G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2005-2008 Project*, 223-270, © 2009 Doppiavoce, Napoli, Italy

INNOVATIVE PROCEDURES FOR DESIGN OF RETAINING STRUCTURES AND EVALUATION OF SLOPE STABILITY

Stefano Aversa [a], Giovanni Barla [b], Sebastiano Rampello [c], Armando L. Simonelli [d]

[a] *Università di Napoli Parthenope, Napoli, Italy, stefano.aversa@uniparthenope.it*
[b] *Politecnico di Torino, Torino, Italy, giovanni.barla@polito.it*
[c] *Università di Roma La Sapienza, Roma, Italy, sebastiano.rampello@uniroma1.it*
[d] *Università del Sannio, Benevento, Italy, alsimone@unisannio.it*

1 INTRODUCTION

The analysis of the seismic behaviour of slopes and the study of soil-structure interaction (SSI) under earthquake condition are very relevant topics both from the theoretical and the practical points of view.
While in the past very simple pseudostatic methods based on conventional seismic coefficients were normally used, performance based design is actually the most suitable approach to analyse foundations, retaining structures, tunnels, caverns, slopes, embankments, earth dams, and so on.
According to this approach, analyses of different level of complexity can be performed depending on the relevance of the problem, the availability of geotechnical and seismological data, the type of performance to be verified. In general it is possible to distinguish among three main categories:

1. *pseudostatic methods and empirical correlations*: these methods, widely diffused in the applications, require the definition of appropriate seismic coefficients "equivalent" to the actual seismic action; it is obvious that different seismic coefficients can be used for different structures and, also, for the same type of structure depending on the required performance;
2. *simplified dynamic analyses*: the main feature of such methods is that the study of SSI is generally uncoupled; different analysis methods attain to this category (*e.g.* displacements methods; subgrade reaction methods); the seismic input should be a set of acceleration time histories;
3. *fully coupled numerical analyses*: also in this case the seismic input should be a set of acceleration time histories.

The researches performed in the framework of Theme 6 referred to this scheme and were mainly devoted to assess different procedures of analysis and to calibrate them against benchmarks from field data and model tests results as much as advanced theoretical studies. Researches were devote to the following:

- **Sub-theme 6.1**: **Deep Excavations and Underground Tunnels in Urban Areas** (coordinated by Stefano Aversa);
- **Sub-Theme 6.2**: **Tunnels and Caverns in Rock** (coordinated by Giovanni Barla)
- **Sub-Theme 6.3**: **Slope Stability** (coordinated by Sebastiano Rampello)
- **Sub-Theme 6.4**: **Deep Foundations** (coordinated by Armando L. Simonelli)

The motivations and the background, the results, the possible developments of the four sub-themes are synthesized in the following pages.

SUB-THEME 6.1: DEEP EXCAVATIONS AND UNDERGROUND TUNNELS

1 INTRODUCTION

The performance of flexible retaining structures and tunnels subjected to seismic actions can be evaluated with several methods at increasing levels of complexity from pseudo-static methods, or simplified dynamic methods, to fully coupled effective stress numerical analyses under dynamic loading.

In the pseudo-static approach, the soil and the structure, be it a flexible retaining wall or a tunnel, are assumed to be subjected to an acceleration which is taken to be constant in space and time and is expressed through a seismic coefficient, K, generally a fraction of the peak acceleration expected at the site. The main conceptual drawback of the method is the selection of a representative value of the seismic coefficient such that the pseudo-static actions produce equivalent effects on the structure as those induced by the seismic actions, i.e. actions that are variable in time and space and transient in nature. In principle an appropriate value of the seismic coefficient should depend on the design ground motion parameters, such as peak ground acceleration, frequency content, and duration, but also on the soil and structure characteristics.

