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Title:

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

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1 **Abstract**

2

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

20

21 **Keywords**

22

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

25

1 **Introduction**

2

3 Tanks and silos are light structures carrying relatively high loads applied in monotonic and (low
4 frequency) cyclic conditions. These structures are usually supported by shallow foundations
5 founded on poor quality sub-soils. Large settlements can be observed upon the first monotonic
6 (pre)loading (e.g. Bjerrum and Overland 1957; Burland 1969; Ricceri 1974; Penman 1978, Muir
7 Wood 1980) and further settlements are due to loading/unloading cycles related to stocking
8 activities, leading, sometimes, to a delayed collapse (e.g. Bozozuk 1979; Bell and Iwakiri 1980;
9 Dogangun *et al.* 2009). Due to the increased need of storage, larger and higher structures are being
10 built all over the world, thus requiring a more careful evaluation of the mechanical performance of
11 their foundation deposits, both in terms of bearing capacity and compressibility behaviour (e.g.
12 Rampello and Callisto 2003). Moreover, such structures can be located in seismic-prone zones and
13 be subjected to proper dynamic (high frequency) loads during earthquakes (e.g. de Sanctis and
14 Russo 2008; Dogangun *et al.* 2009). Therefore, their design necessarily requires the analysis of the
15 effects of the dynamic loading conditions on their stability and serviceability (e.g. Sezen *et al.* 2008;
16 Livaoglu 2008; Sekhar *et al.* 2009; Kianoush and Ghaemmaghami 2011; Moslemi and Kianoush
17 2012).

18 In this context, it can be relevant to account for the essential features of the mechanical behaviour of
19 soils when subjected to cyclic loading: state dependency, early irreversibly, nonlinearity, build-up
20 of excess pore water pressures, evolution of microstructure (de-structuring) and related decrease of
21 nominal stiffness (e.g. Sangrey *et al.* 1969; Castro and Christian 1976; Leroueil and Vaughan 1990;
22 Vucetic and Dobry 1991; Cotecchia and Chandler 1997). These aspects of soil behaviour can be
23 described by adopting advanced constitutive models, developed by adding complexity to the
24 classical single surface Cam-Clay model (e.g. Dafalias and Popov 1975; Pastor *et al.* 1990; Gens
25 and Nova 1993; Asaoka *et al.* 2000; Liu and Carter 2002).

1 In this paper, the cyclic/dynamic behaviour of the shallow foundation of a sugar silo, founded on a
2 cemented clayey soil deposit and subjected to loading/unloading cycles and earthquake actions, is
3 studied through 2D fully-coupled finite element (FE) analyses. The stress-strain behaviour of the
4 foundation soils is modelled using the elastic-perfectly plastic Mohr-Coulomb law for the silty sand
5 layers, while the clayey silt layers are described by the advanced constitutive model developed
6 within the framework of kinematic hardening and bounding surface plasticity by Rouainia and Muir
7 Wood (2000) and named *RMW* in the following. Their calibration is based on the available
8 experimental data obtained from the geotechnical investigation of the site (D'Elia *et al.* 1999;
9 Burghignoli *et al.* 1999). Initially, the first load and unload stages of the silo foundation are
10 simulated and the results in terms of computed settlements are compared with the experimental
11 data. For the dynamic modelling purpose, three different input motions are selected from
12 earthquake databases and scaled to two peak accelerations associated with collapse and damage
13 limit state, respectively. The accelerograms are characterised by a broad variety of mean frequency,
14 Arias intensity and duration values, being therefore representative of a broad range of possible
15 events occurring at the site. The dynamic response of the silo during the earthquake motions is
16 analysed and the results are illustrated in terms of signal amplification, permanent excess pore water
17 pressures, accumulated displacements and progressive loss of structure during and after the seismic
18 loading.

19

1 **The case study**

2

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

7 An extensive geotechnical characterisation of the area, including a large number of in-situ and
8 laboratory tests, was carried out in the 90's. The main results from the site investigation are
9 described in detail by D'Elia *et al.* (1999) and Burghignoli *et al.* (1999). The physical and
10 mechanical soil properties obtained from in-situ and laboratory tests are reported in Figure 2. The
11 results are presented in terms of cone penetration profile, maximum shear modulus from dynamic
12 measurements, soil unit weight, calcium carbonate (CaCO_3) content and compression and unloading
13 indexes obtained from one-dimensional oedometer tests. The typical soil profile of the site consists
14 of two silty sand layers (named L_1 and L_4), at the top and the bottom of the foundation deposit, and
15 two intermediate normally consolidated clayey strata (named L_2 and L_3), characterised by high
16 values of calcium carbonate content ranging between 60 and 80% (Figure 2d). Standard oedometer
17 and undrained triaxial compression tests clearly indicate that both the clayey layers are
18 characterised by the typical mechanical behaviour of cemented soft clays, as discussed later. The
19 water table is located 5 m below ground level.

