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Title:
Performance evaluation of a shallow foundation built on structured clays under seismic loading

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Abstract

Due to the increased need of storage, larger and higher structures are being built all over the world, thus requiring a more careful evaluation of the mechanical performance of their foundation deposits both in terms of bearing capacity and compressibility behaviour. The design of such structures and their serviceability and stability is largely governed by the effects of the dynamic loading conditions principally because of their significantly elevated risk in seismic prone zones. In this paper, numerical analyses using an advanced constitutive model, able to account for the initial soil structure and its progressive degradation, have been performed to investigate the seismic response of a silo foundation built on structured clays. The proposed analyses involve the use of a fully-coupled finite element approach. For the dynamic simulations, three different input motions have been selected from earthquake databases according to the seismic hazard study of the specific site. The results of the silo dynamic response are illustrated in terms of signal amplification, permanent excess pore water pressures, accumulated displacements and structure induced degradation during and after the seismic loading. The dynamic behaviour of the footing indicates that extreme earthquake events can induce large destructuration in natural clays, leading to ground settlements up to twice the observed ones under static loads, which need to be properly accounted for in the design. This suggests that there are significant advantages in using advanced models which recognise the existence of initial soil structure and its subsequent damage due to the applied dynamic loads.

Keywords

Shallow foundation; Seismic response; Natural structured clays; Destructuration; Effective stress dynamic analysis
Introduction

Tanks and silos are light structures carrying relatively high loads applied in monotonic and (low frequency) cyclic conditions. These structures are usually supported by shallow foundations founded on poor quality sub-soils. Large settlements can be observed upon the first monotonic (pre)loading (e.g. Bjerrum and Overland 1957; Burland 1969; Ricceri 1974; Penman 1978, Muir Wood 1980) and further settlements are due to loading/unloading cycles related to stocking activities, leading, sometimes, to a delayed collapse (e.g. Bozozuk 1979; Bell and Iwakiri 1980; Dogangun et al. 2009). Due to the increased need of storage, larger and higher structures are being built all over the world, thus requiring a more careful evaluation of the mechanical performance of their foundation deposits, both in terms of bearing capacity and compressibility behaviour (e.g. Rampello and Callisto 2003). Moreover, such structures can be located in seismic-prone zones and be subjected to proper dynamic (high frequency) loads during earthquakes (e.g. de Sanctis and Russo 2008; Dogangun et al. 2009). Therefore, their design necessarily requires the analysis of the effects of the dynamic loading conditions on their stability and serviceability (e.g. Sezen et al. 2008; Livaoglu 2008; Sekhar et al. 2009; Kianoush and Ghaemmaghami 2011; Mosleini and Kianoush 2012).

In this context, it can be relevant to account for the essential features of the mechanical behaviour of soils when subjected to cyclic loading: state dependency, early irreversibly, nonlinearity, build-up of excess pore water pressures, evolution of microstructure (de-structuring) and related decrease of nominal stiffness (e.g. Sangrey et al. 1969; Castro and Christian 1976; Leroueil and Vaughan 1990; Vucetic and Dobry 1991; Cotecchia and Chandler 1997). These aspects of soil behaviour can be described by adopting advanced constitutive models, developed by adding complexity to the classical single surface Cam-Clay model (e.g. Dafalias and Popov 1975; Pastor et al. 1990; Gens and Nova 1993; Asaoka et al. 2000; Liu and Carter 2002).
In this paper, the cyclic/dynamic behaviour of the shallow foundation of a sugar silo, founded on a cemented clayey soil deposit and subjected to loading/unloading cycles and earthquake actions, is studied through 2D fully-coupled finite element (FE) analyses. The stress-strain behaviour of the foundation soils is modelled using the elastic-perfectly plastic Mohr-Coulomb law for the silty sand layers, while the clayey silt layers are described by the advanced constitutive model developed within the framework of kinematic hardening and bounding surface plasticity by Rouainia and Muir Wood (2000) and named RMW in the following. Their calibration is based on the available experimental data obtained from the geotechnical investigation of the site (D’Elia et al. 1999; Burghignoli et al. 1999). Initially, the first load and unload stages of the silo foundation are simulated and the results in terms of computed settlements are compared with the experimental data. For the dynamic modelling purpose, three different input motions are selected from earthquake databases and scaled to two peak accelerations associated with collapse and damage limit state, respectively. The accelerograms are characterised by a broad variety of mean frequency, Arias intensity and duration values, being therefore representative of a broad range of possible events occurring at the site. The dynamic response of the silo during the earthquake motions is analysed and the results are illustrated in terms of signal amplification, permanent excess pore water pressures, accumulated displacements and progressive loss of structure during and after the seismic loading.
The case study

The sugar silo studied in this paper is located close to the city of Avezzano (AQ), Italy, about 100 km east of Rome. It is a cylindrical reinforced concrete structure with a diameter of 26.5 m and a height of 40 m. The silo is founded on a 34 m wide, 3 m thick cellular raft foundation placed 4 m below the ground level (Figure 1).

An extensive geotechnical characterisation of the area, including a large number of in-situ and laboratory tests, was carried out in the 90’s. The main results from the site investigation are described in detail by D’Elia et al. (1999) and Burghignoli et al. (1999). The physical and mechanical soil properties obtained from in-situ and laboratory tests are reported in Figure 2. The results are presented in terms of cone penetration profile, maximum shear modulus from dynamic measurements, soil unit weight, calcium carbonate (CaCO$_3$) content and compression and unloading indexes obtained from one-dimensional oedometer tests. The typical soil profile of the site consists of two silty sand layers (named L$_1$ and L$_4$), at the top and the bottom of the foundation deposit, and two intermediate normally consolidated clayey strata (named L$_2$ and L$_3$), characterised by high values of calcium carbonate content ranging between 60 and 80% (Figure 2d). Standard oedometer and undrained triaxial compression tests clearly indicate that both the clayey layers are characterised by the typical mechanical behaviour of cemented soft clays, as discussed later. The water table is located 5 m below ground level.