For flexible earth retaining structures, simplified dynamic methods include both displacement-based methods derived from the formulation originally proposed by Newmark (1965) for dams, and subgrade reaction methods. In the latter class of methods, soil-structure interaction is tackled using a decoupled approach in which the ground response is evaluated first in free field conditions and then computed displacements are applied to a set of springs and dashpots restraining the structure. These methods are frequently adopted also for the seismic analysis of tunnel linings, in which the load increments to be applied to the tunnel lining follow from a free-field seismic response analysis. The main problem with the adoption of displacement-based methods for flexible retaining structures is the definition of compatible failure mechanisms, while in subgrade reaction methods there is a problem with the selection of appropriate material parameters for springs and dashpots. Moreover, both for displacement based methods and subgrade reaction methods, representative design input acceleration time histories must be selected.

In principle, numerical methods allow the most comprehensive analysis of the response of flexible retaining structures and tunnels to seismic loading. However, like any other numerical analysis, reliable dynamic soil-structure interaction analyses require a number of simplifications and approximations of the problem, including the definition of a representative soil profile, the selection of representative mechanical properties for each layer, assumptions on ground water conditions and initial state of stress, structural geometry and boundary conditions, and the selection of representative design input acceleration time histories. Among all the idealisations, a major role is played by the constitutive model adopted for the soil, which should be able to reproduce the main features of its mechanical behaviour under cyclic loading, such as irreversibility of deformations, incremental non-linearity, hysteretic dissipation of energy, and memory of previous stress history. This can only be achieved adopting advanced constitutive models developed within the framework of bounding surface plasticity, kinematic hardening plasticity or hypoplasticity, generally not included in the libraries of commercial codes and requiring input parameters not routinely measured in field or laboratory tests. In the context of this research project, simple and advanced numerical analyses were carried out finalised both to a better understanding of the soil-structure response under seismic loading and to supply benchmarks were closed form solutions were not available or to develop simple procedures for design practice.

2 BACKGROUND AND MOTIVATION

Public demand for services to improve the quality of life in the urban environment is constantly growing; almost all greater cities in Italy face the necessity of modifying the existing balance between private and public transport. In the last ten years, due to the lack of finalised action, the quality of life has deteriorated further and further: air pollution has dramatically increased, with negative effects on public health and the cost of national health programmes, and travel times in rush hours became unacceptably long. The objective of many administrations was that of rendering the many functions of city life accessible through the creation of a sustainable and extended net of public transport, such that the use of private transport becomes an option rather than a necessity. These policies in the management of mobility have lead to a significant pressure for the construction of new underground train lines and car parks. This implies the necessity to carry out open excavations and bored tunnels in the urban environment, often in difficult ground, such as soft soils and weak rocks, and below the water table, and almost always in close vicinity of existing buildings and structures. Many important underground project are being carried out in Italy, like those in Milano, Roma and Napoli.

The ability to predict with confidence excavation and tunnelling induced displacement is a crucial aspect of the design because ground movements transmit to adjacent structures as settlements, rotations and distortions of their foundations, which can, in turn, induce damage affecting visual appearance and aesthetics, serviceability or function, and, in the most severe cases, stability of the structures (Burland and Wroth, 1974; Burland et al. 1977; Boscardin and Cording, 1989).

The availability of powerful computer codes, the formulation of constitutive models able to describe the key features of soil behaviour under different stress-paths as much as the development of experimental techniques for laboratory and in-situ tests, have made possible reliable predictions of the performance of such geotechnical structures both during construction and the subsequent life cycle. Monitoring of these structures improved the reliability of traditional design methods and permitted the development of empirical methods able to forecast displacements of the retaining structures and settlements of the ground due to the excavations. Finally the introduction of the methods of verification against limit states with partial safety factors, in agreement with EN1997-1, allowed for evaluating the safety of structures for which it is not easy to define a global safety factor.

