the material response in an element reaches a certain critical value, the element is immediately deleted. It gives great means to imitate concrete spalling phenomena and produce graphical plots that 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 $0,01$.⁸

Pile Reinforcement

Pile reinforcement is normally required to resist the bending and ten-

sile stresses, but may be used to carry a portion of the compression load. Sixteen bars of 20 mm diameter were used as the pile vertical reinforcement in this study. Steel bars of 10 mm diameter were used for the transverse reinforcements. A transverse reinforcement ratio of 0,24% was used in piles provided at a spacing of 200 mm. Beam elements with 2×2 Gauss integration were used to model both vertical and transverse reinforcements in the reinforced concrete pile in this analysis. 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. Strain rate was calculated using the Cowper-Symonds model given in Ref. [6].

$$\frac{\sigma'_d}{\sigma_s} = 1 + \left(\frac{\dot{\varepsilon}}{C} \right)^{1/P} \quad (10)$$

In this equation, σ'_d is the dynamic flow stress at a uniaxial 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 steel are listed in Table 3.

Blast Response of Pile in Different Soil Types

This study investigated the response and damage to the (10 m long) end bearing reinforced concrete (RC) single pile when subjected to blast loads for a standoff distance of 7,5 m in different soil types. As shown in Table 1, four soil types were considered: saturated soil, partially saturated soil, loose dry soil, and dense dry soil, labeled as soil types 1, 2, 3, and 4.

Figure 8 shows the time histories of the horizontal deformation of the pile embedded in the different soils. Pile deformations are presented at three heights from the pile tip: 2,5 m (point A), 5 m (point B), and 7,5 m (point C). This figure demonstrates

that the pile has residual deflection in all the cases. These residual deflections show the occurrence of the plastic deformation of the pile and indicate that the pile has suffered permanent deformation under the buried blast. For the parameters considered in this study, it can be noted that the pile embedded in soil type 3 has the highest pile deformation, and when it is embedded in soil type 2 it has the lowest pile deformation. The pile embedded in soil type 1 was found to have a maximum horizontal residual deflection of 369 mm, and a maximum lateral residual deflection of 247 mm was observed in soil type 2. Further, it was found that the piles embedded in soil types 3 and 4 have maximum residual deflections of 400 and 370 mm, respectively.

Damage to the reinforced concrete was also observed for the pile embedded in all soil types. Figure 9 depicts the concrete effective plastic strain variations of the piles with element erosion that were observed on the pile for a standoff distance of 7,5 m. Effective plastic strain is the damage parameter in the concrete_damage_rel3 material model, which ranges from 0 to 2. The elastic state of the concrete is represented by 0 in blue color and the yielding and post yielding are incorporated between 0 and 2. The residual capacity of the concrete is indicated by 2. As can be observed, it is clear that the pile was critically damaged in all the cases. Concrete elements were eroded at the top end of the pile in all the cases. This indicates that the concrete at the top end of the pile was totally destroyed in all the cases. Reinforcements were found to have been severely deformed at the top end. Figure 9(a) shows that the concrete at the bottom end was also severely damaged in the pile embedded in soil type 1. It can be noted that the pile embedded in soil type 1 suffered the most damage compared to the other two piles. However, the concrete in the middle of the pile suffered the most damage in the pile embedded in soil type 3.

From the above results for pile deformations and pile damage, it can be concluded that under the same buried explosion, piles embedded in soil type 1 or soil type 3 suffer more damage than piles embedded in soil type 2 and soil type 4. As shown in Fig. 6, blast wave pressures are high in soil type 1, and this could be the reason for the severe damage in the

	Density (kg/m ³)	Young's modulus (GPa)	Poisson's ratio	Yield stress (MPa)	Tangent modulus (GPa)	Hardening parameter (β)	C	P
Vertical R/F	7800	210	0,3	548	2	0	40	5
Ties	7800	210	0,3	350	2	0	40	5

Table 3: Material model parameters for main reinforcement and ties²⁹

Fig. 8: Pile deformation in (a) soil type 1 (b) soil type 2 (c) soil type 3 (d) soil type 4

Fig. 9: Pile damage in (a) soil type 1 (b) soil type 2 (c) soil type 3 (d) soil type 4

embedded pile. Even though blast wave pressures are lower in soil type 3 as shown in Fig. 6, the displacement of the soil could be high because of the poor bond between the soil particles. This could therefore be the reason for the severe deformation of the pile embedded in soil type 3 under the buried explosion.