The vertical displacements of the raft foundation and the average applied pressure were recorded during the construction of the silo and the first three loading and unloading cycles related to sugar stocking activities, as reported in Figures 3a and 3b. During the first loading stage, the recorded settlements are very small until an average pressure of about 140 kPa is reached. The gradient of the load-settlement curve then rapidly increases. The unloading stage clearly highlights the irreversible nature of the settlements experienced by the footing during the first loading. In the second loading
stage, irreversible strains seem to develop when the applied pressure exceeds its past maximum value. Due to their relatively high compressibility, the two clayey layers play a crucial role in the settlement response of the silo to the applied loads. Previous numerical analyses of the same case study (D’Elia et al. 1999; Burghignoli et al. 2003) have already demonstrated the importance of using advanced constitutive models to properly describe the mechanical behaviour of the Avezzano cemented clayey soils. Simpler constitutive hypotheses, such as the Modified Cam-Clay (MCC) model (Roscoe and Burland 1968), were found to be unable to successfully capture the main features exhibited by the measured load-settlement curve of the sugar silo under working loads, as shown in Figure 3c. The stress changes in the foundation soil were completely enclosed within the yielding surface and an almost elastic response was predicted using MCC for the two clay layers (Burghignoli et al. 2003). On the contrary, a reasonable agreement between the numerical predictions and measured performance of the silo was obtained using the RMW model which accounts for the effects of structure degradation, as reported in Figure 3c. The advanced model, which allows for a continuous decay of plastic modulus along stress paths inside the bounding surface, reproduced correctly the plastic nature of the strain developed during loading and reloading cycles (Burghignoli et al. 2003).
Seismic hazard analysis and selection of input motions

According to the relevant European and Italian code prescriptions (EN 1998-1 2004; M. LL. PP. 2008), different levels of seismic action need to be adopted depending on the investigated limit state of the structure. In particular, the Italian code prescribes a probability of exceedance of the seismic action, $P$, for the collapse limit state (CLS) equal to 5% and a probability of 63% for the damage limit state (DLS). Therefore, assuming a standard reference life of 50 years for the silo, the return periods of the seismic action, $T_R$, associated with the CLS and the DLS are 975 years and 50 years, respectively. The corresponding peak ground accelerations ($a_{\text{max}}$) predicted at the site according to the seismic hazard study carried out by the Italian National Institute of Geophysics and Volcanology (INGV) on the entire national territory (Gruppo di lavoro MPS 2004) are reported in Table 1. In particular, for a return period of 975 years, associated with the CLS, the seismic hazard analysis predicts a maximum acceleration of 0.313 g while for the return period associated with the DLS the predicted peak ground acceleration is 0.097 g.

Three real Italian accelerograms recorded on bedrock (Eurocode 8 - soil type A) have been extracted from the ITACA (ITalian ACCELERometric Archive) database (http://itaca.mi.ingv.it/ItacaNet/): namely, the E-W horizontal component of the earthquake recorded in Assisi (PG, Italy) during September 1997 and two N-S accelerations (ANT and AQP) registered at L’Aquila (AQ, Italy) during the earthquake of April 2009. The relevant characteristics of the selected acceleration time histories are listed in Table 2 in terms of magnitude ($M_w$), Arias Intensity ($I_a$) as proposed by Arias (1970), epicentral distance, effective duration ($T_{90}$) as defined by Trifunac and Brady (1975), maximum acceleration ($a_{\text{max}}$) and maximum velocity ($v_{\text{max}}$). The records have been filtered to a maximum frequency of 10 Hz and linearly scaled to the maximum acceleration values predicted at the site for the two return periods. The normalised response spectra of the adopted input motions are shown in Figure 4 together with their average spectrum: a
reasonable match with the response spectrum provided by Eurocode 8 (EC8) for soil type A can be observed. Figure 5 reports the acceleration time histories of the selected input motions scaled to 0.313 g: the first one (ANT - NS) is characterised by a dominant frequency of 1.8 Hz, the second (AQP - NS) by a dominant frequency of about 5 Hz, while the third one (ASSISI - EW) by an intermediate value of 2.9 Hz. The selected accelerograms are characterised by a broad variety of dominant frequency, Arias intensity and duration values, being representative of a wide range of possible events occurring at Avezzano.
**Numerical simulations**

Advanced plane strain analyses of the static and dynamic behaviour of the silo shallow foundation have been performed using the two-dimensional finite element code *SWANDYNE II* (Chan 1995), which implements a fully-coupled effective stress approach. The numerical formulation of the FE code is summarised in Appendix I. The mechanical behaviour of the two silty sand layers (L₁ and L₄) has been modelled through a simple Mohr-Coulomb law while the advanced elasto-plastic constitutive model developed by Rouainia and Muir Wood (2000) has been adopted to describe the behaviour of the two cemented clayey strata (L₂ and L₃). The main features of the employed soil constitutive models and their calibration against laboratory data, together with the description of the finite element model of the silo foundation deposit are described in the following.

**Soil constitutive models and calibration**

The constitutive model adopted to investigate seismic response of the silo foundation has been formulated for natural clays within the framework of kinematic hardening with some elements of bounding surface plasticity (Rouainia and Muir Wood 2000). This model converges to the Modified Cam-Clay model for remoulded structureless soils. The *RMW* model allows to reproduce some of the key features of the cyclic behaviour of natural clays as the decay of the shear stiffness with strain amplitude, the corresponding increase of hysteretic damping and the related accumulation of excess pore water pressure and structure degradation under undrained conditions.

The model contains three surfaces. The reference surface controls the state of the soil in its reconstituted, structureless form and describes the intrinsic behaviour of the clay (Burland 1990). The structure surface controls the process of destructuration which can be accompanied by significant strain-softening effects. The bubble, which encloses the elastic domain of the soil, moves within the structure surface following a kinematic hardening rule. The decrease of stiffness with
strain is controlled by an interpolation function which ensures a smooth movement of the elastic
domain towards the structure surface during loading. The Rouainia and Muir Wood model has been
implemented in SWANDYNE II with an explicit stress integration algorithm adopting a constant
strain sub-stepping scheme. For more details on the formulation and implementation of the model
the reader is referred to Rouainia and Muir Wood (2000; 2001) and Zhao et al. (2005). The model
has been successfully employed to simulate both static (Gonzáles et al. 2012; Panayides et al. 2012)
and dynamic geotechnical problems (Elia and Rouainia 2013). Appendix II reports its governing
equations.