20 The vertical displacements of the raft foundation and the average applied pressure were recorded
21 during the construction of the silo and the first three loading and unloading cycles related to sugar
22 stocking activities, as reported in Figures 3a and 3b. During the first loading stage, the recorded
23 settlements are very small until an average pressure of about 140 kPa is reached. The gradient of the
24 load-settlement curve then rapidly increases. The unloading stage clearly highlights the irreversible
25 nature of the settlements experienced by the footing during the first loading. In the second loading

1 stage, irreversible strains seem to develop when the applied pressure exceeds its past maximum
2 value. Due to their relatively high compressibility, the two clayey layers play a crucial role in the
3 settlement response of the silo to the applied loads. Previous numerical analyses of the same case
4 study (D'Elia *et al.* 1999; Burghignoli *et al.* 2003) have already demonstrated the importance of
5 using advanced constitutive models to properly describe the mechanical behaviour of the Avezzano
6 cemented clayey soils. Simpler constitutive hypotheses, such as the Modified Cam-Clay (*MCC*)
7 model (Roscoe and Burland 1968), were found to be unable to successfully capture the main
8 features exhibited by the measured load-settlement curve of the sugar silo under working loads, as
9 shown in Figure 3c. The stress changes in the foundation soil were completely enclosed within the
10 yielding surface and an almost elastic response was predicted using *MCC* for the two clay layers
11 (Burghignoli *et al.* 2003). On the contrary, a reasonable agreement between the numerical
12 predictions and measured performance of the silo was obtained using the *RMW* model which
13 accounts for the effects of structure degradation, as reported in Figure 3c. The advanced model,
14 which allows for a continuous decay of plastic modulus along stress paths inside the bounding
15 surface, reproduced correctly the plastic nature of the strain developed during loading and reloading
16 cycles (Burghignoli *et al.* 2003).

17

1 **Seismic hazard analysis and selection of input motions**

2

3 According to the relevant European and Italian code prescriptions (EN 1998-1 2004; M. LL. PP.
4 2008), different levels of seismic action need to be adopted depending on the investigated limit state
5 of the structure. In particular, the Italian code prescribes a probability of exceedance of the seismic
6 action, P , for the collapse limit state (CLS) equal to 5% and a probability of 63% for the damage
7 limit state (DLS). Therefore, assuming a standard reference life of 50 years for the silo, the return
8 periods of the seismic action, T_R , associated with the CLS and the DLS are 975 years and 50 years,
9 respectively. The corresponding peak ground accelerations ($a_{\max}$) predicted at the site according to
10 the seismic hazard study carried out by the Italian National Institute of Geophysics and
11 Volcanology (INGV) on the entire national territory (Gruppo di lavoro MPS 2004) are reported in
12 Table 1. In particular, for a return period of 975 years, associated with the CLS, the seismic hazard
13 analysis predicts a maximum acceleration of 0.313 g while for the return period associated with the
14 DLS the predicted peak ground acceleration is 0.097 g.

15 Three real Italian accelerograms recorded on bedrock (Eurocode 8 - soil type A) have been
16 extracted from the ITACA (ITalian ACcelerometric Archive) database
17 (<http://itaca.mi.ingv.it/ItacaNet/>): namely, the E-W horizontal component of the earthquake
18 recorded in Assisi (PG, Italy) during September 1997 and two N-S accelerations (ANT and AQP)
19 registered at L'Aquila (AQ, Italy) during the earthquake of April 2009. The relevant characteristics
20 of the selected acceleration time histories are listed in Table 2 in terms of magnitude (M_W), Arias
21 Intensity (I_a) as proposed by Arias (1970), epicentral distance, effective duration (T_{90}) as defined by
22 Trifunac and Brady (1975), maximum acceleration ($a_{\max}$) and maximum velocity ($v_{\max}$). The
23 records have been filtered to a maximum frequency of 10 Hz and linearly scaled to the maximum
24 acceleration values predicted at the site for the two return periods. The normalised response spectra
25 of the adopted input motions are shown in Figure 4 together with their average spectrum: a

1 reasonable match with the response spectrum provided by Eurocode 8 (EC8) for soil type A can be
2 observed. Figure 5 reports the acceleration time histories of the selected input motions scaled to
3 0.313 g: the first one (ANT - NS) is characterised by a dominant frequency of 1.8 Hz, the second
4 (AQP - NS) by a dominant frequency of about 5 Hz, while the third one (ASSISI - EW) by an
5 intermediate value of 2.9 Hz. The selected accelerograms are characterised by a broad variety of
6 dominant frequency, Arias intensity and duration values, being representative of a wide range of
7 possible events occurring at Avezzano.

