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SIMULATION OF SHAKING TABLE TESTS TO STUDY SOIL-STRUCTURE INTERACTION BY MEANS OF TWO DIFFERENT CONSTITUTIVE MODELS

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ABSTRACT

The paper presents the main results of a FEM 3-D model reproducing a physical model subjected to shaking table tests. The tests, performed at the EERC laboratory of Bristol University, have been simulated by means of a new numerical model based on a recent constitutive model characterized by isotropic and kinematic hardening and devoted to granular soil.
The shaking table tests have been performed using: a six-degree of freedom shaking table; a shear-stack; a scaled one-storey steel frame; the Leigthon Buzzard Sand. The tests have been characterized by 11 shaking runs.
As regards the 3-D numerical modeling, the linear elastic material has been considered for the structure, instead the soil has been modeled both with a cap-hardening Drucker-Prager model, often implemented in commercial codes, and with the above mentioned new constitutive model, implemented in the utilized FEM code by the Research Group of Catania University.
Thanks to the great quantity of experimental data, the power of the proposed numerical model in simulation/prediction of dynamic soil-structure interaction can be verified and compared with the capability of other numerical models based on simpler constitutive models.

INTRODUCTION

The study of the dynamic soil-structure behaviour is extremely necessary in order to correctly predict the behaviour of structures during earthquakes, but it is very complex because of the difficulties of performing full scale physical model tests and of the lack of implementation into numerical codes of constitutive models properly suitable to reproduce dynamic soil behaviour.
Among the possible investigation approaches, experimental techniques based on scaled physical models, and in particular shaking table tests represent a very useful tool to study the soil-structure interaction and a landmark for numerical analyses, which are very powerful to predict the behaviour of scaled and/or not scaled structures, but they need of experimental results to be calibrated and validated [Taylor and Crewe, 1996; Gajo and Muir Wood, 1997; Maugeri et al., 2000, Novità 2001; Biondi and Massimino, 2002; Massimino, 2005].
Shaking table tests have the great advantage to be characterized by known initial and boundary conditions . Furthermore, they allow the user to perfectly control the time of application and the nature of the dynamic input. But, unfortunately, shaking table tests are frequently performed on structures directly fixed on the shaking table [Payen et al, 2006], ignoring the fundamental role played by the propagation of the seismic waves through the soil. The utilized equipment has the great advantage to easily include a granular soil deposit.

For what concerns the challenge of choosing constitutive models for use in simulating the behaviour of geotechnical systems under dynamic loading, a significant number of numerical codes [FLAC, 1996; PLAXIS, 1998; STRAUS-7, 1999 and ADINA, Bathe, 1996], which can be used for one-, two-, and three-dimensional problems, are now available for site response analyses, as well as for the study of any geotechnical structure.
Nevertheless, very simple soil constitutive models (elastic-linear, elastic-perfectly plastic Mohr-Coulomb or Drucker-Prager, Cam-Clay, etc.) are implemented in these commercial codes. Actually, geotechnical materials show a great variety of behaviour when subjected to different conditions, so no mathematical model can completely describe this complex behaviour. Each soil model is aimed at a certain class of phenomena, captures their essential features, and disregards what is considered to be of minor importance in that class of applications.
Several studies have shown that when shear strains in the soil are small (which typically occurs when the ground motions are weak or the site consists of stiff soils), it is possible to use the elastic-linear model; for small to medium strains it is convenient to use equivalent linear or nonlinear models [Kodner and Zelasko, 1963; Desai, 1971; Breth et al., 1973; Daniel et al., 1975]. Elastic-plastic models [Drucker et al., 1952; Roscoe et al., 1958; Schofield and Wroth, 1968; Lade, 1977; Nova and Wood, 1979] can more accurately capture

response for sites that experience medium to high strains. For high to very high strains (strong motions affecting soft soil sites) it is necessary to use isotropic-kinematic hardening constitutive models [Gajo and Muir Wood, 1999a, 1999b], incrementally nonlinear models [Darve, 1978, 1990] or hypoplastic models [Chambon et al., 1994; di Prisco et al., 2003, 2006].

According to what has been said up to this point, the implementation of more appropriate and realistic soil constitutive models in numerical codes should be encouraged.

The paper shows the application of an ‘adequate complex’ FEM numerical model to the simulation of shaking table tests on a scaled physical model consisting of a steel frame with shallow foundations resting on a sand deposit confined in a shear stack. The experimental results, in terms of acceleration and displacement of both the structure and the soil, are compared with numerical results obtained both by a common model implemented in commercial codes, which is a cap-hardening Drucker-Prager model, and by a recent elasto-plastic constitutive model with isotropic and kinematic hardening implemented in the utilized FEM code by the Research Group of Catania University [Abate et al., 2008].

DESCRIPTION OF SHAKING TABLE TESTS

The physical model consists of a scaled steel frame resting on a deposit of Leighton Buzzard sand 0.90 m deep, pluviated into a shear stack of dimensions 5 m by 1 m by 1.2 m [Crewe et al., 1995].

The steel model has been designed in order to reproduce a full-scale 2-storey building, by scaling down the geometric properties of the prototype structure using a length scaling factor equal to 6; the other quantities have been scaled as suggested by Iai & Sugano [1999], as extensively reported in Novità, 2001. The steel model is a one storey frame (Fig. 1) characterized by a longitudinal frame span equal to 1.11 m, a transverse span equal to 0.76 m and a storey height equal to 1.30 m. The beams have hollow sections of 50x50x3.2 mm. The columns have hollow sections of 40x40x4 mm. The solid shallow foundation section is 190 mm wide and 3 mm thick. On the roof of the steel model a surcharge of 1.96 kN has been applied by means of eight steel blocks, uniformly distributed. The total weight of the steel frame, not considering the surcharge, is equal to 1.19 kN. The steel model has been located in the middle of the shear stack with a foundation embedment of 100 mm. More details on the steel frame can be find in Novità [2001].