The RMW model has been calibrated using oedometer and undrained triaxial compression tests
carried out on an undisturbed sample representative of the Avezzano cemented clayey layer L2. In
particular, $\lambda^*$ has been derived from the final slope of the one-dimensional compression curve in
the $\ln v : \ln p$ plane, whereas $M_\theta$ has been calibrated against the value of the stress ratio at critical
state observed during the triaxial compression tests. Moreover, the structure surface has been
considered centered on the isotropic axis (i.e. $\eta_0 = 0$), assuming a constant value of the initial
degree of structure ($r_0$). The other parameters, together with the initial values of the hardening
variables, have been derived through a trial and error procedure in order to match the experimental
results. The parameters $A$ and $k$, which govern the rate of bond degradation, have been obtained
from one-dimensional and triaxial compression tests. Figure 6 compares the triaxial test data with
the corresponding model predictions: the general trend is well captured in terms of pre-failure
stress-strain relationships and pore water pressures generated during triaxial shearing up to a
maximum deviatoric strain of 15%. The RMW model correctly predicts the brittle response and the
associate post-peak behaviour observed during the triaxial tests. In addition to matching oedometer
and monotonic triaxial data, results of resonant column (RC) and Down-Hole tests (Figure 2b) have
been employed to assess the small-strain shear stiffness ($G_0$) profile along the foundation deposit. In
the RMW model, the bulk and shear moduli, $K$ and $G$, are assumed to depend linearly on the mean
effective pressure \( p \), according to a typical hypoelastic formulation. Therefore, the elastic parameter \( \kappa^* \) has been calibrated to capture the maximum shear modulus \( G_0 \) obtained from dynamic tests, assuming a value of \( \nu \) equal to 0.25 due to the lack of reliable experimental data. Numerical simulations of strain-controlled undrained cyclic simple shear tests have also been carried out in order to calibrate the model parameters controlling the reduction of shear modulus with cyclic shear strain, \( \gamma \) (e.g. Elia et al. 2011a). The results, reported in Figure 7, demonstrate the ability of the advanced model to capture the decay of the shear stiffness with strain amplitude and the corresponding increase of hysteretic damping, typically observed during laboratory cyclic tests on clays (e.g. Sangrey et al. 1969; Castro and Christian 1976; Vucetic and Dobry 1991). The secant shear modulus for each shear strain amplitude has been assessed after 500 load cycles, a number sufficient to reach steady-state condition. The calculated reduction of normalised shear modulus \( G/G_0 \) and the variation of damping \( D \) with \( \gamma \) are reported in Figure 8. The results cannot be directly compared with those obtained by Vucetic and Dobry (1991) through RC tests on reconstituted clay samples, since the numerical simulations with \( RMW \) are representative of a natural clay behaviour. A part from the initial shear modulus values (Figure 2b), the full data of resonant column tests performed on undisturbed samples retrieved at different depths from the clayey soil were not available and thus cannot be reported in Figure 8. Table 3 summarises the values of the model parameters adopted for soil layer \( L_2 \). These values are similar to the mean values adopted by Burghignoli et al. (2003) in their calibration of \( RMW \) model for the same soil sample data. This confirms the robustness of the calibration procedure and allows to adopt in this study similar model parameters proposed by Burghignoli et al. (2003) for \( L_3 \), as reported in Table 3.

A linear elastic-perfectly plastic model, with a Mohr-Coulomb strength criterion and a non associated flow rule, has been employed to simulate the stress-strain behaviour of the silty sand layers \( L_1 \) and \( L_4 \). In particular, a friction angle of 40° and a constant shear modulus of 110 MPa have been assumed for the first layer \( L_1 \), while 38° and 475 MPa are the corresponding values used
for L₄. The friction angles have been determined from CPT cone resistance profiles (D’Elia et al. 1999; Burghignoli et al. 1999), reported in Figure 2a, whereas the adopted values of the elastic shear modulus are comparable with those obtained from Down-Hole tests (Figure 2b). Moreover, according to the laboratory data (Figure 2c), unit weights, \( \bar{\gamma} \), of 17 kN/m³ and 18 kN/m³ have been used for L₁ and L₄, respectively. Although the limitations associated with the use of a single surface model like Mohr-Coulomb in dynamic analyses are well-known, the lack of experimental data on the silty sand layers L₁ and L₄ has resulted in a challenging task to calibrate and use advanced constitutive models for granular materials (Alyami et al. 2009; Elia et al. 2011b).

Finite element model

A mesh of 1170 isoparametric quadrilateral finite elements with 8 solid nodes and 4 fluid nodes has been used in this study (Figure 9), considering a foundation deposit 200 m wide and 70 m deep. The silo reinforced concrete footing has been simulated with a 3 m thick, 34 m wide rigid block, located 4 m below ground surface. The above-ground structure has not been modelled in this work, as the analysis of soil-structure interaction is beyond the scope of the paper. Standard boundary conditions have been assumed during the static analyses: the solid nodes at the bottom of the mesh have been fixed in both vertical and horizontal directions, while the nodes along the lateral sides of the mesh have been fixed in the horizontal direction only. The ground water level has been located 1 m below the foundation level, in agreement with the site investigation data. Base and lateral hydraulic boundaries have been assumed as impervious while drained condition has been imposed at the top of the mesh in all the simulations. The assumed hydraulic conductivity for the two clayey layers is equal to 1.0E-08 m/s (Fioravante et al. 1994), while a typical value of 1.0E-05 m/s has been adopted for the two silty sand layers.

An initial elastic static analysis has been performed to define the geostatic stress state. For consistency with previous numerical works (e.g. D’Elia et al. 1999 and Burghignoli et al. 2003), a
coefficient of earth pressure at rest $K_0$ equal to 0.5 has been assumed for all layers. The first loading
and unloading cycles of the silo have then been simulated. The foundation settlements computed at
the end of the two static stages assuming plane strain conditions are in good agreement with the
results of previous axisymmetric studies (i.e. Burghignoli et al. 2003). Figure 3c reports the final
settlements obtained during the loading and unloading stages (indicated as A and B) which
represent the starting points of the dynamic simulations discussed in the following sections.

