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2014

Jayasinghe, L. B., Thambiratnam, D. P., Perera, N., & Jayasooriya, J. H. A. R. (2014). Blast response of reinforced concrete pile using fully coupled computer simulation techniques. *Computers and Structures*, 135, 40-49.

<https://hdl.handle.net/10356/81532>

<https://doi.org/10.1016/j.compstruc.2014.01.017>

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Computer Analysis of Blast Response of Reinforced Concrete Pile

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ABSTRACT

Pile foundations are key elements for transferring loads from structures to stronger sub soil layers through the weaker upper soil layers. The strength and stability of pile foundation will hence affect the safe performance of structures. This paper treats the response of a reinforced concrete pile to a buried explosion using computer simulations. The soil is assumed to be saturated and homogeneous sandy. Horizontal deformation and tensile stress in the pile are evaluated and the influence of the reinforcement on the pile response is investigated. The findings of this study can serve as a reference for future analysis and design.

Key words: underground explosion; pile foundation; reinforced concrete; numerical simulation

1. INTRODUCTION

Many countries across the world have encountered a significant increase in terrorist bomb attacks over the past two or three decades. These bombs are especially targeted at significant and iconic buildings either by indoor and outdoor explosions. The explosion in the World Trade Center in New York on February 26, 1993 is a well-known example of a powerful bomb explosion which occurred inside a building. There has also been an increase in car or truck bomb explosions in the vicinity of buildings during the recent past. This scenario could result in a very large charge weight 5 to 10 tons TNT equivalency from large vehicles parked very close to a building. Alfred P. Murrah Federal Building bombing incident in Oklahoma City on April 19, 1995 is one of the largest terrorist attacks caused by a truck bomb which detonated outside the building. Such increasing terrorist attacks have led researchers to investigate the vulnerability of structures to random and unexpected loads such as impacts and blasts.

Reinforced concrete is the prime structural material widely used for the construction of protective structures as well as significant buildings. As a result of very rapid dynamic loading of structures during a blast loading, structural materials are subjected to very high strain rates. High strain rates cause enhancement in both tensile and compressive strength of the structural materials as well as change in the post yield behavior. According to Bischoff and Perry [1] the design compressive strength of the concrete can increase by about 25 to 30 percent during dynamic loading of the concrete. However, based on experiments conducted by Ross et al.[2], the concrete compressive strength enhancement was between 200 to 300 percent at strain rates between 100 to 1000 s^{-1} . Blast pressures normally produce high strain rates in the range of 100 to 10000 s^{-1} . In order to design structures to withstand blast loading, it is important to understand the response of structural elements when subjected to large stress and strain rates through explosive loadings. Research has been carried out on the blast response of structures by considering the dynamic nonlinear responses of individual structural components such as beams, slabs and columns.

Lan et al.[3] and Bao and Li [4] carried out numerical simulations using the non-linear explicit Finite Element (FE) code LS-DYNA [5] to study the dynamic response of reinforced concrete columns subjected to blast loads. Bao and Li [4] verified the numerical model through experimental studies and proposed a formula to estimate the residual axial capacity based on the mid height displacement. Xu et al. [6] presented a numerical study on the concrete spallation in reinforced concrete slabs under various blast loading conditions. Tai et al. [7] also conducted a numerical simulation study on the blast response of reinforced concrete slabs and investigated the influence of reinforcement ratio, explosive weight and standoff distance. Jayasooriya et.al [8] proposed a method to assess the vulnerability and residual capacity of building frames and components when subjected to near field blast events.

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 due to blast loading, the superstructure becomes vulnerable to failure. Some studies on laterally loaded piles can be found in the literature. Poulos [9] analyzed the behavior of laterally loaded piles using the

continuum theory. It was found that the major factors influencing the pile behavior are the pile flexibility and the length to diameter ratio, for both fixed-head and free-head piles. Budhu and Davies [10] presented a numerical analysis of single laterally loaded piles embedded in cohesion-less soil which was modeled as an elastic material. Randolph [11] studied the response of flexible pile to lateral loading using numerical simulation. He treated the soil as an elastic continuum with a linearly varying soil modulus and developed a formula to determine the maximum bending moment induced in a free-headed pile.