All these progress almost exclusively have concerned the response under static conditions. In the case of seismic actions the study of the soil-structure interaction is still a complex problem to analyse. The analysis can be performed by means of pseudostatic methods based on limit equilibrium only in very simple conditions (i.e. cantilever sheet pile walls or single propped sheet pile walls), using conventional seismic coefficients defined with reference to other structures. Also the recent EN 1998 -5 faced in marginal way the design of the underground structures. These considerations have lead to a significant pressure for the study of the complex seismic interaction between ground and underground structures, with the main purpose to develop reliable and simple design procedures.

3 RESEARCH STRUCTURE

The research consisted in the following steps:
1 *Literature review:* survey of the scientific literature on the seismic behaviour of flexible retaining structures and tunnel linings.
2 *Physical modelling*: Centrifuge tests on reduced scale models in order to study the behaviour of shallow tunnels and flexible retaining walls under seismic actions performed at the Schofield Centre of the Cambridge University Engineering Department (CUED).
3 *Advanced numerical modelling*: Finite Element formulation of equilibrium and dynamic coupled flow for a two-phase medium. Identification of constitutive models capable of reproducing the main features of soil behaviour under cyclic and dynamic actions. Implementation of the selected models in existing Finite Element codes.
4 *Simplified dynamic and dynamic analyses*: the results obtained by means of centrifuge tests and advanced numerical modelling and centrifuge tests were compared with those from simplified dynamic and dynamic analyses.
5 *Pseudo-static analyses*: the principal aim of the comparison between the results of pseudo-static approach and the dynamic analyses is the definition of criteria and procedures to select rationally an appropriate value of the seismic coefficient to be used in pseudo-static methods.

4 MAIN RESULTS

4.1 Centrifuge testing

Thirteen centrifuge tests were carried out in order to define an experimental database for calibrating simplified methods and advanced numerical analyses of the behaviour of shallow tunnels and flexible retaining walls under seismic actions.
The Cambridge centrifuge permitted to run dynamic tests up to a maximum acceleration of 80g. The seismic input consists of trains of approximately sinusoidal waves of different frequency (Madabhushi et al. 1998); during one test the same model is subjected to several trains of waves of different frequency, amplitude and duration. Models were prepared using a standard fine dry silica sand, namely Leighton Buzzard Sand.
Nine tests were carried out on models of pairs of retaining walls, six of which cantilevered and three of which propped against each other, while four tests were performed on tunnel models.
The results of the tests on tunnels are reported in Lanzano (2009). The major findings of the experimental program were:

1. The maximum amplification occurs at the reference alignment, while along the free-field and the tunnel alignment the amplification was less significant, showing, in this last case, a filter effect of the tunnel.
2. The LVDTs measurements at two symmetrical points on the model surface show that densification occurred in the sand model during the tests. The measurements on the loose sand models gave rise to settlements about twice larger than those on dense sand models.
3. The time histories of bending moment and hoop force measured in the tunnel lining show the accumulation of stresses in the lining during the earthquake, probably caused by soil densification.

The models of flexible retaining walls were instrumented to measure deformation, axial load in the props and bending moments in the walls, acceleration at different locations in the model or at its boundaries, acceleration of the walls, horizontal displacements of the walls. A layout of the monitoring devices adopted for a test on a pair of propped wall is shown in Figure 1.