8

1 **Numerical simulations**

2
3 Advanced plane strain analyses of the static and dynamic behaviour of the silo shallow foundation
4 have been performed using the two-dimensional finite element code *SWANDYNE II* (Chan 1995),
5 which implements a fully-coupled effective stress approach. The numerical formulation of the FE
6 code is summarised in Appendix I. The mechanical behaviour of the two silty sand layers (L_1 and
7 L_4) has been modelled through a simple Mohr-Coulomb law while the advanced elasto-plastic
8 constitutive model developed by Rouainia and Muir Wood (2000) has been adopted to describe the
9 behaviour of the two cemented clayey strata (L_2 and L_3). The main features of the employed soil
10 constitutive models and their calibration against laboratory data, together with the description of the
11 finite element model of the silo foundation deposit are described in the following.

12 13 *Soil constitutive models and calibration*

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

21 The model contains three surfaces. The reference surface controls the state of the soil in its
22 reconstituted, structureless form and describes the intrinsic behaviour of the clay (Burland 1990).
23 The structure surface controls the process of destructuration which can be accompanied by
24 significant strain-softening effects. The bubble, which encloses the elastic domain of the soil, moves
25 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 (González *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, λ^* has been derived from the final slope of the one-dimensional compression curve in the $\ln v : \ln p$ plane, whereas M_θ 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

1 effective pressure p , according to a typical hypoelastic formulation. Therefore, the elastic parameter
 2 κ^* has been calibrated to capture the maximum shear modulus G_0 obtained from dynamic tests,
 3 assuming a value of ν equal to 0.25 due to the lack of reliable experimental data. Numerical
 4 simulations of strain-controlled undrained cyclic simple shear tests have also been carried out in
 5 order to calibrate the model parameters controlling the reduction of shear modulus with cyclic shear
 6 strain, γ (e.g. Elia *et al.* 2011a). The results, reported in Figure 7, demonstrate the ability of the
 7 advanced model to capture the decay of the shear stiffness with strain amplitude and the
 8 corresponding increase of hysteretic damping, typically observed during laboratory cyclic tests on
 9 clays (e.g. Sangrey *et al.* 1969; Castro and Christian 1976; Vucetic and Dobry 1991). The secant
 10 shear modulus for each shear strain amplitude has been assessed after 500 load cycles, a number
 11 sufficient to reach steady-state condition. The calculated reduction of normalised shear modulus
 12 G/G_0 and the variation of damping D with γ are reported in Figure 8. The results cannot be directly
 13 compared with those obtained by Vucetic and Dobry (1991) through RC tests on reconstituted clay
 14 samples, since the numerical simulations with *RMW* are representative of a natural clay behaviour.
 15 A part from the initial shear modulus values (Figure 2b), the full data of resonant column tests
 16 performed on undisturbed samples retrieved at different depths from the clayey soil were not
 17 available and thus cannot be reported in Figure 8. Table 3 summarises the values of the model
 18 parameters adopted for soil layer L_2 . These values are similar to the mean values adopted by
 19 Burghignoli *et al.* (2003) in their calibration of *RMW* model for the same soil sample data. This
 20 confirms the robustness of the calibration procedure and allows to adopt in this study similar model
 21 parameters proposed by Burghignoli *et al.* (2003) for L_3 , as reported in Table 3.
 22 A linear elastic-perfectly plastic model, with a Mohr-Coulomb strength criterion and a non
 23 associated flow rule, has been employed to simulate the stress-strain behaviour of the silty sand
 24 layers L_1 and L_4 . In particular, a friction angle of 40° and a constant shear modulus of 110 MPa
 25 have been assumed for the first layer L_1 , while 38° and 475 MPa are the corresponding values used

for L_4 . 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 L_1 and L_4 , 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 L_1 and L_4 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

1 coefficient of earth pressure at rest K_0 equal to 0.5 has been assumed for all layers. The first loading
2 and unloading cycles of the silo have then been simulated. The foundation settlements computed at
3 the end of the two static stages assuming plane strain conditions are in good agreement with the
4 results of previous axisymmetric studies (i.e. Burghignoli *et al.* 2003). Figure 3c reports the final
5 settlements obtained during the loading and unloading stages (indicated as A and B) which
6 represent the starting points of the dynamic simulations discussed in the following sections.