This paper treats the response of reinforced concrete (RC) 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. The modeling techniques used in the present study have been validated against experimental results. A brief description of the background on modeling is presented in the early part of this paper. Response of the pile to an underground explosion is then presented. Reinforcement is required in concrete piles to resist bending and tensile stresses, but it may also be used to carry a portion of the compression load. Pile reinforcement was considered as a parameter to study its effect on the dynamic response of the RC pile subjected to blast loads.

2. EXPERIMENTAL SET UP & VALIDATION OF MODELLING TECHNIQUE

In a previous paper, the authors [12] validated the modeling techniques and the analysis procedure used to study the blast response of a pile by comparing their results from numerical simulations with those from the experimental results in reference [13]. This validation procedure will be briefly discussed below.

Experimental data from the centrifuge model tests of Shim [13] were used in the validation process. The centrifuge tests were conducted at the 440 g-ton centrifuge facility located at the University of Colorado, Boulder. Shim carried out a series of 70-g centrifuge tests to investigate the blast wave propagation and response of piles embedded in saturated sand. 14.3cm long Aluminum tubes were used as model piles throughout the tests. Two different boundary conditions at the top of the model piles were employed in the tests. One was the fixed boundary conditions and the other was the free boundary condition with an axial load [13]. Cylindrical shape model explosives were placed at the mid-depth of soil in the centrifuge tests. Three different standoff distances from the explosive to the piles were

considered in the study. Figure 1 shows a schematic diagram of the set up with the pile fixed at the top [12]. Detailed description of the experiments can be found in Shim [13].

Figure 1. Configuration of a set-up [12]

In their previous paper, the authors [12] developed a 3D finite element (FE) model by considering prototype dimensions in Shim's [13] study to validate their modeling techniques and analysis procedure. Blast wave propagation in the soil and the horizontal pile deformation were studied in their study and the results were compared with the experimental results in [13]. Figure 2(a) shows the comparison of their blast wave pressures in the soil with Shim's [13] results. Figure 2(b) plots horizontal residual deformations of the pile along its height for different three standoff distances obtained from the authors; study and those from reference [13]. From these Figures, it is clearly evident that the modeling techniques used by the authors are adequately validated. A detailed description of the modeling techniques can be found in Jayasinghe et al. [12].

Figure 2. Validation of modeling technique (a) Blast wave propagation in the soil [12]
(b) Residual horizontal pile deformation [12]

3. NUMERICAL SIMULATION

The present investigates the blast response of RC pile using dynamic computer simulation technique. The numerical models must have the capability to model the detonation of the explosives, blast wave propagation through ground, interaction of the blast wave with the pile and the pile response. The finite element modeling code LS-DYNA was used for the computer simulation to meet these requirements. LS-DYNA is mainly based on an explicit time integration scheme, and implicit solution has been added gradually in recent years. In the LS-DYNA explicit analysis, the equation of motion is integrated in time by using the central differences method, which requires very small time steps to ensure a stable solution. The LS-DYNA explicit code is thus particularly suitable for impact, crash and blast simulations. Furthermore, the performance of LS-DYNA has been verified by numerous users for the simulations of blast loading on reinforced concrete structures [8, 14].

3.1. FE model

The modeling techniques used in the present study are the same as in Jayasinghe et al. [12]. The FE model was developed for a RC pile to evaluate its response for a buried explosion. A 10m length pile with hollow circular cross section was considered. The RC pile section was selected to have the same elastic bending stiffness as the Aluminum pile is used in [12] and is shown in Figure 3. The clear concrete cover to the ties is taken as 25mm, as shown in Figure 3(a). Figure 3(b) shows the mesh discretization for the concrete elements and reinforcing cage used in this study. Considering the symmetry of the set-up the FE model involved a quarter of the air domain, soil domains and explosive and half of the pile, with appropriate boundary conditions at the symmetry edges. Figure 4 shows the complete 3D FE model which includes air, soil, explosive and pile.