The soil utilized for the test is the dry Leighton Buzzard sand, which has been used for many years for shaking table tests at the EERC of Bristol University [Taylor et al., 1994; Taylor and Crewe, 1996; Gajo and Muir Wood, 1997, 1998; Paolucci and Pecker, 1997; Carafa et al., 1998; Maugeri et al., 1999; Maugeri et al., 2000, Novità 2001, Dietz and Muir Wood, 2007]. It is an uncemented sand with sub-rounded particles, whose main properties are reported in the table 1. In particular, for the estimation of G the following procedure has been used.

Table 1. Leighton Buzzard Sand: some geotechnical properties.

D_{50} (mm)	$C=D_{60}/D_{10}$	G_s	γ_{dmax} (kN/m^3)	γ_{dmin} (kN/m^3)	e_{max}	e_{min}
0.94	2.128	2.679	17.94	15.06	0.79	0.49

Firstly, the shear modulus G_0 at very low strain level could be computed using the following Hardin and Drnevich [1972] expression:

$$G_0(d) = \frac{3230(2.973 - e)^2 \sqrt{\sigma_m^{'}}}{1 + e} \qquad (1)$$

where e is the void ratio and σ_m’ the mean effective confining stress. It is assumed e = 0.6, according to Dietz and Muir Wood [2007]; while σ_m’ is computed at half of the sand deposit depth assuming K_0 = 0.45 as suggested by Stroud [1971]. This procedure has leaded to G_0 = 25 MPa.

The operational G to be used in the present analysis is obtained using the Cavallaro et al. [2001] degradation law of G/G_0 versus the shear strain γ increasing:

$$\frac{G(\gamma_{yz})}{G_0} = \frac{1}{1 + 20 \cdot (\gamma(\%))^{0.9}} \qquad (2)$$

considering the measured maximum shear strain. In particular, for the shaking run XI, discussed in the following, γ = 2% has been measured, which leads, due to expression (2) to G/G_0 = 0.026 and thus to G = 0.65 MPa. The corresponding value of the damping ratio ξ = 25 % has been evaluated according to Dietz and Muir Wood [2007]. From the back-analysis of the shaking table tests G has been evaluated equal to 1.15 MPa and ξ equal to 20%.

The sand has been pluviated (Fig. 2) into the shear stack (Fig. 3) maintaining the height of deposition equal to 60 cm in order to obtain a relative density equal to D_R=50 % and a shear strength angle equal to φ = 40° according to the following expressions reported by Cavallaro et al. [2001]:

$$D_r(\%) = 0.555 \cdot h_d(cm) + 14.7 \qquad (3)$$

$$\varphi(°) = 0.238 \cdot D_r(\%) + 28.4 \qquad (4)$$

The shear stack (Fig. 3) is formed of a series of rectangular rings of aluminum box section each of which is linked to the rings above and below through neoprene blocks, which give flexibility to the longitudinal containment in order to reproduce as closely as possible free field conditions [Gajo & Muir Wood, 1998]. The long sides of the shear stack are lined with lubricated neoprene sheets in order to reduce lateral friction; the short sides of the shear stack are lined with neoprene sheets covered with sand in order to be able to mobilize necessary complementary shear stresses.

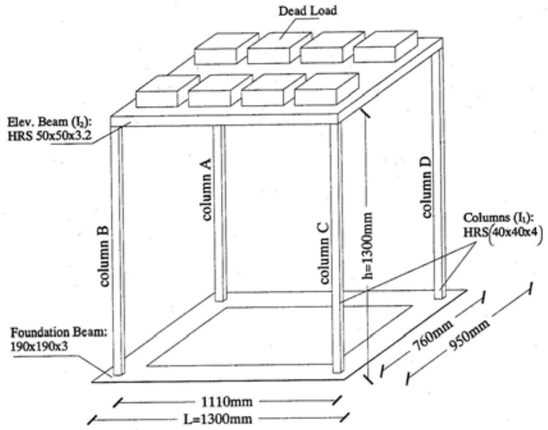

Fig. 1. The steel frame.

Fig. 2. Sieve for the deposition of the Leighton Buzzard sand into the shear stack.

Fig. 3. Shear stack: short side [Biondi e Massimino, 2002].

The whole system (Fig. 4) is placed on the six-degree of freedom shaking table (of dimensions 3m by 3m) available at the Earthquake Engineering Research Centre (EERC) of the University of Bristol (Fig. 5). It consists of a cast aluminum seismic platform capable of carrying a maximum payload of 21 tones. The platform is mounted on a 100 tone isolating block and is driven horizontally and vertically by eight 300mm stroke, 70 kN servo hydraulic actuators giving simultaneous full control of motion of platform in all the six degrees of freedom (it has been observed that even for intended unidirectional shaking it is necessary to control all six degrees of freedom in order to avoid undesired parasitic motions). Hydraulic power for the actuators is provided by five pairs of hydraulic pumps capable of delivering 900 litre/min at a working pressure of 230 bar. The table operates up to 100 Hz. Motion amplitude can be varied by the application of a scalar multiplier to the excitation waveform [Dietz and Muir Wood, 2007].