7 Figure 10 shows the initial shear modulus (G_0) profiles obtained at the end of the first loading and
8 unloading simulation along the centre axis of the footing. The figure also reports the G_0 profiles and
9 the shear modulus values obtained from in-situ Down-Hole and resonant column tests, respectively
10 (Figure 2b). The initial stiffness profile along the foundation deposit at the end of the static loading
11 phase plays a crucial role in the simulated dynamic behaviour of the silo and, as such, has been
12 checked to be consistent with the experimental data. Once the static analyses of the foundation have
13 been completed, the response of the silo during an earthquake motion has been studied, as the silo
14 structure is located in a seismic-prone zone (EERI 2009). According to modern performance-based
15 design approaches (EN 1998-1 2004), the results of these FE simulations represent a class A
16 prediction of the silo foundation dynamic response, as no direct displacement and pore pressure
17 measurements during real seismic motions are available. In the dynamic analyses of the silo, the
18 selected input records have been directly applied to the solid nodes at the base of the mesh (now
19 fixed in the vertical direction only) as prescribed horizontal displacement time histories. It should
20 be noted that the peak ground accelerations predicted by the seismic hazard study of the site
21 correspond to input motions at ‘outcropping’ bedrock. Nevertheless, no deconvolution analysis of
22 the seismic actions has been performed in this study as this may lead to controversial results in
23 highly nonlinear soil deposits (e.g. Kwok *et al.* 2007; Towhata 2008). Tied-node boundary
24 conditions along the vertical sides of the mesh have been employed for the dynamic simulations to
25 avoid spurious wave reflections at the boundaries of the foundation layer. Their effectiveness in

1 absorbing the energy induced by the seismic action has been proved by Zienkiewicz *et al.* (1999). In
2 addition, a parametric study of the FE model length has shown that the adopted horizontal
3 dimension (i.e. 200 m), in conjunction with the tied-node boundaries, is sufficient to properly
4 simulate the free-field conditions at the edges of the model. No numerical damping has been
5 introduced through the time step integration scheme (i.e. $\beta_1 = \beta_1^* = \beta_2 = 0.5$, see Appendix I), while
6 10% and 2% of Rayleigh damping, associated with the frequencies of 0.80 Hz and 2.40 Hz, has
7 been added in the dynamic simulations to the silty sand and cemented clayey layers, respectively.
8 Each earthquake has been applied at the bedrock of the foundation deposit after the first loading,
9 when the silo is full of sugar, and after the first unloading, when it is empty. The different G_0
10 profiles obtained at the end of the two static phases (see Figure 10) justifies the option of
11 performing two dynamic analyses for each input motion. Therefore, a total of twelve FE dynamic
12 simulations have been carried out: six of them have been performed to study the CLS conditions,
13 applying at bedrock the three earthquakes scaled to a maximum acceleration of 0.313 g, and the
14 other six to analyse the DLS, scaling the input motions to a maximum acceleration of 0.097 g. In all
15 cases, the seismic response of the silo foundation has been analysed for the entire duration of the
16 selected records and the following 10 seconds, using a time step of 0.005 s equal to the time interval
17 of the earthquake traces. The key results of the FE dynamic simulations, which represent the main
18 contribution of this work over previous studies, are discussed in the following section.

19

1 **Results of the dynamic simulations and discussion**

2

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

12

13 *Wave propagation and acceleration field*

14 The periods of maximum amplification of the seismic wave during its propagation through the soil
15 deposit have been identified normalising the response spectra of the accelerations recorded at
16 ground surface below the silo by the corresponding input ones. The normalised response spectra for
17 the CLS and DLS simulations are reported in Figures 11a and 11b, respectively. In the CLS case,
18 the maximum amplification occurs in the range of periods between 0.6 and 0.9 s, but the first
19 natural period cannot be clearly identified (Figure 11a). The AQP input motion is de-amplified at
20 ground surface, as its dominant frequency is far from the first mode of the system, while the soil
21 deposit tends to amplify the maximum acceleration of the ANT and ASSISI earthquake events at
22 surface. For each input motion, higher amplification always occurs when the seismic action hits the
23 empty silo. Very similar results are obtained in the case of DLS simulations, as shown in Figure
24 11b: the period of maximum amplification can now be recognized at about 0.6 s, representing the
25 first mode period of the soil deposit. Moreover, the dynamic simulations performed using a lower

1 return period (i.e. DLS case) exhibit much higher amplification effects as compared to the CLS
2 analyses, even though they are characterised by lower energy content. The smaller shear strain
3 amplitude induced in the soil deposit by the less extreme events enhances the transmission of the
4 high frequencies of the seismic signal and increases the amplification of the peak accelerations at
5 surface.