Figure 10 shows the initial shear modulus ($G_0$) profiles obtained at the end of the first loading and
unloading simulation along the centre axis of the footing. The figure also reports the $G_0$ profiles and
the shear modulus values obtained from in-situ Down-Hole and resonant column tests, respectively
(Figure 2b). The initial stiffness profile along the foundation deposit at the end of the static loading
phase plays a crucial role in the simulated dynamic behaviour of the silo and, as such, has been
checked to be consistent with the experimental data. Once the static analyses of the foundation have
been completed, the response of the silo during an earthquake motion has been studied, as the silo
structure is located in a seismic-prone zone (EERI 2009). According to modern performance-based
design approaches (EN 1998-1 2004), the results of these FE simulations represent a class A
prediction of the silo foundation dynamic response, as no direct displacement and pore pressure
measurements during real seismic motions are available. In the dynamic analyses of the silo, the
selected input records have been directly applied to the solid nodes at the base of the mesh (now
fixed in the vertical direction only) as prescribed horizontal displacement time histories. It should
be noted that the peak ground accelerations predicted by the seismic hazard study of the site
correspond to input motions at ‘outcropping’ bedrock. Nevertheless, no deconvolution analysis of
the seismic actions has been performed in this study as this may lead to controversial results in
highly nonlinear soil deposits (e.g. Kwok et al. 2007; Towhata 2008). Tied-node boundary
conditions along the vertical sides of the mesh have been employed for the dynamic simulations to
avoid spurious wave reflections at the boundaries of the foundation layer. Their effectiveness in
absorbing the energy induced by the seismic action has been proved by Zienkiewicz et al. (1999). In addition, a parametric study of the FE model length has shown that the adopted horizontal dimension (i.e. 200 m), in conjunction with the tied-node boundaries, is sufficient to properly simulate the free-field conditions at the edges of the model. No numerical damping has been introduced through the time step integration scheme (i.e. $\beta_1 = \beta_1^* = \beta_2 = 0.5$, see Appendix I), while 10% and 2% of Rayleigh damping, associated with the frequencies of 0.80 Hz and 2.40 Hz, has been added in the dynamic simulations to the silty sand and cemented clayey layers, respectively. Each earthquake has been applied at the bedrock of the foundation deposit after the first loading, when the silo is full of sugar, and after the first unloading, when it is empty. The different $G_0$ profiles obtained at the end of the two static phases (see Figure 10) justifies the option of performing two dynamic analyses for each input motion. Therefore, a total of twelve FE dynamic simulations have been carried out: six of them have been performed to study the CLS conditions, applying at bedrock the three earthquakes scaled to a maximum acceleration of 0.313 g, and the other six to analyse the DLS, scaling the input motions to a maximum acceleration of 0.097 g. In all cases, the seismic response of the silo foundation has been analysed for the entire duration of the selected records and the following 10 seconds, using a time step of 0.005 s equal to the time interval of the earthquake traces. The key results of the FE dynamic simulations, which represent the main contribution of this work over previous studies, are discussed in the following section.
Results of the dynamic simulations and discussion

A summary of the results obtained at the end of the twelve fully-coupled effective stress dynamic simulations is reported for both CLS and DLS cases in Table 4 in terms of maximum vertical displacement and maximum acceleration recorded at ground surface. In the following, ANT#0.313L is the name adopted for the simulation during which the ANT earthquake, scaled to 0.313g, has been applied at the bedrock of the foundation deposit after the first loading (full silo), whereas ANT#0.313U represents the dynamic analysis where the same input motion has been imposed to the empty silo (i.e. after the first unloading). Similar name distinction is adopted for the remaining earthquakes. The performance of the Avezzano silo is described in terms of wave propagation, displacement field, destructuration and stress-strain behaviour induced by the seismic action.

Wave propagation and acceleration field

The periods of maximum amplification of the seismic wave during its propagation through the soil deposit have been identified normalising the response spectra of the accelerations recorded at ground surface below the silo by the corresponding input ones. The normalised response spectra for the CLS and DLS simulations are reported in Figures 1a and 1b, respectively. In the CLS case, the maximum amplification occurs in the range of periods between 0.6 and 0.9 s, but the first natural period cannot be clearly identified (Figure 1a). The AQP input motion is de-amplified at ground surface, as its dominant frequency is far from the first mode of the system, while the soil deposit tends to amplify the maximum acceleration of the ANT and ASSISI earthquake events at surface. For each input motion, higher amplification always occurs when the seismic action hits the empty silo. Very similar results are obtained in the case of DLS simulations, as shown in Figure 1b: the period of maximum amplification can now be recognized at about 0.6 s, representing the first mode period of the soil deposit. Moreover, the dynamic simulations performed using a lower
return period (i.e. DLS case) exhibit much higher amplification effects as compared to the CLS analyses, even though they are characterised by lower energy content. The smaller shear strain amplitude induced in the soil deposit by the less extreme events enhances the transmission of the high frequencies of the seismic signal and increases the amplification of the peak accelerations at surface.

The free-field behaviour of the soil deposit has also been investigated. Figures 12a and 12b show the normalised spectra recorded, respectively, at ground surface below the silo and in free-field conditions. The presented results refer to the AQP#0.313L and AQP#0.313U simulations. As it can be expected, the normalised spectra along a vertical profile situated between the foundation and the model boundaries (i.e. free-field) are not affected by the loading history of the silo (Figure 12b). These results demonstrate the effectiveness of the tied-node boundaries adopted in the dynamic simulations.

Figures 13a and 13b present the normalised profiles with non-dimensional depth of the maximum acceleration along the silo axis for the CLS and DLS simulations, respectively. With the exception of the AQP#0.313L simulation, when an overall de-amplification of the input motion is observed, the profiles are always characterised by a significant amplification of the seismic signal at the top of the soil deposit. In particular, amplification factors of about 2 over the peak base amplitude are registered when the silo is empty and the earthquakes are scaled to 0.313 g, while the overall amplification occurring between the bedrock and the ground surface is bigger when events with lower return period hit the empty structure. The ANT input motion represents the most demanding event, being characterised by a dominant period (0.55 s) very close to the first mode of vibration of the soil deposit. During the ANT#0.097U analysis an amplification factor of 3.78 over the peak bedrock amplitude is recorded, while the ASSISI earthquake represents an intermediate loading condition between the ANT and AQP events.
Displacement field

Although higher amplifications along the soil deposit are obtained when the silo is empty, more demanding situations in terms of induced displacements and cumulated excess pore water pressures are represented by the cases when the earthquakes are imposed to a fully loaded structure, because of the higher applied load.

In particular, in the case of the ANT#0.313L simulation, the evolution with time of vertical displacements of nodes along the silo axis is shown in Figure 14a. The maximum settlement, recorded at surface, is equal to 0.17 m, while the vertical displacement obtained at the contact line between L3 and L4 is negligible (0.005 m). The foundations settlement computed during the seismic action is considerably higher that the one accumulated during the first applied load of the silo (75 mm). This is related to a higher degradation of structure induced in the cemented clayey soils by the earthquake action, as discussed later. The computed displacement time histories become constant immediately after the end of the earthquake (i.e. after 30 s) in all the monitored nodes, indicating a stable behaviour of the foundation after the seismic action.