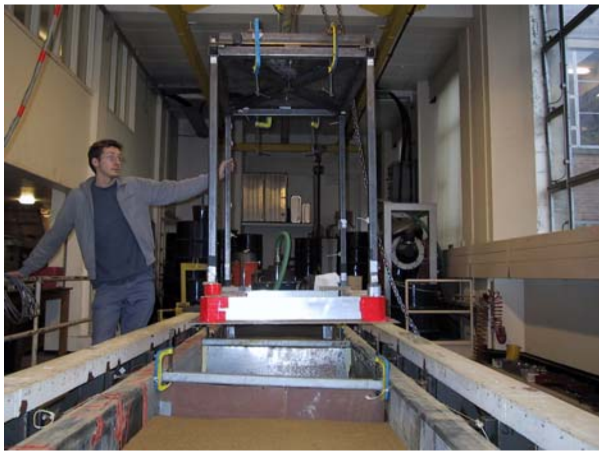

Fig. 4. Complete soil-structure physical model.

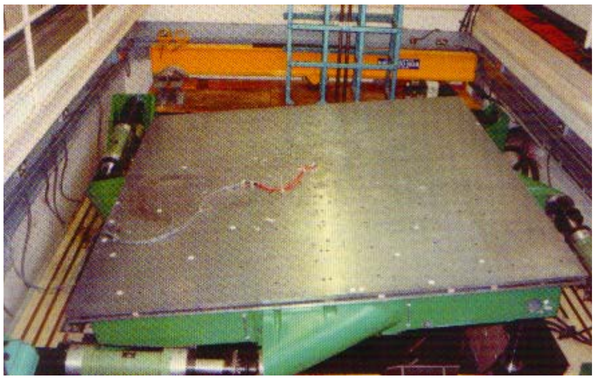

Fig. 5. Shaking table used for the experimental analysis.

The model has been monitored by means of 23 accelerometers and 15 displacement transducers (Figs. 6 and 7). In particular, there are:

- 13 Setra model 141 piezo-electric and unidirectional accelerometers (Fig. 6 and 8), characterized by an operational frequency range of 0÷300 Hz, 3 of which put on the shaking table, in order to control the seismic input according to the longitudinal (S1), transversal (S2), and vertical (S3) direction; 3 of which put on the shear stacks walls (S4, S5, S6) and 7 put on the steel frame (from S7 to S13);

- 10 Dytran accelerometers model 3101A with an operational frequency range of 0.5÷5000 Hz, put in the sand according to two different depths (Fig. 6 and 9): 40 cm (D27, D28, D29, D30) and 80 cm from the bottom of the shear-stack (D31, D31 bis, D32, D32 bis, D35, D35 bis), in order to observe the variation of acceleration amplitude in the sand with the depth;
- 3 Indikon no-contact magnetic displacement transducers (Fig. 7 and 10), put on the soil surface, in order to record the soil surface vertical displacements (Ind24, Ind25, Ind26);
- 10 Celesco displacement transducers (Fig. 7 and 11), put on the steel frame, in order to record the column horizontal displacements (C20, C21, C22, C23) and the foundation settlements (from C14 to C19).

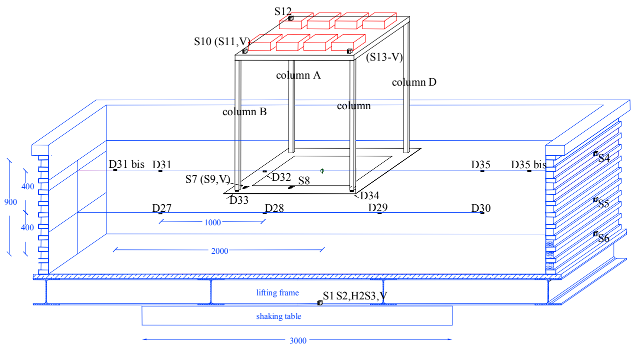

Fig. 6. Distribution of Setra and Dyran accelerometers [Biondi & Massimino, 2002].

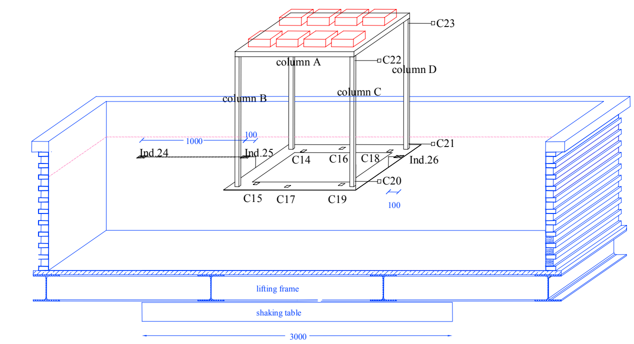

Fig. 7. Distribution of Indikon and Celesco displacement transducers [Biondi & Massimino, 2002].

The tests have been performed by applying a series of sine-dwell horizontal displacement time-histories in the long direction of the shear-stack, with a constant frequency equal to 2 Hz, which is adequately lower than the resonant frequency of the soil-structure system estimated as 3.5 Hz [Biondi and Massimino, 2002].
The input horizontal acceleration time-history consists of a series of sinusoidal cycles with variable amplitude, initially building up to the chosen maximum amplitude (PHA) over 5 cycles, then held constant over 20 cycles and then reduced to zero over a further 5 cycles. Eleven shaking runs, characterized by different PHA have been performed (Table 2).
Fig. 12 shows the input horizontal acceleration time-history for the last run XI.

Fig. 8. The S7 accelerometer put on the foundation.

Fig. 9. Location of the Dytran accelerometers at 40 cm from the shear stack bottom.

Fig. 10. The Ind26 displacement transducers.

Fig. 11. The Ind26 displacement

NUMERICAL MODELING

The described physical model has been modeled by a 3-D FEM model, using the ADINA code.
In particular, the soil deposit and the foundation have been modeled using 20-noded 3-D solid elements, the steel frame has been modeled using 2-noded Hermitiam beam elements, and the steel roof plate has been modeled using shell elements. Totally there are 5001 nodes and four groups of elements: the soil, the foundation, the steel frame and the steel roof plate.