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

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

1 *Displacement field*

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

6 In particular, in the case of the ANT#0.313L simulation, the evolution with time of vertical
7 displacements of nodes along the silo axis is shown in Figure 14a. The maximum settlement,
8 recorded at surface, is equal to 0.17 m, while the vertical displacement obtained at the contact line
9 between L_3 and L_4 is negligible (0.005 m). The foundations settlement computed during the seismic
10 action is considerably higher than the one accumulated during the first applied load of the silo (75
11 mm). This is related to a higher degradation of structure induced in the cemented clayey soils by the
12 earthquake action, as discussed later. The computed displacement time histories become constant
13 immediately after the end of the earthquake (i.e. after 30 s) in all the monitored nodes, indicating a
14 stable behaviour of the foundation after the seismic action.

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

24 A significant advantage of the fully-coupled nonlinear approach adopted in this study is that it is
25 based on effective stress analysis, accounting for the dynamic interaction between the soil skeleton

1 and the pore fluid. The excess pore water pressures cumulated inside the soil deposit throughout the
2 shaking are shown in Figure 14c for different depths along the silo axis. The clayey layers behave
3 essentially in undrained conditions during the shaking, cumulating high positive pore water
4 pressures which increase with depth. This can be attributed to the low soil permeability which does
5 not allow dissipation of pore water pressures in a short interval of time, such as the duration of the
6 seismic action and the following 10 seconds. Positive excess pore pressures are also recorded at the
7 bottom of the silty sand layer, with a maximum value of 76 kPa at the depth of 50 m, while the pore
8 pressures oscillate around zero in the top granular material due to the proximity to the drained
9 hydraulic boundary.

10 Figures 15a and 15b show, respectively, the contour lines of vertical and horizontal displacements
11 at the end of the dynamic action. The overall behaviour of the soil deposit in terms of vertical
12 displacement (Figure 15a) is the one typically observed in the case of rigid footings (i.e. the
13 settlements are almost constant below the foundation). Maximum positive displacements (i.e.
14 directed upwards) of about 0.10 m are recorded outside the foundation area at ground surface.
15 Figure 15b confirms the non-symmetric response in terms of permanent horizontal displacement
16 shown in Figure 14b. Moreover, it can be observed how the maximum values (of about 0.30 m) are
17 obtained below the corners of the footing and not along its centre line, at a depth of 15 m below
18 ground surface (i.e. in layer L_2).

19 The contour lines of excess pore pressures obtained at the end of the earthquake action are shown in
20 Figure 15c. Positive values increasing with depth are recorded below the foundation in L_2 , up to a
21 maximum of about 150 kPa. This is associated with the observed accumulation of plastic strains and
22 the corresponding loss of structure induced by the shaking at the interface between L_2 and L_3 , due to
23 the high difference between the two cemented clayey soils in terms of strength and initial degree of
24 structure (see Table 3). Positive excess pore water pressures are also recorded along the silo axis at
25 the interface between L_3 and L_4 , with a maximum value of 70 kPa, and in the bottom Mohr-

1 Coulomb layer, reaching the maximum value of 120 kPa at bedrock level.

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

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

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

14

15 *Destructuration and stress-strain behaviour*

16 To better understand the effects of the dynamic action on the behaviour of the silo, the evolution of
17 degree of structure with time has been monitored in the cemented soils during the application of the
18 different input motions. This represents one of the first attempts to investigate the effects of
19 earthquake loading on the induced structure degradation in natural clayey deposits. In particular, the
20 time histories of the *RMW* state variable r recorded along the silo axis during the ANT#0.313L
21 analysis are shown for different depths of L_2 and L_3 in Figures 16a and 16b, respectively. The
22 structure degradation induced with depth by the static load related to the first stocking activity of
23 the silo is negligible in both clayey layers, as the starting value of the state variable r is essentially
24 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
25 induces significant destructuration, especially in the top natural clay where the structure

1 progressively reduces with time and depth (Figure 16a), reaching the maximum loss of structure of
2 54% at the bottom of the layer ($z = 16$ m). Smaller destructuration effects during the shaking are
3 observed in the clayey soil L_3 (Figure 16b) where a maximum loss of structure of about 19% is
4 recorded at the depth of 18 m. Moreover, the time histories of the state variable r remain constant in
5 all the monitored nodes of layers L_2 and L_3 during the 10 seconds following the seismic action. This
6 is consistent with the observed behaviour of the soil deposit described in the previous section in
7 terms of permanent displacements. For comparison, Figures 16c and 16d report the corresponding
8 time histories of the *RMW* state variable r recorded in layers L_2 and L_3 , respectively, when the
9 empty silo is subjected to the same earthquake motion (i.e. during the ANT#0.313U analysis). A
10 similar pattern in terms of structure degradation can be observed in this case, with smaller
11 destructuration occurring in the clay layers overall. When the silo is empty, the maximum predicted
12 loss of structure is equal to 41% in layer L_2 and 16% in layer L_3 . This comparison reveals that
13 higher permanent displacements are induced by the input motion when the silo is fully loaded.
14 Finally, the stress-strain curves recorded during the ANT#0.313L analysis along the axis in the four
15 layers are shown in Figure 17. It can be observed that the clayey soils L_2 and L_3 (where the
16 advanced *RMW* model has been adopted) exhibit a continuous change of stiffness and the
17 accumulation of plastic shear deformations all through the shaking (Figures 17b and 17c). In
18 contrast, the stress-strain curves recorded at the stress points situated at the top and bottom L_1 and
19 L_4 layers are representative of an elastic behaviour, with very small accumulation of plastic strains
20 during the earthquake excitation (Figures 17a and 17d). These results corroborate the hypothesis
21 that the displacements accumulated during the shaking are essentially controlled by the mechanical
22 behaviour of the two natural clay layers L_2 and L_3 .