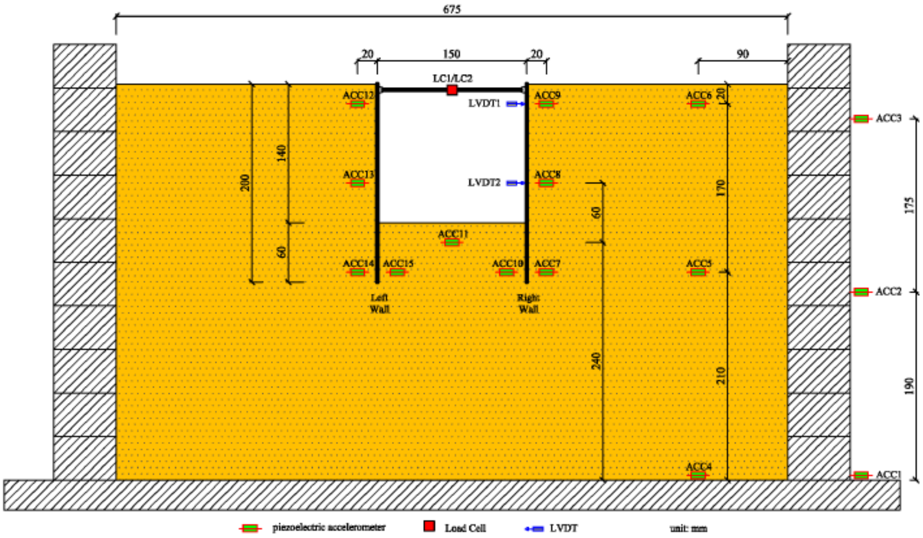


Figure 1. Test CW1: transducers layout.

Main results of the experimental programme on flexible retaining walls can be synthesized in the followings points:

1. Maximum accelerations recorded in the proximity to the retaining walls are slightly larger than those measured in free-field conditions;
2. Amplifications in terms of Arias intensity are larger than those expressed in terms of maximum accelerations, especially for earthquakes whose nominal frequency is close to the fundamental frequency of the ground.
3. During the dynamic phases cantilever retaining walls accumulate permanent displacements toward the excavation; an earthquake produces an increase of displacements only if it has an acceleration higher than the previous ones.
4. Bending moments permanently increase during earthquakes; they increase only if the wall has not experienced any other earthquake of higher acceleration.

4.2 Advanced numerical modelling

The main goal of the numerical modeling activities was twofold: i) obtaining a better understanding of the seismic behaviour of the soil-retaining structure system; ii) to use the results as a benchmark to develop simplified analysis methods in current seismic design.

For the modeling of cyclic/dynamic behaviour of fine-grained soils, the research has been focused on the following three constitutive models:

1. the Bounding Surface plasticity with radial mapping model of Tamagnini & D'Elia (1999);
2. the Bounding Surface plasticity with multiple, kinematic hardening loading surfaces of Rouainia & Wood (2000);
3. the Hypoplastic model of Masin (2005).

For coarse grained soils the research have been focused on the following constitutive models:

1. The BSKH elastoplastic model of Papadimitriou & Bouckovalas (2002);
2. The HP model of von Wolffersdorff (1997).

The model of Papadimitriou & Bouckovalas (2002) was used for FE simulations of the results of centrifuge tests through the code ABAQUS® standard. The calibration of the model for the

Leighton Buzzard sand has been carried out based on the experimental data obtained by Visone (2009).
The results of the simulation indicate that there is a clear amplification effect of the horizontal accelerations, moving from the model base to the ground surface. The amplification effect is significant in a medium-high frequency range. The evolution with time of the bending moments on the tunnel lining in four points placed along the diagonals at 45° from the horizontal and vertical axes show that the magnitude of the bending moments in all points tends to increase almost monotonically with time. This is due to the progressive accumulation of plastic strains in the soil surrounding the tunnel lining during the seismic stage, in agreement with the observed behaviour.

4.3 Simplified dynamic and dynamic analyses

4.3.1 Tunnels

Simplified dynamic analyses were carried out to develop a simple criterion for evaluating the internal force increments induced by an earthquake in the trasversal plane of a tunnel lining, according to the objectives of the research project. The proposed methodology allowed for evaluating the influence of the kinematic interaction on the basis of the results of a set of dynamic analyses, performed for several subsoil conditions, tunnel geometries and characteristics of the seismic event. Three subsoil profiles were considered, which correspond to deposits of typical soft clay, medium-dense sand and gravel. The dynamic analyses were performed by means of Plaxis® 8.2, by modeling the soil as an equivalent linear viscous-elastic medium. Interesting results were obtained in terms of bending moments in the tunnel lining.