Table 2. Peak horizontal acceleration (PHA) recorded by S1.

RUN	PHA (g)
I	0.08
II	0.10
III	0.15
IV	0.16
V	0.20
VI	0.31
VII	0.35
VIII	0.40
IX	0.45
X	0.49
XI	0.53

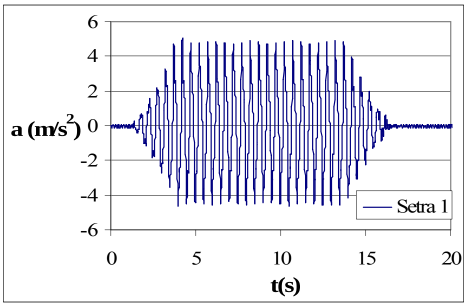

Fig. 12. Input horizontal acceleration time-history for run XI.

Fig. 13 shows the undeformed mesh and the imposed boundary and load conditions. In particular, all the nodes that represent the shear-stack base have been blocked along the z-direction; the nodes of the two short walls of the shear stack are linked by "constrain equations" that impose the same horizontal y-translation; the nodes of the two long walls have been blocked along the x-direction. Three load conditions have been applied to the FEM model: a "mass proportional load" has been applied to the whole system, in order to take into account the weight of the present materials; the surcharge applied on the roof of the steel frame has been simulated by means of a uniform load; finally, the same sinusoidal input motion, applied during the tests as a horizontal displacement time history, has been applied at the bottom boundary of the finite element model.
Furthermore, the Rayleigh damping factors α and β have been evaluated according to the relations with the soil damping ratio ξ and the input frequency ω:

$$\alpha = \xi \cdot \omega \tag{5}$$

$$\beta = \xi / \omega \tag{6}$$

For what concerns the adopted constitutive models, the structure (steel frame and foundation) has been modeled using the linear elastic constitutive model with the steel parameters: E=2.10·108 kN/m^2, ν=0.3 and ρ=7.85 kNs^2/m^4. The soil has

been modeled with a cap hardening Drucker-Prager model available in the ADINA code [Bathe, 1996], as well as with the distortional hardening Severn-Trent sand model [Gajo and Muir Wood, 1999 a-b].
The soil constitutive model parameters are reported in Table 3. Some of these are estimated directly by means of laboratory tests. The remainder (W, D', R' and L' for the cap-hardening Drucker-Prager model, and R, A, B, k and k_d for the Severn-Trent model) are fixed using an error and trial procedure.

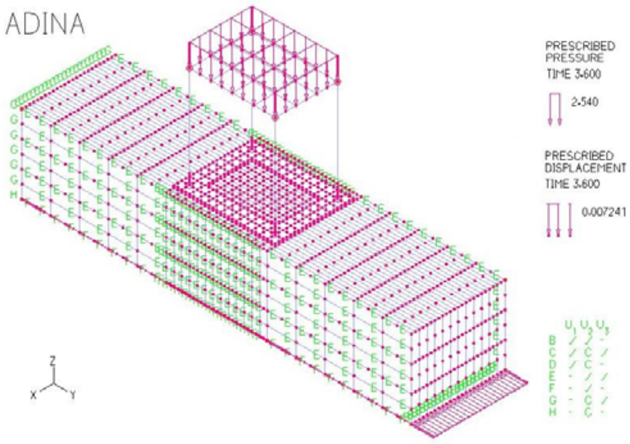

Fig. 13. Adopted FEM model.

Table 3. Constitutive model parameters.

Parameters	**Drucker-Prager**	**Severn-Trent**
G (MPa)	1.15	1.15
ν	0.3	0.3
ρ (kNs^2/m^4)	1.54	1.54
α	0.204	-
k'	0.3	-
W	-0.13	-
D'	$7.25 \cdot 10^{-4}$	-
L'	0	-
R'	2	-
λ	-	0.03
v_λ	-	1.969
ϕ_{cv} (°)	-	40
R	-	0.1
A	-	1.2
k_d	-	1.0
B	-	0.0030
k	-	2.0

COMPARISON BETWEEN EXPERIMENTAL AND NUMERICAL RESULTS

Due to the great quantity of experimental and numerical results available, only the most significant are presented in the following. In particular, only the results related to the shaking run XI, which is one of the most significant, are shown. Moreover, all the instruments able to record the horizontal accelerations along the short side of the shear stack, as well as the vertical accelerations, are not taken into account, considering that the input motion is applied along the long side of the shear stack. As regards the other accelerometers only the accelerometers D28, D32, S7 and S10 are taken into account to investigate on amplification/de-amplification phenomena along a fixed vertical alignment. Furthermore, the records of the accelerometer D35 are compared with those of the accelerometer D32 to compare the soil response in free-field conditions (D35) with the soil response under the steel frame (D32). Finally, the records of the displacement transducers C14, C16 and C18 are similar to those of the displacement transducers C15, C17 and C19, as well as the records of the displacement transducers C21 and C23 are very similar to those of the displacement transducers C20 and C22, because the input motion is applied along the long side of the shear stack and the steel frame is symmetrically loaded, thus none torsional movement occurs. So, the C14, C16, C18, C21 and C23 records are not taken into account.