1 **Conclusion**

2

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

10 In the numerical work undertaken here, three acceleration time histories were selected and scaled
11 from earthquake databases according to the seismic hazard study of the specific site. The dynamic
12 behaviour of the silo was analysed in both full and empty conditions by applying the selected
13 earthquakes at the bedrock of the foundation deposit. The overall behaviour of the silo, in terms of
14 response spectra and profiles of maximum acceleration with depth, indicated a higher amplification
15 of the seismic motion when the earthquake hits the empty structure. However, larger displacements
16 and higher accumulated excess pore water pressures were predicted when the earthquake was
17 applied to a fully loaded silo. This is due to the higher inertia of the shaken mass (i.e. higher applied
18 load) and the significant loss of structure induced with depth by the seismic action, especially in the
19 top cemented soil. The positive excess pore water pressures generated in the clayey layers are
20 indicative of an undrained contractive behaviour associated with a continuous accumulation of
21 plastic strains during the shaking. Additional foundation settlements related to consolidation
22 processes were calculated at the end of the earthquake loading. The performance of the silo footing
23 under dynamic conditions must be considered with care during the design of the structure, as
24 extreme earthquake events can induce ground settlements up to two times larger than those caused
25 by static loads.

1 More generally, numerical modelling of shallow foundations in seismic zones suggests that there
2 are significant advantages in using advanced models which recognise the existence and subsequent
3 loss of structure in natural soils. With the current advances in geotechnical research, it is nowadays
4 possible to undertake sophisticated numerical modelling using advanced soil constitutive models to
5 accurately predict the deformation behaviour of geo-structures under dynamic loading. This will
6 allow the full implementation of seismic performance-based design for geotechnical structures.

7

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11

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1 **Appendix I: Numerical formulation of the finite element code**

2 In the context of FE analysis, assuming that the relative velocity of the fluid phase is negligible, the
3 system of ordinary equation that results from the u - p formulation can be written as follows (Biot
4 1941; Zienkiewicz *et al.* 1999):

$$6 \quad \begin{cases} [M]\ddot{\mathbf{u}} + [C]\dot{\mathbf{u}} + [K]\mathbf{u} - [Q]\mathbf{p} = \mathbf{f}^s \\ [Q]^T \dot{\mathbf{u}} + [S]\dot{\mathbf{p}} + [H]\mathbf{p} = \mathbf{f}^p \end{cases} \quad (1)$$

7
8 where $\mathbf{u}$ is the solid phase displacement vector and $\mathbf{p}$ is the pore fluid pressure vector, $[M]$ is the
9 mass matrix, $[K]$ is the stiffness matrix, $[C]$ is the viscous damping matrix, $[Q]$ is the coupling
10 matrix between the motion and flow equations, $[H]$ is the permeability matrix, $[S]$ is the
11 compressibility matrix, $\mathbf{f}^p$ is the force vector for the fluid phase and $\mathbf{f}^s$ is the force vector for the
12 solid phase. Frequency dependent viscous damping is included via the Rayleigh damping matrix
13 (Clough and Penzien 1993):

$$15 \quad [C] = \alpha[M] + \beta[K] \quad (2)$$

16
17 where the factors α and β are related to the modal damping coefficients according to the
18 relationship:

$$20 \quad \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2D}{\omega_m + \omega_n} \begin{Bmatrix} \omega_m \omega_n \\ 1 \end{Bmatrix} \quad (3)$$

21
22 These coefficients can be calculated by selecting one value of damping ratio D and two
23 frequencies, ω_m and ω_n , outside which damping is larger than the selected value.