The time histories of horizontal displacements (relative to bedrock) recorded at different depths along the centre of the footing are reported in Figure 14b. Again, very small permanent horizontal displacements are recorded in the bottom silty sand layer at the end of the simulation, due to the modest plastic strain accumulation predicted by Mohr-Coulomb model adopted for this stratum. Moreover, the nodes in the top layers cumulate permanent positive horizontal displacements (i.e. directed to the right), thus indicating an overall non-symmetric behaviour of the deposit with respect to the centre axis. The maximum horizontal displacement recorded at ground surface below the centre of the footing is equal to 0.025 m, while the one calculated at a depth of 25 m is almost zero.

A significant advantage of the fully-coupled nonlinear approach adopted in this study is that it is based on effective stress analysis, accounting for the dynamic interaction between the soil skeleton.
and the pore fluid. The excess pore water pressures cumulated inside the soil deposit throughout the shaking are shown in Figure 14c for different depths along the silo axis. The clayey layers behave essentially in undrained conditions during the shaking, cumulating high positive pore water pressures which increase with depth. This can be attributed to the low soil permeability which does not allow dissipation of pore water pressures in a short interval of time, such as the duration of the seismic action and the following 10 seconds. Positive excess pore pressures are also recorded at the bottom of the silty sand layer, with a maximum value of 76 kPa at the depth of 50 m, while the pore pressures oscillate around zero in the top granular material due to the proximity to the drained hydraulic boundary.

Figures 15a and 15b show, respectively, the contour lines of vertical and horizontal displacements at the end of the dynamic action. The overall behaviour of the soil deposit in terms of vertical displacement (Figure 15a) is the one typically observed in the case of rigid footings (i.e. the settlements are almost constant below the foundation). Maximum positive displacements (i.e. directed upwards) of about 0.10 m are recorded outside the foundation area at ground surface. Figure 15b confirms the non-symmetric response in terms of permanent horizontal displacement shown in Figure 14b. Moreover, it can be observed how the maximum values (of about 0.30 m) are obtained below the corners of the footing and not along its centre line, at a depth of 15 m below ground surface (i.e. in layer L2).

The contour lines of excess pore pressures obtained at the end of the earthquake action are shown in Figure 15c. Positive values increasing with depth are recorded below the foundation in L2, up to a maximum of about 150 kPa. This is associated with the observed accumulation of plastic strains and the corresponding loss of structure induced by the shaking at the interface between L2 and L3, due to the high difference between the two cemented clayey soils in terms of strength and initial degree of structure (see Table 3). Positive excess pore water pressures are also recorded along the silo axis at the interface between L3 and L4, with a maximum value of 70 kPa, and in the bottom Mohr-
Coulomb layer, reaching the maximum value of 120 kPa at bedrock level.

At the end of the dynamic simulation, the satisfaction of the equilibrium condition for both the solid and the fluid phase has been imposed (i.e. the long term, fully drained, condition has been studied).

This leads to a consolidation settlement below the footing of 0.032 m, which corresponds to 19% of the maximum vertical displacement cumulated during the earthquake action.

The displacement behaviour of the silo foundation subjected to different levels of earthquake events under different initial static conditions is summarised in Table 4. It can be observed that in the CLS case the final settlement predicted at ground surface ranges between 170 and 2 mm, while smaller vertical displacements of the footing, ranging between 37 and 0.1 mm, are obtained during the DLS simulations. The numerical investigation indicates how the seismic action could represent a very demanding loading condition for the structure, especially in the case of CLS predictions. This may induce a final vertical displacement of the foundation more than twice bigger than the one caused by the application of the static load.

Destructuration and stress-strain behaviour

To better understand the effects of the dynamic action on the behaviour of the silo, the evolution of degree of structure with time has been monitored in the cemented soils during the application of the different input motions. This represents one of the first attempts to investigate the effects of earthquake loading on the induced structure degradation in natural clayey deposits. In particular, the time histories of the $RMW$ state variable $r$ recorded along the silo axis during the ANT#0.313L analysis are shown for different depths of $L_2$ and $L_3$ in Figures 16a and 16b, respectively. The structure degradation induced with depth by the static load related to the first stocking activity of the silo is negligible in both clayey layers, as the starting value of the state variable $r$ is essentially equal to its initial value $r_0$ (i.e. 9.5 for $L_2$ and 5.2 for $L_3$). On the contrary, the earthquake action induces significant destructuration, especially in the top natural clay where the structure
progressively reduces with time and depth (Figure 16a), reaching the maximum loss of structure of 54% at the bottom of the layer (z = 16 m). Smaller destructuration effects during the shaking are observed in the clayey soil L₃ (Figure 16b) where a maximum loss of structure of about 19% is recorded at the depth of 18 m. Moreover, the time histories of the state variable $r$ remain constant in all the monitored nodes of layers L₂ and L₃ during the 10 seconds following the seismic action. This is consistent with the observed behaviour of the soil deposit described in the previous section in terms of permanent displacements. For comparison, Figures 16c and 16d report the corresponding time histories of the RMW state variable $r$ recorded in layers L₂ and L₃, respectively, when the empty silo is subjected to the same earthquake motion (i.e. during the ANT#0.313U analysis). A similar pattern in terms of structure degradation can be observed in this case, with smaller destructuration occurring in the clay layers overall. When the silo is empty, the maximum predicted loss of structure is equal to 41% in layer L₂ and 16% in layer L₃. This comparison reveals that higher permanent displacements are induced by the input motion when the silo is fully loaded.

Finally, the stress-strain curves recorded during the ANT#0.313L analysis along the axis in the four layers are shown in Figure 17. It can be observed that the clayey soils L₂ and L₃ (where the advanced RMW model has been adopted) exhibit a continuous change of stiffness and the accumulation of plastic shear deformations all through the shaking (Figures 17b and 17c). In contrast, the stress-strain curves recorded at the stress points situated at the top and bottom L₁ and L₄ layers are representative of an elastic behaviour, with very small accumulation of plastic strains during the earthquake excitation (Figures 17a and 17d). These results corroborate the hypothesis that the displacements accumulated during the shaking are essentially controlled by the mechanical behaviour of the two natural clay layers L₂ and L₃.
Conclusion

In this paper, a fully-coupled numerical analysis of the dynamic behaviour of the shallow foundation of a sugar silo founded on structured clay was presented. The constitutive model adopted accounts for the influence of initial structure in the clay together with progressive destructuration, and allows the prediction of a continuous change of stiffness and cumulated plastic shear deformations. The model parameters were calibrated using a series of oedometer and undrained triaxial tests carried out on undisturbed samples representative of the structured clayey soils.