Numerical simulation by the cap-hardening Drucker-Prager model

Using the cap-hardening Drucker-Prager model a quite good agreement between experimental and numerical results has been obtained in terms of accelerations (Fig. 14). The experimental results show an acceleration de-amplification at 40 cm from the shaking table (D28), then an acceleration amplification approaching to the steel frame foundation (D32 and S7). The numerical simulation seems to capture quite well the records of the accelerometers D32, S7 and S10, while it does not capture the de-amplification recorded by the accelerometer D28. Comparing the experimental results regarding the accelerometer D32 (Fig. 14), which is located underneath the steel frame foundation , with those regarding the accelerometer D35 (Fig. 15), which is far from the frame, i.e. it is in free-field conditions, it is possible to see a great difference. This result confirms once more the importance to take into account soil-structure interaction. This result is in some way captured also by the numerical simulation. Numerical results close to the experimental ones have been also obtained in terms of horizontal displacements (Fig. 16).
An important discrepancy between experimental and numerical results exist considering the vertical displacements (Figs. 17 and 18).
In particular, Fig. 17 shows a heave of the sand surface level around the foundation. This numerical result, opposite to the experimental one (the sand surface level actually fell), is connected to the fact that the stress-strain behaviour and the dilatancy of sand are related both to the relative density and to the effective mean stress. Unfortunately, the cap-hardening Drucker-Prager model does not take into account these aspects of sand behaviour, considering only the stress ratio dependency.

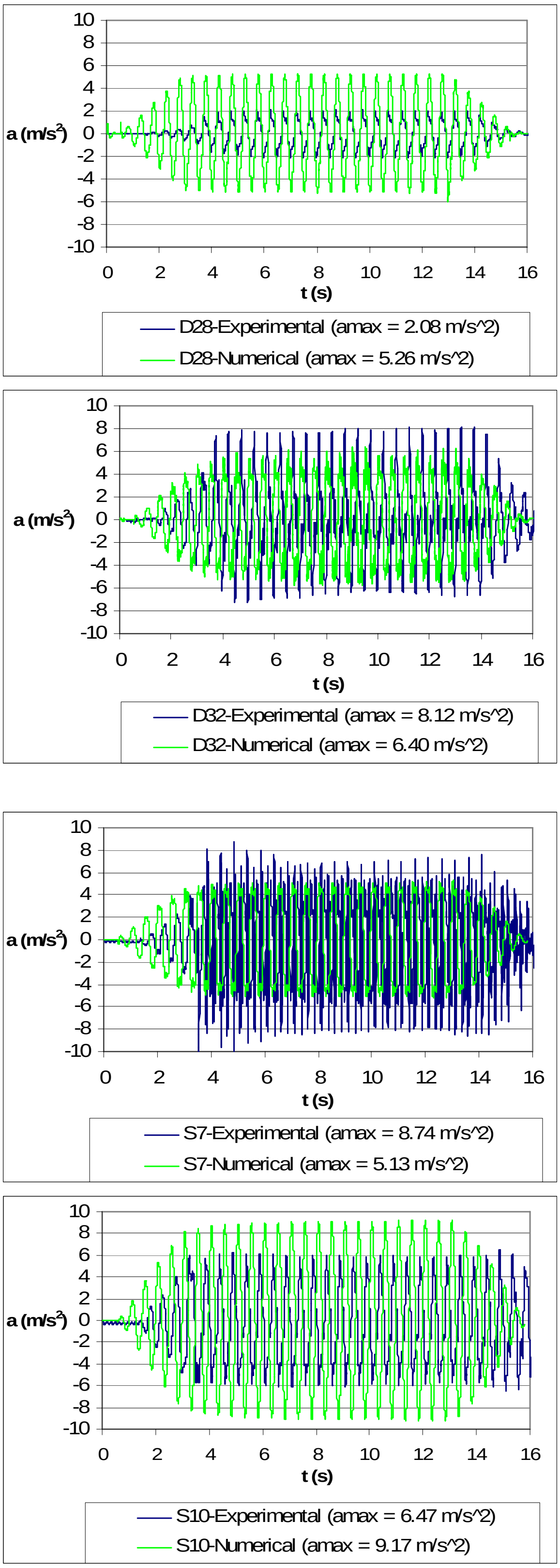

Fig. 14. Accelerations along a specific alignment; the numerical results are obtained using the cap-hardening Drucker-Prager model.

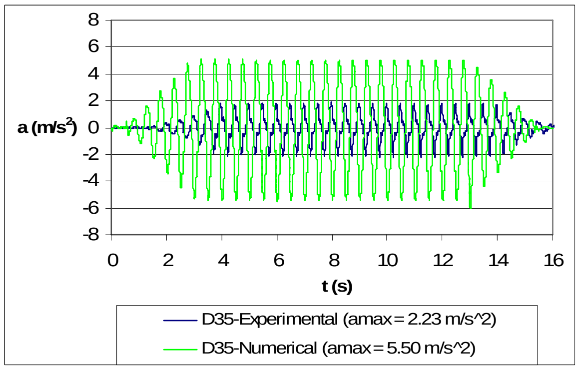

Fig. 15. Acceleration at the accelerometer D35 (soil surface in free-field conditions); the numerical results are obtained using the cap-hardening Drucker-Prager model.

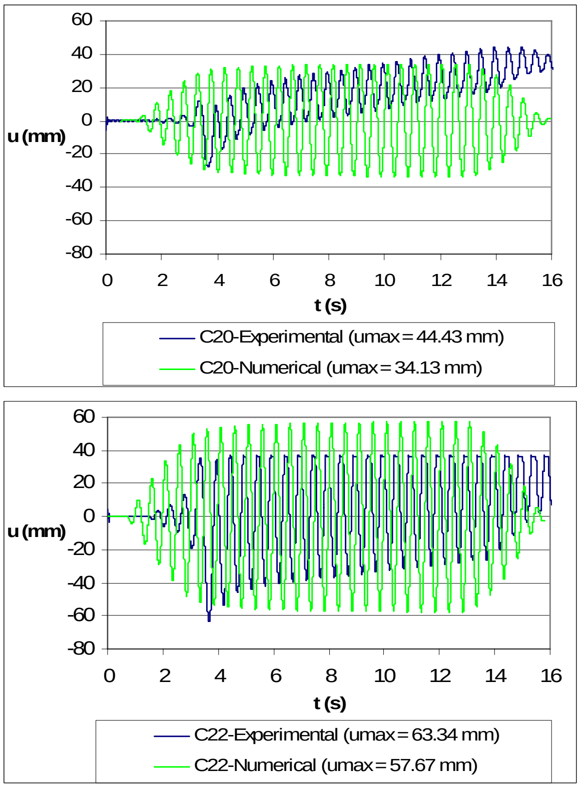

Fig. 16. Steel frame horizontal displacements; the numerical results are obtained using the cap-hardening Drucker-Prager model.