24 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 $\{\mathbf{u}_n, \dot{\mathbf{u}}_n, \ddot{\mathbf{u}}_n, \mathbf{p}_n, \dot{\mathbf{p}}_n\}$ have been obtained at time t_n , the integration consists of updating $\{\mathbf{u}_{n+1}, \dot{\mathbf{u}}_{n+1}, \ddot{\mathbf{u}}_{n+1}, \mathbf{p}_{n+1}, \dot{\mathbf{p}}_{n+1}\}$ at the next time step t_{n+1} according to the Generalised Newmark scheme (Katona and Zienkiewicz 1985). In particular, for the solid phase:

$$\begin{aligned}\ddot{\mathbf{u}}_{n+1} &= \ddot{\mathbf{u}}_n + \Delta \ddot{\mathbf{u}}_n \\ \dot{\mathbf{u}}_{n+1} &= \dot{\mathbf{u}}_n + [\ddot{\mathbf{u}}_n + \beta_1 \Delta \ddot{\mathbf{u}}_n] \Delta t \\ \mathbf{u}_{n+1} &= \mathbf{u}_n + \dot{\mathbf{u}}_n \Delta t + 0.5 [\ddot{\mathbf{u}}_n + \beta_2 \Delta \ddot{\mathbf{u}}_n] \Delta t^2\end{aligned}\tag{4}$$

Similarly for the fluid phase:

$$\begin{aligned}\dot{\mathbf{p}}_{n+1} &= \dot{\mathbf{p}}_n + \Delta \dot{\mathbf{p}}_n \\ \mathbf{p}_{n+1} &= \mathbf{p}_n + [\dot{\mathbf{p}}_n + \beta_1^* \Delta \dot{\mathbf{p}}_n] \Delta t\end{aligned}\tag{5}$$

where the coefficients:

$$\begin{aligned}\beta_1 &\geq 0.5 \\ \beta_2 &\geq 0.5(0.5 + \beta_1)^2 \\ \beta_1^* &\geq 0.5\end{aligned}\tag{6}$$

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.

1 Appendix II: Constitutive model formulation

2

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

5

$$6 \quad f_r = \frac{3}{2M_\theta^2} s : s + (p - p_c)^2 - (p_c)^2 = 0 \quad (7)$$

7

8 The bubble surface is written as:

9

$$10 \quad f_b = \frac{3}{2M_\theta^2} (s - s_{\bar{\alpha}}) : (s - s_{\bar{\alpha}}) + (p - p_{\bar{\alpha}})^2 - (Rp_c)^2 = 0 \quad (8)$$

11

12 The structure surface is given by:

13

$$14 \quad F = \frac{3}{2M_\theta^2} [s - (r-1)\boldsymbol{\eta}_0 p_c] : [s - (r-1)\boldsymbol{\eta}_0 p_c] + (p - rp_c)^2 - (rp_c)^2 = 0 \quad (9)$$

15

16 where p_c is the effective stress which defines the size of the reference surface, R is the size of the
17 bubble, M_θ is a dimensionless scaling function for deviatoric variation of the critical state stress
18 ratio, $\boldsymbol{\eta}_0$ a deviatoric tensor controlling the structure, r is the ratio of the sizes of the structure and
19 the reference surfaces, p and s are the mean pressure and deviatoric stress tensor and the symbol ‘:’
20 indicates a summation of products. Since the model describes the response of the soil skeleton, all
21 stresses are effective stresses (the primes have been dropped for simplicity). The dots over symbols
22 indicate an infinitesimal increment of the corresponding quantity, whereas bold-face symbols
23 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:

$$\dot{r} = -\frac{k}{(\lambda^* - \kappa^*)}(r-1)\dot{\epsilon}_d \quad (10)$$

where λ^* and κ^* 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 $\dot{\epsilon}_d$ is assumed to have the following form:

$$\dot{\epsilon}_d = \left[(1-A)(\dot{\epsilon}_v^p)^2 + A(\dot{\epsilon}_q^p)^2 \right]^{1/2} \quad (11)$$

where A is a non-dimensional scaling parameter and $\dot{\epsilon}_q^p$ and $\dot{\epsilon}_v^p$ 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, $\dot{\epsilon}_v^p$, given by:

$$\frac{\dot{p}_c}{p_c} = \frac{\dot{\epsilon}_v^p}{\lambda^* - \kappa^*} \quad (12)$$

If a stress increment requires movement of the bubble relative to the structure surface, the following kinematic hardening is invoked:

$$\dot{\bar{\alpha}} = \dot{\hat{\alpha}} + \frac{\dot{p}_c}{p_c}(\bar{\alpha} - \hat{\alpha}) + \mu(\sigma_c - \sigma) \quad (13)$$

2

3 where $\bar{\alpha}$ and $\hat{\alpha} = p_c [r\mathbf{I} + (r-1)\boldsymbol{\eta}_0]$ denote the locations of the centre of the bubble and structure
4 surface respectively, σ_c is the conjugate stress and μ is a positive scalar of proportionality. It
5 should be noted that the centre of the structure surface and the deviator of $\hat{\alpha}$ represents the
6 anisotropy of the soil due to structure. The deviator of $\hat{\alpha}$ therefore degrades to zero as r degrades to
7 unity.