FE element analyses were also carried out in the longitudinal direction modelling the tunnel as a series of elastic beams supported by springs, both in the longitudinal and the transversal plane. The stiffness of the springs were evaluated using the formulas presented by St. John & Zahrah (1984).
Unlike closed-form solutions, this method accounts for the effects of important factors such as the change in mechanical and geometric characteristics of the tunnel and in stiffness of the springs (along tunnel axis) as well as the presence of constraints at the ends of the structure.
The numerical analyses have been carried out considering the same tunnel and three idealized ground conditions.
In conclusion, the FE method gave rise to internal forces very similar to the analytical methods in terms of internal forces, with the exception of the following situations:

(a) when the tunnel traverses distinct geological media with sharp contrast in stiffness;
(b) when the length of the tunnel is comparable with the wave length of the seismic input;
(c) when the ends of the tunnel are connected to a station end wall or a rigid, massive structure such as a ventilation building, which represent a constraint for the ends of the tunnel.

4.3.2 Flexible retaining walls

A number of numerical analyses were carried out to study the seismic behaviour of flexible retaining walls (cantilever and propped) embedded in a coarse-grained soil with different computer codes (Plaxis®; Flac®; ANSYS®) and different types of constitutive models: the Mohr-Coulomb elastic-perfectly plastic model and the isotropic hardening model HSM in Plaxis; the elastic-perfectly plastic model with Drucker-Prager yield surface in ANSYS. The

viscous damping of the soil was modelled via the Rayleigh approach. Flac analyses were performed using an elastic-plastic model with purely hysteretic damping. Boundaries were modelled to account for radiation damping.. Acceleration time histories were selected from a database of Italian seismic events (Scassera et al. 2006).
The influence of the mesh size was firstly assessed at the preliminary stage of this study. Emphasis was then placed on the effect of soil damping ratio. Results from these simulations showed that bending moments and prop forces (for the propped walls) during and after the earthquake are significantly greater than the static ones, as a consequence of the increase in contact stress at the soil-wall interface induced by the seismic excitation (Aversa et al. 2007). This result was confirmed by experimental date from centrifuge tests (Aversa et al. 2008).
A limited number of analyses were carried out with the FD method by assuming the walls to be elastic-perfectly plastic. Propagation of the two seismic records considered in this analysis (Tolmezzo and Assisi) causes instantaneous attainments of the available shear strength in soil zones interacting with the walls, resulting in a progressive increase in bending moments and axial load in the prop (Figure 2).