Eight noded hexagonal solid elements were used for all parts except reinforcing cage. 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.

Figure 3. (a) Pile cross-section (b) Concrete solid elements and Reinforcing beam elements

Figure 4. Finite element model

3.2. Material models

A wide variety of materials were modeled, including high explosives, air, soil, and the RC pile made of concrete and steel. The material models used in the study are described below.

3.2.1. Explosives

H6 explosive was considered in the FE models and it 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, P , as a function of the relative volume, V and initial energy per volume, E , such that [5]

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

In the above equation, A , B , R_1 , R_2 and ω are constants pertaining to the explosive. Material parameters and JWL constants for H6 explosive are available in [15] and were used in the present simulations.

3.2.2. Air

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 , as given by [5]

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

In the above equation, C_0 , C_1 , C_2 , C_3 , C_4 , C_5 , and C_6 are constants and $\mu = \frac{\rho}{\rho_0} - 1$, where

$\frac{\rho}{\rho_0}$ is the ratio of current and initial densities. Material parameters and EOS constants for air available in [12] were used in these simulations.

3.2.3. Soil

Fully saturated sand was considered in this study. It 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 for use. 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 computation. To avoid such a angularity, a modified Mohr-Coulomb failure criterion was adopted in this material model [16]. Material parameters available in [12] and were used in the present simulations.

3.2.5. Concrete

LS-DYNA contains several material models that can be used to represent concrete. The Winfrith concrete model, also called the smeared crack model, was used in this investigation. This material model was originally developed for the nuclear industry for use in a FE analysis

to predict the local and global response of RC structures subjected to explosive and impact loadings [17]. It has the ability to provide details on cracks and their propagation.

The shear failure surface proposed by Ottosen [18] is used in the plasticity portion of the Winfrith concrete model. Failure surface of this material model is given by [17]

$$\frac{AJ_2}{(f'_c)^2} + \frac{\lambda\sqrt{J_2}}{f'_c} + \frac{BI_1}{f'_c} - 1 = 0 \quad \text{Eq. 1}$$

where,

$$\lambda = K_1 \cos\left[-\frac{1}{3} \cos^{-1}(K_2 \cos 3\theta)\right] \quad \text{for } \cos 3\theta \geq 0 \quad \text{Eq. 2}$$

$$\lambda = K_1 \cos\left[\frac{\pi}{3} - \frac{1}{3} \cos^{-1}(-K_2 \cos 3\theta)\right] \quad \text{for } \cos 3\theta < 0 \quad \text{Eq. 3}$$

$$\cos 3\theta = \frac{\sqrt{3}}{2J_2^{1.5}} J_3 \quad \text{Eq. 4}$$

I_1 is the first stress invariant

J_2 and J_3 are the deviatoric stress tensors

θ is the lode angle

A, B, K_1 and K_2 are called the shape parameters

The constants A and B control the meridional shape of the shear failure surface and the constants K_1 and K_2 define the shape of the shear failure surface in the octahedral plane. The required input parameters for the Winfrith concrete material model were provided via empirical formulae as in Equation 5 and 6.

$$\text{Initial tangent Modulus, } TM = 57000\sqrt{f'_c} \quad (\text{psi}) \quad \text{Eq. 5}$$

$$\text{Uniaxial tensile strength, } UTS = 7\sqrt{f'_c} \quad (\text{psi}) \quad \text{Eq. 6}$$

Where, f'_c is the unconfined compressive strength of the concrete in psi.