The shaking phenomenon leads, in the present case, to a sand densification and to effective mean stress levels that cause sand settlements along the all sand surface. This phenomenon is somewhat described using Severn-Trent sand, in which the stress-strain response depends on both the specific volume (and thus the relative density) and the mean stress.

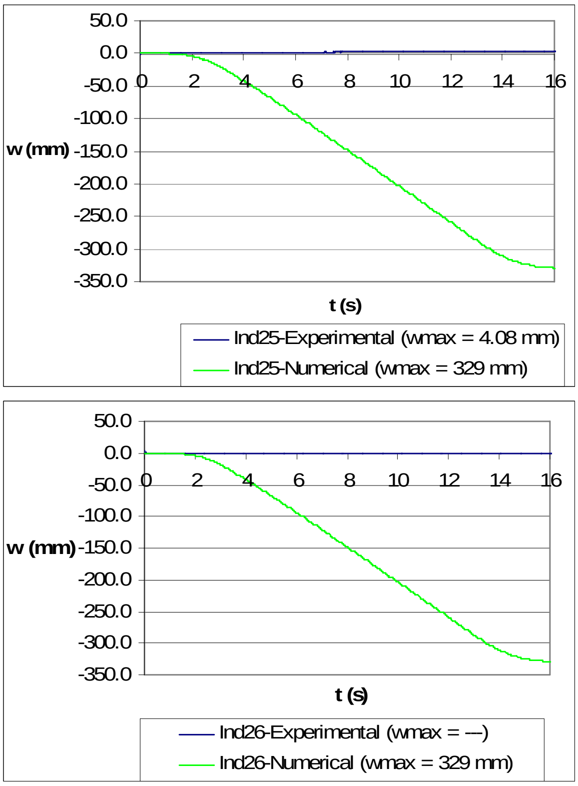

Fig.17. Soil surface settlements; the numerical results are obtained using the cap-hardening Drucker-Prager model.

<u>Numerical simulation by the Severn-Trent model</u>

On the contrary of what observed with the cap-hardening Drucker-Prager model, using the Severn-Trent model the soil de-amplification at 40 cm from the shaking table (D28) is perfectly captured. Moreover, a good agreement between experimental and numerical results has been obtained for the frame roof (S10). Nevertheless, significant discrepancies exist considering the accelerations below (D32) and at (S7) the steel frame foundation (Fig. 19).
This could be due to some up-lift and slip phenomena occurred at the soil-foundation interface and not captured by the finite element model. By the way, a very good agreement exists between the experimental and numerical results if we consider the accelerometer D35 (Fig. 20), i.e. the free-field conditions.
Also the steel frame horizontal displacements predicted numerically (Fig. 21) are less close to the experimental ones in comparison with those predicted with the cap-hardening Drucker-Prager model.
Nevertheless, very interesting results have been achieved in terms of vertical displacements (Figs. 22 and 23). In particular, it is very important to underline that with the Severn-Trent model it has been possible to simulate the soil surface settlements (Fig. 22), due to sand densification during shaking. Furthermore, from Fig. 23, it si possible to see that there is an extremely good agreement between experimental and numerical results at C15. This agreement lightly deteriorates going from C15 to C19. This could be due to an unintended eccentricity of the surcharge towards columns C and D.

CONCLUSION

To study a soil-structure interaction a physical model of a steel frame resting on sand soil has been tested using a shaking table. The experimental results have been reported in detail in terms of horizontal accelerations and horizontal and vertical displacements, by means of many accelerometers and displacement transducers, monitoring the behaviour of the foundation soil and the steel structure.

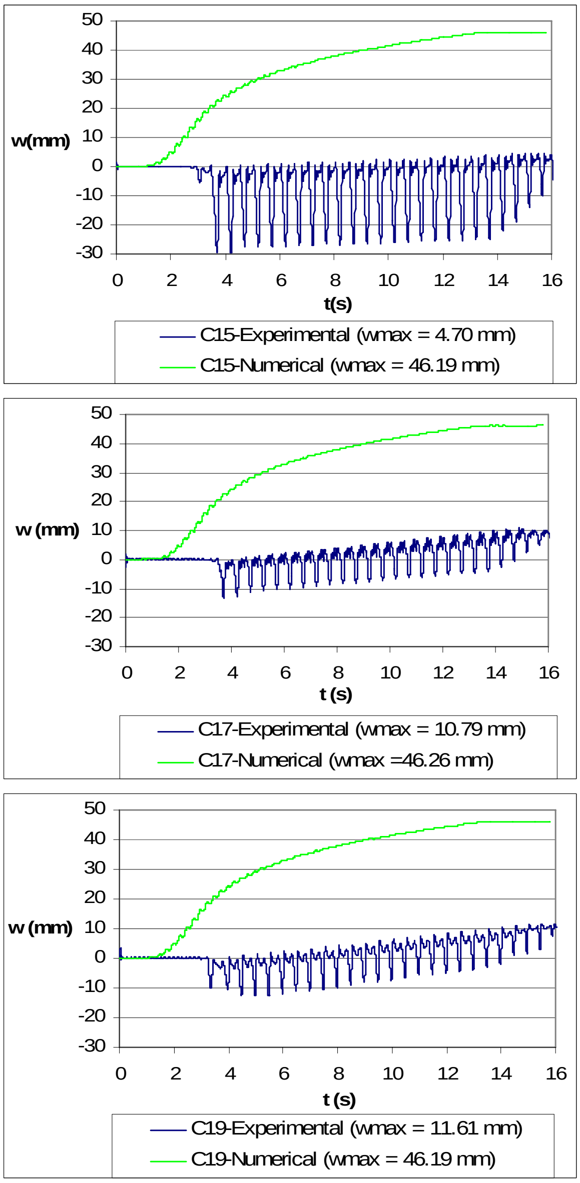

Fig. 18. Foundation settlements; the numerical results are obtained using the cap-hardening Drucker-Prager model.