In the numerical work undertaken here, three acceleration time histories were selected and scaled from earthquake databases according to the seismic hazard study of the specific site. The dynamic behaviour of the silo was analysed in both full and empty conditions by applying the selected earthquakes at the bedrock of the foundation deposit. The overall behaviour of the silo, in terms of response spectra and profiles of maximum acceleration with depth, indicated a higher amplification of the seismic motion when the earthquake hits the empty structure. However, larger displacements and higher accumulated excess pore water pressures were predicted when the earthquake was applied to a fully loaded silo. This is due to the higher inertia of the shaken mass (i.e. higher applied load) and the significant loss of structure induced with depth by the seismic action, especially in the top cemented soil. The positive excess pore water pressures generated in the clayey layers are indicative of an undrained contractive behaviour associated with a continuous accumulation of plastic strains during the shaking. Additional foundation settlements related to consolidation processes were calculated at the end of the earthquake loading. The performance of the silo footing under dynamic conditions must be considered with care during the design of the structure, as extreme earthquake events can induce ground settlements up to two times larger than those caused by static loads.
More generally, numerical modelling of shallow foundations in seismic zones suggests that there are significant advantages in using advanced models which recognise the existence and subsequent loss of structure in natural soils. With the current advances in geotechnical research, it is nowadays possible to undertake sophisticated numerical modelling using advanced soil constitutive models to accurately predict the deformation behaviour of geo-structures under dynamic loading. This will allow the full implementation of seismic performance-based design for geotechnical structures.

Acknowledgements

The Authors would like to thank the two anonymous reviewers for their valuable comments and suggestions.
References


Appendix I: Numerical formulation of the finite element code

In the context of FE analysis, assuming that the relative velocity of the fluid phase is negligible, the system of ordinary equation that results from the \( u-p \) formulation can be written as follows (Biot 1941; Zienkiewicz et al. 1999):

\[
\begin{align*}
\left[ [M] \dot{\mathbf{u}} + [C] \mathbf{u} + [K] \mathbf{u} - [Q] \mathbf{p} \right] &= \mathbf{f}^s \\
\left[ [Q]^T \dot{\mathbf{p}} + [S] \mathbf{p} + [H] \mathbf{p} \right] &= \mathbf{f}^p
\end{align*}
\] (1)

where \( \mathbf{u} \) is the solid phase displacement vector and \( \mathbf{p} \) is the pore fluid pressure vector, \([M]\) is the mass matrix, \([K]\) is the stiffness matrix, \([C]\) is the viscous damping matrix, \([Q]\) is the coupling matrix between the motion and flow equations, \([H]\) is the permeability matrix, \([S]\) is the compressibility matrix, \(\mathbf{f}^p\) is the force vector for the fluid phase and \(\mathbf{f}^s\) is the force vector for the solid phase. Frequency dependent viscous damping is included via the Rayleigh damping matrix (Clough and Penzien 1993):

\[
[C] = \alpha [M] + \beta [K]
\] (2)

where the factors \( \alpha \) and \( \beta \) are related to the modal damping coefficients according to the relationship:

\[
\begin{bmatrix}
\alpha \\
\beta
\end{bmatrix} = \frac{2D}{\omega_m + \omega_n} \begin{bmatrix}
\omega_m \omega_n \\
1
\end{bmatrix}
\] (3)

These coefficients can be calculated by selecting one value of damping ratio \( D \) and two frequencies, \( \omega_m \) and \( \omega_n \), outside which damping is larger than the selected value.

The algebraic counterparts of Equations (1) are obtained by applying a time-integration scheme.
Assuming that the values of displacements, pore pressures and their time derivatives \( \{u_n, \dot{u}_n, \ddot{u}_n, p_n, \dot{p}_n\} \) have been obtained at time \( t_n \), the integration consists of updating \( \{u_{n+1}, \dot{u}_{n+1}, \ddot{u}_{n+1}, p_{n+1}, \dot{p}_{n+1}\} \) at the next time step \( t_{n+1} \) according to the Generalised Newmark scheme (Katona and Zienkiewicz 1985). In particular, for the solid phase:

\[
\begin{align*}
\ddot{u}_{n+1} &= \ddot{u}_n + \Delta \ddot{u}_n \\
\dot{u}_{n+1} &= \dot{u}_n + [\ddot{u}_n + \beta_1 \Delta \ddot{u}_n] \Delta t \\
u_{n+1} &= u_n + \dot{u}_n \Delta t + 0.5[\ddot{u}_n + \beta_2 \Delta \ddot{u}_n] \Delta t^2
\end{align*}
\]  

(4)

Similarly for the fluid phase:

\[
\begin{align*}
\dot{p}_{n+1} &= \dot{p}_n + \Delta \dot{p}_n \\
p_{n+1} &= p_n + [\dot{p}_n + \beta_1^* \Delta \dot{p}_n] \Delta t
\end{align*}
\]  

(5)

where the coefficients:

\[
\begin{align*}
\beta_1 &\geq 0.5 \\
\beta_2 &\geq 0.5(0.5 + \beta_1)^2 \\
\beta_1^* &\geq 0.5
\end{align*}
\]  

(6)

are typically chosen for unconditional stability of the recurrence scheme (Zienkiewicz et al. 1999). The substitution of the above approximations into Equations (1) leads to a system of coupled nonlinear equations which are solved iteratively by the FE code using the Newton-Raphson procedure.
Appendix II: Constitutive model formulation

Figure 18 shows the three characteristic surfaces of the RMW model in the $p : q$ plane and its formulation is summarised in the following. The expression of the reference surface is:

$$f_r = \frac{3}{2M_\theta^2} s : s + (p - p_c)^2 - (p_c)^2 = 0$$  \hspace{1cm} (7)$$

The bubble surface is written as:

$$f_b = \frac{3}{2M_\theta^2} (s - s_\alpha) : (s - s_\alpha) + (p - p_\sigma)^2 - (Rp_c)^2 = 0$$  \hspace{1cm} (8)$$