The crack width formulation used in the Winfrith concrete model is based on the studies of Wittmann et al. [19]. They studied the specific fracture energy, for a large number of concrete samples with varying compressive strengths, aggregate size, etc. Thus, the specific fracture energy was taken from their results and used.

3.2.6. Steel

Vertical reinforcement and ties in reinforced concrete pile were modeled as elastic perfectly-plastic materials using the plastic kinematic model available in LS-DYNA. Kinematic hardening with strain rate effects was implemented for the reinforcement. Strain rate is accounted for by using the Cowper-Symonds model given by [5]

$$\frac{\sigma'_d}{\sigma_s} = 1 + \left(\frac{\dot{\varepsilon}}{C} \right)^{1/P} \quad \text{Eq. 3}$$

In the above equation, 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 material constants. Material model parameters for steel are listed in Table 1.

Table 1. Material model parameters for main reinforcement and ties [20]

3.3. Boundary conditions and Load application

The bottom of the mesh 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 symmetry planes were constrained. The nodes along the interfaces between the air and soil were merged. Fixed boundary conditions were considered in the top and bottom of pile. LS-DYNA provides different ways to apply the blast load to a model. However, the present study adopts the fully coupled numerical simulation approach.

The simulations were conducted in two steps. First, stress initialization was allowed to induce steady pre-stress in the model under gravity loads. Due to this dynamic relaxation, stresses in the soil and pile act as initial conditions for the blast analysis. Then, the explosive was initiated just after the dynamic relaxation phase.

3.4. Constrained Lagrange in solid formulation

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 of 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.

3.5. Validation for modeling reinforced concrete

As mentioned earlier, the modeling techniques used in this study have been successfully validated by comparing the dynamic response of a pile foundation to a buried explosion [12] with the experimental results in [13]. Further validation of RC material model is carried out in this part of the paper. The experimental investigations (experiment no.2) carried out by Woodson and Baylot [21] were used for this purpose. A series of experiments with five different two-story, quarter-scale RC structures have been conducted by Woodson and Baylot [21] to investigate the response and damage of the exterior columns when subjected to blast loads. A detailed description of the experimental program can be found in Woodson and Baylot [21].

A 3D FE model for the first floor center column in experiment no. 2 was developed for the comparison of results as shown in Figure 5(a). The cross section of the column was 89mm x 89mm square with 8 number of deformed steel wires each having a cross sectional area of 32.2mm^2 . Steel wires each having a cross sectional area of 3.22mm^2 were provided at 100 mm spacing as cross ties. 7.10 kg of C4 explosive placed 0.2286m above the ground at a standoff distance of 1.07m was considered.

Figure 5. (a) Validation model (b) Comparison of the mid-height deflection

Figure 5(b) shows the comparison of the experimental and numerical results for the deflection-time histories at the mid-height of the column. The peak deflection obtained from the present numerical analysis is marginally less than the experimental result. The residual deflections are almost same in both the present analysis and the experimental results. This indicates that the present numerical simulation results agree well with the results from the experiment in [21]. This result and the validation discussed in section 2, provide confidence in the choice of parameters and the analysis used in this study.

4. RESULTS AND DISCUSSION

This study investigated the response and damage of a RC pile when subjected to buried blast loads located at the mid-height of the pile and at a standoff distance of 7.5m. Pile deformation and crack propagation were studied. The horizontal deformation of pile was obtained at 7 monitoring points on the pile as shown in Figure 6. Analysis of the FE model was carried out for 1.5 seconds duration. Using the High Performance Computing facilities at the Queensland University of Technology, simulation took 96 hours to solve when using four parallel processors.