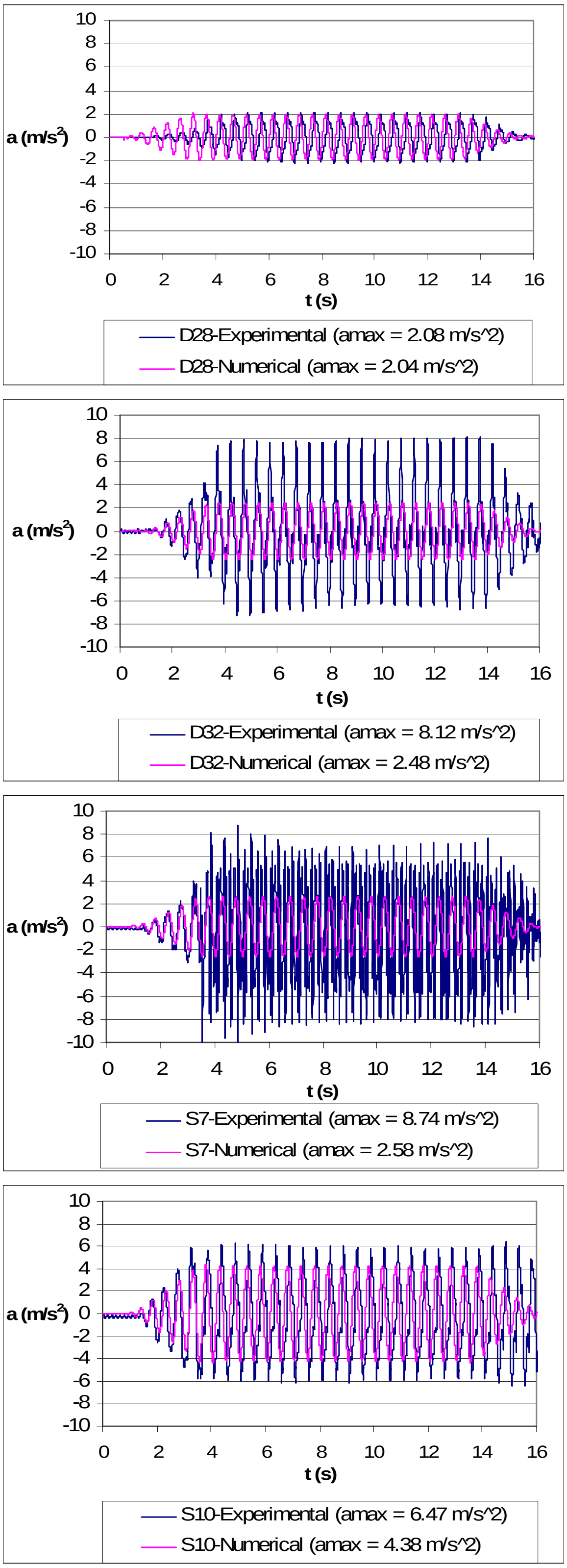

Fig. 19. Accelerations along a specific alignment; the numerical results are obtained using the Severn-Trent model.

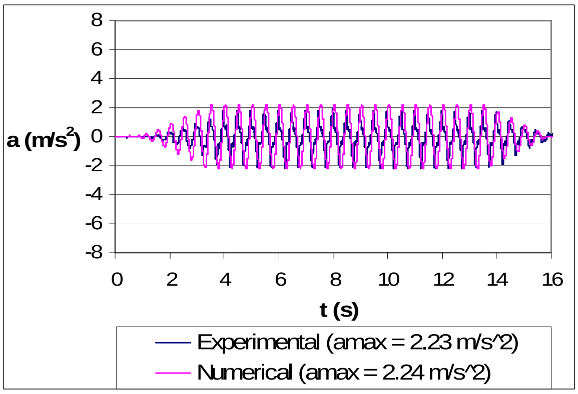

Fig. 20. Acceleration at the accelerometer D35 (soil surface in free-field conditions); the numerical results are obtained using the Severn-Trent model.

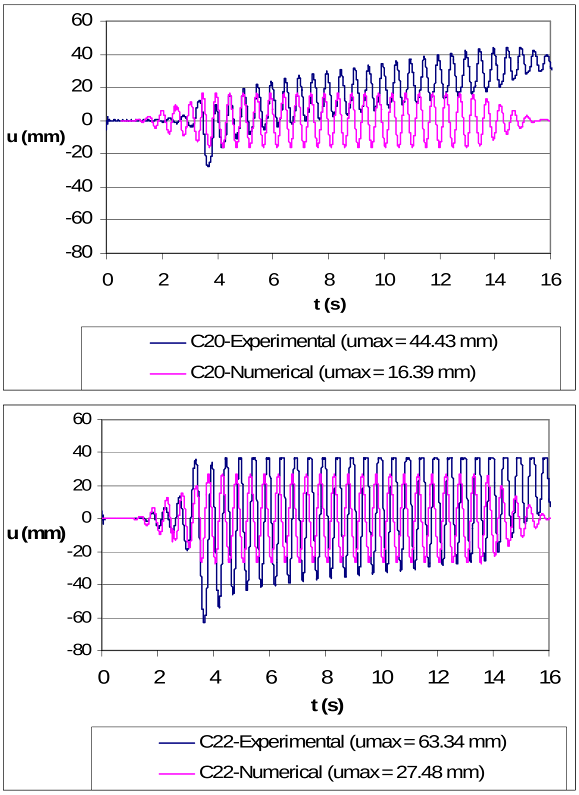

Fig.21. Steel frame horizontal displacements; the numerical results are obtained using the Severn-Trent model.