Fig. 10: Pile deformation for standoff distance of 10 m in (a) soil type 1 (b) soil type 2 (c) soil type 3 (d) soil type 4

Fig. 11: Pile damage for standoff distance of 10 m in (a) soil type 1 (b) soil type 2 (c) soil type-3 (d) soil type 4

The Effect of the Standoff Distance

As described in section “Ground Motion and Free-Field Stresses”, blast wave pressures in the soil decrease with increasing distance from the charge. Thus, using the proposed numerical method, further studies were carried out to investigate the effect of standoff distance on the blast response of

piles embedded in different soil types. In this section, pile deformation and damage are presented for the standoff distances of 10 and 15 m from the explosive.

In Fig. 10, the time histories of the horizontal deformations of the pile at three heights from the pile tip: 2,5 m (point A), 5 m (point B) and 7,5 m (point C) are presented for the standoff distance of 10 m from the explosion.

It also demonstrates that the piles have suffered permanent deformation under the buried blast. It can be noted that the pile embedded in soil type 3 has the highest pile deformation and when it is embedded in soil type 2, it has the lowest pile deformation. The pile embedded in soil type 1 was found to have a maximum horizontal residual deflection of 165 mm, and a maximum lateral residual deflection

Fig. 12: Pile damage for standoff distance of 15 m in (a) soil type 1 (b) soil type 2 (c) soil type 3 (d) soil type 4

of 157 mm was observed in soil type 2. In addition, it was found that the piles embedded in soil types 3 and 4 had deflected by 280 and 270 mm, respectively. Figure 11 shows the concrete effective plastic strain variation of the pile with the element erosion that was observed on the pile for a standoff distance of 10 m. It is clear that piles were critically damaged in all the cases as in the standoff distance of 7,5 m. As expected, pile damages and deformations decreased. In this case also, piles embedded in soil type 1 and soil type 3 suffered more damage than piles embedded in soil types 2 and 4. However, the deformed shape of the pile embedded in soil type 3 is different from that in the previous case (standoff distance of 7,5 m).

Figure 12 shows the concrete effective plastic strain variations of the piles with the element erosion that were observed on the piles for a standoff distance of 15 m. In this case also, the concrete in the top and bottom ends of the pile embedded in soil type 1 were totally destroyed. A maximum lateral deformation of 85 mm was found in the pile. Spalling was also observed at the top ends of the piles embedded in soil types 2, 3, and 4. The pile embedded in soil type 2 was found to have a maximum horizontal residual deflection of 54 mm, and it was found that the piles embedded in soil types 3 and 4 had deflected by 80 and 75 mm, respectively. Although the pile embedded in soil type 3 had larger deformations than the other three cases for

the standoff distances of 7,5m and 10 m, the pile embedded in soil type 1 deformed more for the standoff distance of 15 m.

Conclusion

A coupled numerical model was used to study the dynamic response of reinforced concrete pile foundation to a buried explosion using the commercial computer program LS-DYNA. This study investigated the blast response of a single pile embedded in several soil types: saturated soil, partially saturated soil, loose dry soil, and dense dry soil. The soil was modeled using the FHWA soil material model, and blast wave propagation in soils was validated with the predicted pressures using the TM5-855-1. Horizontal pile deformation and damages on the pile were obtained from the numerical simulations. On the basis of the parameters considered in the study and the presented results, the following main conclusions can be drawn.

1. Performance of the piles embedded in saturated soil and loose dry soil is more severe than that of the pile embedded in partially saturated soil and dense dry soil when subjected to the same buried explosion.
2. As blast wave pressures are high in saturated soil, they cause severe damage in the pile. Even though blast wave pressures are low in loose dry soil, the displacement of the soils might be high because of the poor bond between soil particles. This

might therefore be the reason for the severe deformation of the pile embedded in loose dry soil under a buried explosion.

3. Pile damages and deformations decrease with distance from the explosive.
4. For scaled distances 1 and 1,3 m/kg^{1/3}, the pile embedded in loose dry soil has the maximum pile deformation.
5. For a scaled distance of 1,9 m/kg^{1/3}, the pile embedded in saturated soil has the maximum pile deformation.
6. The modeling techniques developed in this study and the outcomes of the study improve our understanding in this area and could be useful in future studies.

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