Figure 6. Monitoring points on the pile [12]

Figure 7 shows the time histories of the horizontal deformation of the pile at the 7 monitoring points. It can be observed that the pile has suffered permanent deformation under the buried blast and the maximum residual deformation of 382mm occurs at the monitoring point E located 6m above from the pile tip (Figure 6). The shape of this RC pile deformation is similar to the deformation shape in the Aluminum pile which was investigated earlier in Jayasinghe et al. [12]. Figure 8 compares the residual horizontal deformations of the RC pile along its height obtained from the present analysis, for the 7.5m standoff distance, with the corresponding values from the previous investigation [12]. It is evident that the deflections in the RC pile are greater than those in the Aluminum pile, even-though both piles had the same elastic bending stiffness. Moreover, there are residual deflections in the present RC pile indicating plastic deformations under the effect of the blast loads. The residual deflections experienced in the Aluminium pile treated earlier were much smaller. These findings

emphasizes that results from model testing of a pile made with a certain material cannot always be extended to a prototype pile made with a different material. Shim [13] used an Aluminium tube instead of a RC pile in his centrifuge tests. The results of his tests therefore cannot be used to predict the deflections in a prototype RC pile if there are plastic deformations in the pile response.

Figure 7. Pile deformation

Figure 8. Comparison of Horizontal deformation of pile

This FE model took 430ms for the stress initialization in the first phase of the analysis. Detonation of the explosive was initiated just after the stress initialization. Figure 9 depicts the cracks that were observed in the pile (for the stand-off distance of 7.5m) at different time instances. The numerical simulation showed that cracks first start appearing at the mid-height of the pile as can be seen in Figure 9(a); then, at the ends of the pile and propagate along the pile. When a concrete element fails under a tensile stress, a crack is flagged in a plane normal to the maximum principle stress. Figure 9(c) demonstrates that the pile ends have suffered significant amount of cracking, with the top enduring more severe cracking. Maximum crack width observed in the pile was 4.26mm. Based on these observations, this pile is liable to fail from its top end.

Figure 9. Cracks in the pile at (a) 438ms (b) 441ms (c) 1.5s

5. PARAMETRIC STUDY

Using the FE model discussed in section 3 above, numerical simulations were performed to evaluate the dynamic response of the RC pile with a circular solid section. When lateral forces are applied to a pile, bending moments develop in the pile. Where these moments exceed the design bending resistance of the pile, reinforcement is required to resist the tensile

stresses. A parametric study was carried out for the following cases illustrated in Figure 10. Transverse reinforcement ratio ρ_v of 0.24% was used in Piles A, C, E provided at a nominal spacing of 200 mm. For piles B, D, and F, spacing of the transverse reinforcement was determined in accordance with the requirement in BS 8004 [22]. This Standard recommends that the minimum lateral reinforcement should be 0.2% of the gross volume of the pile in the body of the pile, and 0.6% of the gross volume of the pile at each end, distributed across a length of about three times the smaller dimension of the pile cross section. Thus for piles B, D and F, lateral reinforcement of 10 mm diameter bars were spaced at 150 mm ($\rho_v = 0.30\%$) in the body of the pile and at 65 mm ($\rho_v = 0.64\%$) at each end of the pile. Figure 10 and Table 2 provide the details of the pile reinforcement. The blast responses of the piles for 500 kg TNT situated at the mid depth of the soil were determined. The material constants and EOS parameters for the TNT explosive available in [23] were used in the present study.

Figure 10. Pile geometry and reinforcement details

Table 2. Longitudinal and transverse reinforcements of the pile.

Figure 11 shows the residual horizontal deformations of the RC piles along their heights. Responses of piles A, C and E almost coincide with the responses of piles B, D and F. The pairs of piles with matching residual horizontal deflections have the same longitudinal reinforcement, which has a significant influence. It evident that pile deflections decrease with increase in the longitudinal reinforcement. The results in Figure 11 also show that the transverse reinforcement of the pile does not have much effect on the pile deflections. Under the same blast loads, cracks in the piles with ties at nominal spacing are greater than those in the piles with ties provided in accordance with BS 8004 [22], as shown in the Figure 12. The transverse reinforcement increases the shear capacity of the pile, and also it provides confinement to the core concrete and lateral restraint against buckling of the vertical reinforcement.