For the simulation of the shaking table test results two different constitutive models have been used: the cap-hardening Drucker-Prager model and the Severn-Trent model. The last one has been recently implemented by the Authors in a FEM code.

Both the constitutive models are able to capture in a some way the dynamic behaviour of foundation soil and steel frame. In particular, the cap-hardening Drucker-Prager constitutive model is able to capture the horizontal accelerations in the foundation soil and in the steel frame, as well as the horizontal displacements of the steel frame. While it fails in the prediction of vertical displacements, particularly for the soil outside the foundation.

The Severn-Trent constitutive model is able to predict the behaviour of the soil in terms of horizontal accelerations and in terms of horizontal and vertical displacements. In particular, the model is able to predict soil densification due to shaking, while the cap-hardening Drucker-Prager constitutive model is not able to predict it. The Severn-Trent model is also able to predict the behaviour of the steel frame in terms of both accelerations and horizontal and vertical displacements. Some limited discrepancies still remain at the soil-foundation interface. This could be done to possible foundation up-lifting, which is outside of the aim of the present paper and required a new modelling, which is underway.

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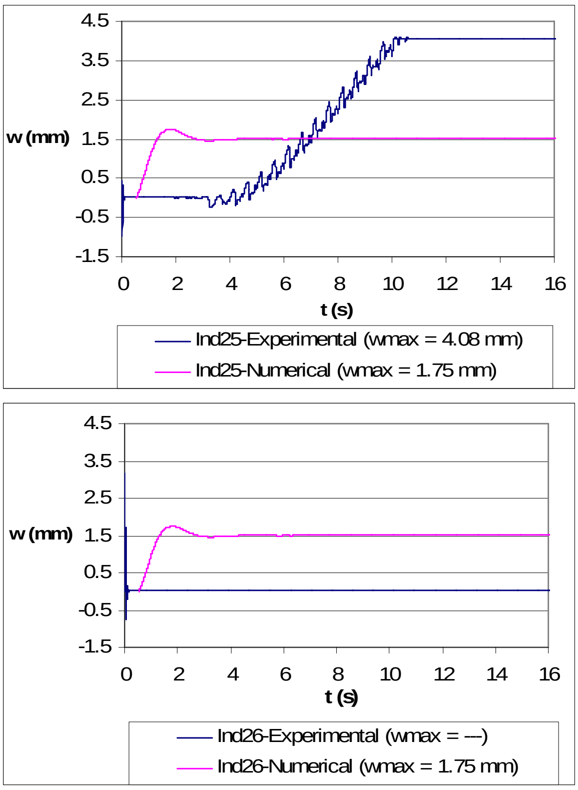

Fig. 22. Soil surface settlements; the numerical results are obtained using the Severn-Trent model.

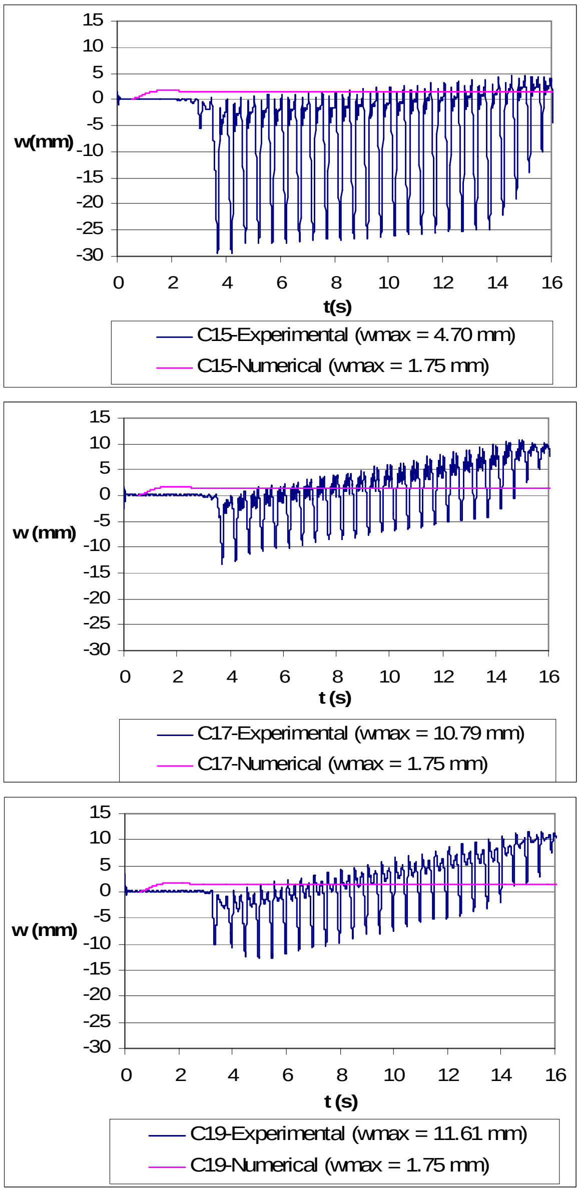

Fig. 23. Foundation settlements; the numerical results are obtained using the Severn-Trent model.

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