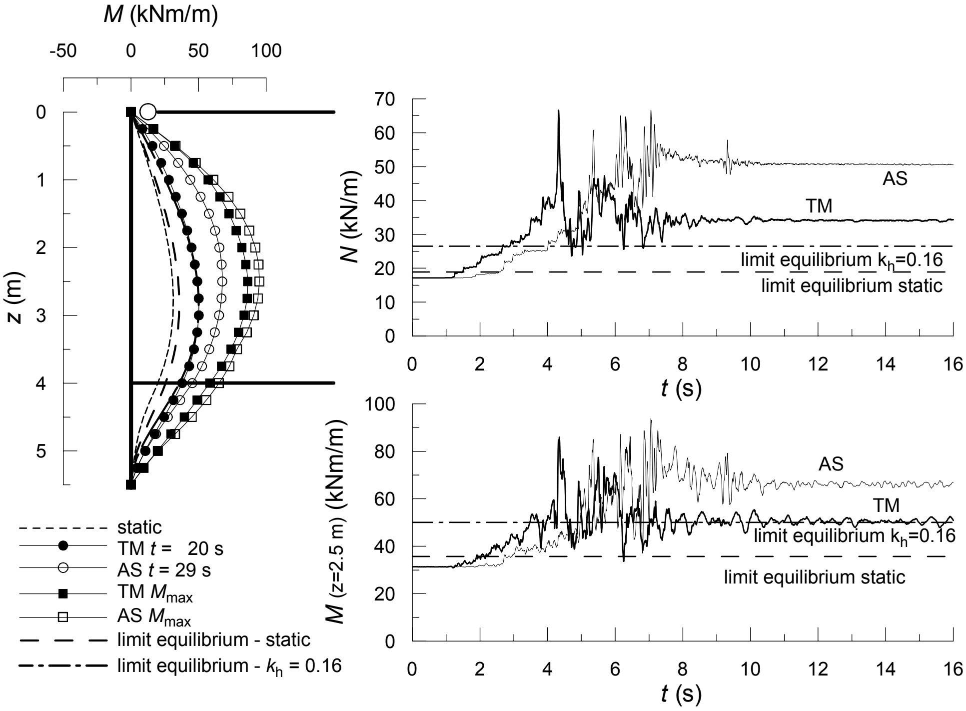


Figure 2. Bending moment and axial load in the prop.

The results showed that, while the maximum bending moment cannot exceed the yield moment, the computed wall displacements are about 30 % larger than those evaluated assuming that the walls are linearly elastic.
The main results of this study are described in detail in: Callisto & Soccodato (2007, 2009), Callisto et al. (2007, 2008).

4.4 Pseudostatic procedure analysis: revisited approaches and innovative criteria

In pseudo-static analyses of tunnels internal forces are commonly calculated from the free field shear strain profile, i.e. neglecting the kinematic interaction between the tunnel lining and the ground. This lead to the development of new expression capable to account at a very simple level the effect related to kinematic interaction phenaomena. .

For flexible retaining structures, pseudo-static solutions have been used in the past as possible design procedures for gravity retaining walls (Richards & Elms 1979, Whitman 1990). Conversely, less attention has been paid for the seismic design of embedded retaining walls. In these structures, a significant part of the resisting forces is provided by the passive reaction of the soil located below the bottom of the excavation, and this resistance is significantly affected by inertial effects. Therefore, during an earthquake a continuous interaction occurs between actions and resistances. Upon instantaneous attainment of the available strength, cantilever and single-propped walls undergo rigid-body movements in a way which is qualitatively similar to the behaviour of gravity retaining walls.
An attempt was made to define a rational approach for seismic design of cantilevered embedded retaining walls (Callisto & Soccodato, 2007; Callisto et al., 2007, 2008). Only the case of dry coarse grained soils was considered. For a given retaining wall, a critical horizontal seismic coefficient k_c can be evaluated by performing iteratively a limit-equilibrium analysis, and finding the value of the seismic coefficient for which soil strength is fully mobilised. Permanent displacements can then be assumed to result from a Newmark-type integration of the relative motion, and increase as k_c decreases. For a given soil and a given excavation height H, the value of k_c depends essentially on the embedded length d.
Therefore, the embedded length should be chosen on the basis of the maximum displacement allowed for the severe seismic event.

5 DISCUSSION

The objective of understanding the seismic soil-structure interaction for both circular tunnels and cantilever and single propped retaining walls was pursued by means of both centrifuge testing and numerical simulations of different complexities. The investigation has been performed with main reference to unsaturated coarse grained soils. For this type of soil the comparison among results of different computer codes and physical models has shown common features in terms of co-seismic and post-seismic bending moments and other internal forces, displacements of the structure and of the ground level.
Different computer codes have been tested. Particular attention has been devoted to the effects of the mesh size, to the boundary conditions, to the way of considering damping. The role of different constitutive models has been investigated too. Sophisticated constitutive models able to reproduce behaviour of both coarse grained and fine grained soils, based on Boundary Surface Plasticity or on Hypoplasticity, have been implemented in FE code, to extend the centrifuge results to more complex situations and ground conditions. At the same time, more common constitutive models, available in the libraries of commercial computer codes, have been tested.
In addition simple pseudostatic methods to design these types of structures have been developed. Some of the obtained results have been already taken into account in the recent Italian building code (DM 14/01/2008). For the case of cantilever and single propped sheet pile walls a sort of capacity design procedure has been proposed.
To sum up, it may be stated that most of the research objectives were attained with reference to the considered structures and that some of the factors affecting the seismic soil-structure interaction under plane strain conditions were recognised, their influence being isolated and evaluated.