The structure surface is given by:

$$F = \frac{3}{2M_\theta^2} \left[ s - (r - 1) \eta_0 p_c \right] : \left[ s - (r - 1) \eta_0 p_c \right] + (p - rp_c)^2 - (rp_c)^2 = 0$$  \hspace{1cm} (9)$$

where $p_c$ is the effective stress which defines the size of the reference surface, $R$ is the size of the bubble, $M_\theta$ is a dimensionless scaling function for deviatoric variation of the critical state stress ratio, $\eta_0$ a deviatoric tensor controlling the structure, $r$ is the ratio of the sizes of the structure and the reference surfaces, $p$ and $s$ are the mean pressure and deviatoric stress tensor and the symbol ‘$:'$ indicates a summation of products. Since the model describes the response of the soil skeleton, all stresses are effective stresses (the primes have been dropped for simplicity). The dots over symbols indicate an infinitesimal increment of the corresponding quantity, whereas bold-face symbols indicate tensors.
The scalar variable \( r \), which is a monotonically decreasing function of both plastic volumetric and shear strain, represents the progressive degradation of the material as follows:

\[
\dot{r} = -\frac{k}{(\lambda^- - \kappa^+)} (r-1) \dot{\varepsilon}_d
\]

(10)

where \( \lambda^- \) and \( \kappa^+ \) are the slopes of normal compression and swelling lines in the ln \( v \) : ln \( p \) compression plane (being \( v \) the soil specific volume) and \( k \) is a parameter which controls the structure degradation with strain. The rate of the destructuration strain \( \dot{\varepsilon}_d \) is assumed to have the following form:

\[
\dot{\varepsilon}_d = \left[ (1-A)(\dot{\varepsilon}_q^p)^2 + A(\dot{\varepsilon}_v^p)^2 \right]^{\frac{1}{2}}
\]

(11)

where \( A \) is a non-dimensional scaling parameter and \( \dot{\varepsilon}_q^p \) and \( \dot{\varepsilon}_v^p \) are the plastic shear and volumetric strain rate, respectively.

Volumetric hardening rule is adopted in the model, where the change in size of the reference surface, \( p_c \), is controlled only by plastic volumetric strain rate, \( \dot{\varepsilon}_v^p \), given by:

\[
\frac{\dot{p}_c}{p_c} = \frac{\dot{\varepsilon}_v^p}{\lambda^- - \kappa^+}
\]

(12)

If a stress increment requires movement of the bubble relative to the structure surface, the following kinematic hardening is invoked:
\[ \hat{\alpha} = \dot{\alpha} + \frac{\dot{p}_e}{p_e} (\bar{\alpha} - \hat{\alpha}) + \mu (\sigma - \sigma) \]  

(13)

where \( \bar{\alpha} \) and \( \hat{\alpha} = p_e \left[ rI + (r-1)\eta \right] \) denote the locations of the centre of the bubble and structure surface respectively, \( \sigma \) is the conjugate stress and \( \mu \) is a positive scalar of proportionality. It should be noted that the centre of the structure surface and the deviator of \( \hat{\alpha} \) represents the anisotropy of the soil due to structure. The deviator of \( \hat{\alpha} \) therefore degrades to zero as \( r \) degrades to unity.

The plastic modulus \( H \) is assumed to depend on the distance between the current stress and the conjugate stress and is given by:

\[ H = H_c + \frac{Bp_e^3}{(\lambda - \kappa')R \left( \frac{b}{b_{\text{max}}} \right)^\psi} \]  

(14)

where \( H_c \) is the plastic modulus at the conjugate stress, \( B \) and \( \psi \) are two additional material properties, \( b = \bar{n} : (\sigma - \sigma) \) is the normalised distance between the bubble and the structure surface and \( b_{\text{max}} = 2(r/R - 1)\bar{n} : (\sigma - \bar{\alpha}) \) is its maximum value.

Finally, the bulk and shear moduli, \( K \) and \( G \), are assumed to depend linearly on the mean effective pressure \( p \):

\[ K = \frac{p}{\kappa'} \quad G = \frac{3(1-2\nu)}{2(1+\nu)} K \]  

(15)

where \( \nu \) is a constant Poisson’s ratio.
Notation

A parameter controlling relative proportion of distorsional and volumetric destructuration

B stiffness interpolation parameter

b normalised distance between bubble and structure surface

$b_{\text{max}}$ maximum value of $b$

F structure yield surface

$f_r$ reference yield surface

$f_b$ bubble yield surface

G shear modulus

H plastic modulus

$H_c$ plastic modulus at conjugate stress

I second rank identity tensor

K bulk modulus

k parameter controlling rate of loss of structure with damage strain

$M_\theta$ dimensionless scaling function for deviatoric variation of critical state stress ratio

$\bar{n}$ normalised stress gradient on the bubble

$p$ mean effective stress

$P_c$ stress variable controlling size of the surfaces

$q$ scalar deviator stress

$R$ ratio of sizes of bubble and reference surface

$r$ parameter describing ratio of sizes of structure and reference surfaces

$r_0$ initial value of $r$

$s$ tensorial deviator stress

$u, \Delta u$ pore and excess pore pressure

$v$ specific volume

$\bar{\alpha}$ location of the centre of the bubble

$\hat{\alpha}$ location of the centre of the structure surface

$\varepsilon_v^p$ volumetric strain

$\varepsilon_q^p$ deviatoric strain

$\varepsilon_d$ damage strain
\( \gamma \) cyclic shear strain amplitude
\( \bar{\gamma} \) unit weight
\( \eta_0 \) dimensionless deviatoric tensor (anisotropy of structure)
\( \kappa^* \) slope of swelling line in \( \ln v : \ln p \) compression plane
\( \lambda^* \) slope of normal compression line in \( \ln v : \ln p \) compression plane
\( \mu^* \) positive scalar of proportionality
\( \nu \) Poisson’s ratio
\( \sigma \) effective stress tensor
\( \sigma_c \) conjugate stress
\( \psi \) stiffness interpolation exponent