Figure 11. Pile deformations

Figure 12. Cracks in the piles

LS-DYNA [5] has the capability of providing various information related to cracks such as crack widths, location, direction and number of cracking elements for the Winfrith concrete material model. The number of cracked elements and maximum crack width were obtained for each pile from the numerical simulation as shown in Table 3. It shows that the maximum crack width and also number of cracked elements decrease with increase in the longitudinal and transverse reinforcements. It can also be seen that the maximum crack width in the piles is greater than the allowable crack width (of 0.3mm) provided in the reinforced concrete design codes. It may therefore be considered that the piles have failed under this blast load.

Table 3. Number of cracked elements and Maximum crack widths

6. CONCLUSION

A coupled numerical model was used to study the dynamic response of reinforced concrete pile foundation to a buried explosion using the commercial finite element computer program LS-DYNA. Modeling techniques had been validated in a previous paper by comparing the numerical results with those from an experimental study. Due to its ability to provide details on cracks and their propagation, the Winfrith concrete material model was chosen to model the concrete, and it was validated with experimental results in reference [21]. Based on the results, the following main conclusions can be drawn.

1. It is not possible to accurately predict the response of a prototype structure (reinforced concrete) from centrifuge test results when different materials are used in the model and the prototype, both of which were designed to have the same elastic bending stiffness.
2. Longitudinal reinforcement in a pile has a significant effect on pile deformations under blast loads. These deformations decrease with increase in the longitudinal reinforcement.
3. Proper detailing of ties in a pile can cause significant reductions in the degree of direct damage under blast loads.
4. Overall, cracks in the pile decrease with the increase in both the longitudinal and transverse reinforcements.

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Figure 1. Configuration of set-up [12]

(a)

(b)

Figure 2. Validation of modeling technique (a) Blast wave propagation in the soil [12]
(b) Residual horizontal pile deformation [12]

Figure 3. (a) Pile cross-section (b) Concrete solid elements and Reinforcing beam elements

Figure 4. Finite element model

Figure 5. (a) Validation model (b) Comparison of the mid-height deflection

Figure 6. Monitoring points on the pile [12]

Figure 7. Pile deformation

Figure 8. Comparison of Horizontal deformation of pile

Figure 9. Cracks in the pile at (a) 438ms (b) 441ms (c) 1.5s

Figure 10. Pile geometry and reinforcement details

Figure 11. Piles deformations

Pile A

Pile B

Pile C

Pile D

Pile E

Pile F

Figure 12. Cracks in the piles

TABLES

Table 1. Material model parameters for main reinforcement and ties [20]

	Density (kg/m ³)	Young's modulus (Gpa)	Poission'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 2. Longitudinal and transverse reinforcements of the pile

Pile type	Longitudinal steel	Transverse steel	
		at middle	at ends
A	16T25	T10-200mm ($\rho_v = 0.24\%$)	T10-200mm ($\rho_v = 0.24\%$)
B	16T25	T10-150mm ($\rho_v = 0.30\%$)	T10-65mm ($\rho_v = 0.64\%$)
C	16T20	T10-200mm ($\rho_v = 0.24\%$)	T10-200mm ($\rho_v = 0.24\%$)
D	16T20	T10-150mm ($\rho_v = 0.30\%$)	T10-65mm ($\rho_v = 0.64\%$)
E	16T16	T10-200mm ($\rho_v = 0.24\%$)	T10-200mm ($\rho_v = 0.24\%$)
F	16T16	T10-150mm ($\rho_v = 0.30\%$)	T10-65mm ($\rho_v = 0.64\%$)

Table 3. Number of cracked elements and Maximum crack widths

	Pile A	Pile B	Pile C	Pile D	Pile E	Pile F
Number of cracked elements	1900	1482	1910	1588	2172	1899
Maximum crack width (mm)	3.31	2.83	3.15	3.22	3.56	3.18