6 VISIONS AND DEVELOPMENTS

The present research activity was mainly addressed to the study of the seismic behaviour of tunnels and cantilever and single propped sheet pile walls in homogeneous dry coarse grained soils. Centrifuge model results provided a very good base for calibrating design procedures of different complexities. Some of the results can be directly applicable in the design; some others require an increase of knowledge both in terms of experimental analyses and in terms of numerical simulation. In particular, the case of saturated fine grained soils requires more simulations; the behaviour other types of underground structures, very common in urban areas (i.e., multi-propped retaining wall; parallel tunnels; ecc.), should be more deeply investigated. Similarly the very relevant case of seismic interaction of underground structures in highly inhomogeneous soils or in correspondence of sharp changes in structures stiffness (i.e., the connection between tunnels and deep propped excavation for underground railway stations, ecc.) is still an unsolved problem. These problems could be tackled with the same type of approach used in the last three years: physical models and highly sophisticated computer codes to define a target in order to calibrate methods of analyses of different complexity, including simple pseudostastic approaches for some of the cases. The results of this research might provide guidance for the design of new underground structures, but would also assist in the seismic evaluation of the many existing tunnels and excavations present in urban areas, originally designed without taking into account seismic actions or simulating seismic loads in very simple and conventional way.

7 REFERENCES

Boscardin, M.D., Cording, E.J. (1989). Building response to excavation-induced settlements. J. Geotech. Eng., 115, 1, 1-21.

Burland, J.B., Broms, B.B., DeMello, V.F.B. (1977). Behaviour of foundations and structures, IX ICSMFE Tokio, 2, 495-546.

Burland, J.B., Wroth, C.P., (1974). Settlements of buildings and associated damage. State of the Art Review. Conf. Settlement of Structures, Pentech Press, London, 611-654.

Madabhushi S.P.G., Schofield A.N., Lesley S., (1998), A new Stored Angular Momentum (SAM) based earthquake actuator, Centrifuge 98, Balkema, Rotterdam

Masin, D. (2005). A hypoplastic costitutive model for clay. Int. J. Num. Anal. Meth. Geomech. 29, 311–336.

Newmark, N.M. (1965). “Effects of earthquakes on dams and embankments.” Fifth Rankine lecture. Géotechnique 15(2), 139–193.

Papadimitriou, A., Bouckovalas, G. (2002). Plasticity model for sand under small and large cyclic strains: a multiaxial formulation. Soil Dynamics and Earthquake Engineering, 22, 191-204.

Richards, R., and Elms, D.G. (1979). “Seismic behavior of gravity retaining walls.” Journal of the Geotechnical Engineering Division, ASCE, 105(GT4), pp. 449-464.

Rouainia, M., Wood, D. M. (2000). A kinematic hardening constitutive model for natural clays with loss of structure. Géotechnique 50 (2), 153–164.

Scasserra G., Lanzo G., Mollaioli F., Stewart J.P., Bazzurro P., Decanini L.D. (2006). Preliminary comparison of round motions from earthquakes in Italy with ground motion prediction equations for active tectonic regions. Proc. of the 8th U.S. National Conference on Earthquake Engineering, San Francisco, 18-22 Aprile 2006, CD rom, Paper No. 1824.

St. John C. M. & Zahrah T. F. (1984). Seismic Design Considerations for Underground Structures, Advances in Tunnelling Technology and Subsurface Use, Vol.4(3), pp. 105-112.