\( a_{\text{max}} \) earthquake maximum acceleration
\( D \) damping ratio
\( I_a \) Arias intensity
\( M_W \) earthquake moment magnitude
\( P \) probability of exceedance
\( T_{90} \) earthquake effective duration
\( T_R \) return period
\( \alpha, \beta \) modal damping coefficients
\( \beta_1, \beta_2, \beta_1^* \) generalised Newmark parameters
\( \lambda \) annual frequency of exceedance
\( \omega \) angular frequency
<table>
<thead>
<tr>
<th>Limit State</th>
<th>Probability of exceedance in 50 years $P$ (%)</th>
<th>Return period $T_r$ (years)</th>
<th>Annual frequency of exceedance $\lambda$ (years$^{-1}$)</th>
<th>Maximum acceleration $a_{\text{max}}$ (g)</th>
</tr>
</thead>
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<tr>
<td>CLS</td>
<td>5</td>
<td>975</td>
<td>0.0010</td>
<td>0.313</td>
</tr>
<tr>
<td>DLS</td>
<td>63</td>
<td>50</td>
<td>0.0200</td>
<td>0.097</td>
</tr>
</tbody>
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**Table 1** Results of the seismic hazard study at the site of the silo
<table>
<thead>
<tr>
<th>Station</th>
<th>Earthquake</th>
<th>Component</th>
<th>Magnitude $M_W$</th>
<th>Arias Intensity $I_a$ (m/s)</th>
<th>Epicentral distance (km)</th>
<th>Duration $T_{90}$ (s)</th>
<th>$a_{\text{max}}$ (g)</th>
<th>$v_{\text{max}}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Antrodoco (ANT)</td>
<td>L’Aquila Mainshock 2009</td>
<td>NS</td>
<td>6.3</td>
<td>0.0176</td>
<td>23.0</td>
<td>21.32</td>
<td>0.026</td>
<td>0.013</td>
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<tr>
<td>L’Aquila - V. Aterno (AQP)</td>
<td>L’Aquila Aftershock 2009</td>
<td>NS</td>
<td>4.6</td>
<td>0.0518</td>
<td>0.73</td>
<td>2.12</td>
<td>0.156</td>
<td>0.029</td>
</tr>
<tr>
<td>Assisi - Stallone (ASSISI)</td>
<td>Umbria-Marche 1997</td>
<td>EW</td>
<td>6.0</td>
<td>0.2349</td>
<td>21.4</td>
<td>4.13</td>
<td>0.188</td>
<td>0.050</td>
</tr>
</tbody>
</table>

**Table 2** Main characteristics of the selected seismic actions
<table>
<thead>
<tr>
<th>Layer</th>
<th>$\bar{\gamma}$ (kN/m$^3$)</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>$M_{\theta}$</th>
<th>$R$</th>
<th>$B$</th>
<th>$\psi$</th>
<th>$r_0$</th>
<th>$A$</th>
<th>$k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_2$</td>
<td>16.6</td>
<td>0.11</td>
<td>0.0023</td>
<td>1.55</td>
<td>0.4</td>
<td>11</td>
<td>2.5</td>
<td>9.5</td>
<td>0.2</td>
<td>1.0</td>
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<tr>
<td>$L_3$</td>
<td>18.3</td>
<td>0.11</td>
<td>0.0016</td>
<td>1.40</td>
<td>0.4</td>
<td>11</td>
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<td>5.2</td>
<td>0.2</td>
<td>0.7</td>
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</table>

Table 3 RMW model parameters for layers $L_2$ and $L_3$
<table>
<thead>
<tr>
<th>Name</th>
<th>$a_{\text{bed}}$ (g)</th>
<th>settlement (mm)</th>
<th>$a_{\text{ground}}$ (g)</th>
<th>$a_{\text{ground}}/a_{\text{bed}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANT#0.313L</td>
<td>0.313</td>
<td>170</td>
<td>0.455</td>
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<tr>
<td>AQP#0.313L</td>
<td>0.313</td>
<td>8.4</td>
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<td>0.59</td>
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<td>0.313</td>
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<tr>
<td>ANT#0.313U</td>
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<td>0.686</td>
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<tr>
<td>AQP#0.313U</td>
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<td>0.358</td>
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<td>ASSISI#0.313U</td>
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</tr>
<tr>
<td>ANT#0.097L</td>
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<td>37.0</td>
<td>0.180</td>
<td>1.86</td>
</tr>
<tr>
<td>AQP#0.097L</td>
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<td>2.5</td>
<td>0.109</td>
<td>1.12</td>
</tr>
<tr>
<td>ASSISI#0.097L</td>
<td>0.097</td>
<td>6.8</td>
<td>0.168</td>
<td>1.73</td>
</tr>
<tr>
<td>ANT#0.097U</td>
<td>0.097</td>
<td>9.4</td>
<td>0.367</td>
<td>3.78</td>
</tr>
<tr>
<td>AQP#0.097U</td>
<td>0.097</td>
<td>0.1</td>
<td>0.116</td>
<td>1.19</td>
</tr>
<tr>
<td>ASSISI#0.097U</td>
<td>0.097</td>
<td>1.4</td>
<td>0.258</td>
<td>2.66</td>
</tr>
</tbody>
</table>

*Table 4* Results of the advanced dynamic analyses in terms of settlement and maximum accelerations at ground surface
Fig 1 Cross section of the sugar silo in Avezzano (from Burghignoli et al. 1999)
Fig. 2 Physical and mechanical soil properties at Avezzano (modified from D’Elia et al. 1999 and Burghignoli et al. 1999)
Fig. 3 Sugar silo performance in terms of a) measured average applied pressure, b) measured settlements and c) measured and predicted load-settlement curves
Fig. 4 Normalised response spectra of the selected input motions and comparison with EC8 response spectrum
Fig. 5 Acceleration time histories scaled to 0.313 g
Fig. 6 RMW model calibration against undrained triaxial tests carried out on a sample of soil layer L$_2$
Fig. 7 RMW model prediction of an undrained cyclic simple shear ($p = 300$ kPa) tests carried out on a sample of soil layer $L_2$
Fig. 8 Shear modulus degradation and damping curves predicted by RMW model for soil layer $L_2$
Fig. 9 Adopted FE mesh and boundary conditions
Fig. 10 $G_0$ profiles along the centre line: comparison between computed and measured values
Fig. 11 Ground response spectra of accelerations normalised by the input spectra for a) CLS and b) DLS simulations
Fig. 12 Normalised response spectra calculated during the AQP#0.313L and AQP#0.313U simulations a) below the silo and b) in free-field conditions.
Fig. 13 Normalised profiles of maximum acceleration along the silo axis for a) CLS and b) DLS simulations
Fig. 14 Time histories of a) vertical displacement, b) relative horizontal displacement and c) excess pore pressure recorded along the silo axis during the ANT#0.313L analysis.
Fig. 15 Contour lines of a) vertical displacement, b) relative horizontal displacement and c) excess pore pressure recorded at the end of the ANT#0.313L analysis
Fig. 16 Time histories of the $RMW$ state variable $r$ recorded along the silo axis at different depths a) during the ANT#0.313L analysis in layer $L_2$; b) during the ANT#0.313L analysis in layer $L_3$; c) during the ANT#0.313U analysis in layer $L_2$; d) during the ANT#0.313U analysis in layer $L_3$
Fig. 17 Stress-strain curves recorded along the silo axis at a depth of a) 4 m; b) 10 m; c) 30 m and d) 50 m during the ANT#0.313L analysis
Fig. 18 Characteristic surfaces of RMW model in $p : q$ plane