8 The plastic modulus H is assumed to depend on the distance between the current stress and the
9 conjugate stress and is given by:

10

$$H = H_c + \frac{Bp_c^3}{(\lambda^* - \kappa^*)R} \left(\frac{b}{b_{\max}} \right)^{\psi} \quad (14)$$

12

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

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

18

$$K = \frac{p}{\kappa^*} \quad G = \frac{3(1-2\nu)}{2(1+\nu)} K \quad (15)$$

20

21 where ν is a constant Poisson's ratio.

Notation

A	parameter controlling relative proportion of distortional and volumetric destructure
B	stiffness interpolation parameter
b	normalised distance between bubble and structure surface
$b_{\max}$	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
$\mathbf{I}$	second rank identity tensor
K	bulk modulus
k	parameter controlling rate of loss of structure with damage strain
M_θ	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
$\mathbf{s}$	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
ε_v^p	volumetric strain
ε_q^p	deviatoric strain
ε_d	damage strain

γ cyclic shear strain amplitude

$\bar{\gamma}$ unit weight

η_0 dimensionless deviatoric tensor (anisotropy of structure)

κ^* slope of swelling line in $\ln v : \ln p$ compression plane

λ^* slope of normal compression line in $\ln v : \ln p$ compression plane

μ positive scalar of proportionality

ν Poisson's ratio

σ effective stress tensor

σ_c conjugate stress

ψ stiffness interpolation exponent

$a_{\max}$ 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

α, β modal damping coefficients

$\beta_1, \beta_2, \beta_1^*$ generalised Newmark parameters

λ annual frequency of exceedance

ω angular frequency

<i>Limit State</i>	<i>Probability of exceedance in 50 years P (%)</i>	<i>Return period T_R (years)</i>	<i>Annual frequency of exceedance λ (years⁻¹)</i>	<i>Maximum acceleration $a_{\max}$ (g)</i>
CLS	5	975	0.0010	0.313
DLS	63	50	0.0200	0.097

Table 1 Results of the seismic hazard study at the site of the silo

<i>Station</i>	<i>Earthquake</i>	<i>Component</i>	<i>Magnitude</i> <i>M_W</i>	<i>Arias Intensity</i> <i>I_a</i> (m/s)	<i>Epicentral</i> <i>distance</i> (km)	<i>Duration</i> <i>T₉₀</i> (s)	<i>a_{max}</i> (g)	<i>v_{max}</i> (m/s)
Antrodoco (ANT)	L'Aquila Mainshock 2009	NS	6.3	0.0176	23.0	21.32	0.026	0.013
L'Aquila - V. Aterno (AQP)	L'Aquila Aftershock 2009	NS	4.6	0.0518	0.73	2.12	0.156	0.029
Assisi - Stallone (ASSISI)	Umbria-Marche 1997	EW	6.0	0.2349	21.4	4.13	0.188	0.050

Table 2 Main characteristics of the selected seismic actions

	$\bar{\gamma}$ (kN/m ³)	λ^*	κ^*	M_θ	R	B	ψ	r_0	A	k
L ₂	16.6	0.11	0.0023	1.55	0.4	11	2.5	9.5	0.2	1.0
L ₃	18.3	0.11	0.0016	1.40	0.4	11	2.5	5.2	0.2	0.7

Table 3 *RMW* model parameters for layers L₂ and L₃

<i>Name</i>	a_{bed} (g)	<i>settlement</i> (mm)	a_{ground} (g)	$a_{\text{ground}}/a_{\text{bed}}$
ANT#0.313L	0.313	170	0.455	1.45
AQP#0.313L	0.313	8.4	0.185	0.59
ASSISI#0.313L	0.313	32.0	0.415	1.32
ANT#0.313U	0.313	46.0	0.686	2.19
AQP#0.313U	0.313	2.0	0.358	1.14
ASSISI#0.313U	0.313	8.9	0.698	2.23
ANT#0.097L	0.097	37.0	0.180	1.86
AQP#0.097L	0.097	2.5	0.109	1.12
ASSISI#0.097L	0.097	6.8	0.168	1.73
ANT#0.097U	0.097	9.4	0.367	3.78
AQP#0.097U	0.097	0.1	0.116	1.19
ASSISI#0.097U	0.097	1.4	0.258	2.66

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 L₂

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 L₂

Standard boundaries in static analyses
Tied-nodes in dynamic analyses

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 in layer L_3 ; c) during the ANT#0.313U analysis in layer L_2 ; d) during the ANT#0.313U analysis in layer 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