Tamagnini, C., D’Elia, M. (1999). A simple bounding surface model for bonded clays. In: Jamiolkowski, M., Lancellotta, R., Lo Presti, D. (Eds.), Pre–failure Deformation Characteristics of Geomaterials. Balkema, Rotterdam.

von Wolffersdorff, P. A. (1996). A hypoplastic relation for granular materials with a predefined limit state surface. Mechanics of Cohesive-Frictional Materials, 1:251-271.

Whitman R.V. (1990). Seismic design behavior of gravity retaining walls. Proc. ASCE Spec. Conf. on Design and Performance of Earth retaining Structures, GSP 25, ASCE, 817-842.

8 RELUIS REFERENCES

Aversa S., Maiorano R.M.S, Tamagnini C (2007). Influence of damping and soil model on the computed seismic response of flexible retaining structures. In: Geotechnical Aspects of EC8. Madrid, 25 settembre 2007, BOLOGNA: Patron Editore

Aversa S., Callisto L., Conti R., Madabushi F., Maiorano R.M.S., Soccodato F. e Viggiani G. (2008). Seismic soil-structure interaction for flexible retaining walls. Seismic Engineering International Conference commemoratine the 1908 Messina and Reggio Calabria Earthquake, Sciacca (Reggio Calabria), 8-11 luglio 2008.

Bilotta E., Lanzano G., Russo G., Aiello V., Conte E., Silvestri F., Santucci de Magistris F. (2006). Sollecitazioni indotte da sisma in gallerie circolari interrate. Incontro Annuale dei Ricercatori di Geotecnica IARG 2006, Pisa.

Bilotta E., Lanzano G., Russo G., Santucci de Magistris F., Aiello V., Conte E., Silvestri F., Valentino M. (2006). Pseudostatic and dynamic analyses of tunnels in transversal and longitudinal directions. Proc. 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Grece.

Bilotta E., Lanzano G., Russo G., Santucci de Magistris F., Aiello V., Conte E., Silvestri F., Valentino M. (2007). Analisi pseudostatica e dinamica di gallerie in aree sismiche. Proc. XII Convegno Nazionale "L'Ingegneria Sismica in Italia", ANIDIS.

Bilotta E., Lanzano G., Russo G., Santucci de Magistris F., Silvestri F. (2007). Methods for the seismic analysis of transverse section of circular tunnels in soft ground. Workshop of ERTC12 - Evaluation Committee for the Application of EC8 Special Session XIV ECSMGE, Madrid, Patron Ed., Bologna.

Bilotta E., Lanzano G., Russo G., Santucci de Magistris F., Silvestri F. (2008). An early-stage design procedure for circular tunnel lining under seismic actions. Proc.14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China.

Callisto L. & Soccodato F.M. (2007). Seismic analysis of an embedded retaning structure in coarse-grained soils. 4th Int. Conf. on Earthquake Gotechnical Engineering. Thessaloniki.

Callisto L., Soccodato F.M. e Conti R. (2007). Analisi del comportamento sismico di paratie in terreni a grana grossa. Incontro Annuale dei Ricercatori di Geotecnica (IARG), Salerno.

Callisto L., Soccodato FM, Conti R (2007). Analisi del comportamento sismico di paratie in terreni a grana grossa. Atti dell'Incontro Annuale dei Ricercatori di Geotecnica – IARG 2007, Salerno

Callisto L. e Aversa S. (2008). Dimensionamento di opere di sostegno soggette ad azioni sismiche. In: Opere Geotecniche in Condizioni Sismiche. Torino, 2-3 dicembre 2008, BOLOGNA: PATRON EDITORE.

Callisto L., Soccodato F.M. & Conti R. (2008). Analysis of the seismic behaviour of propped retaining structures. Proc. Geotechnical Earthquake Engineering and Soil Dynamics IV Conference, Sacramento (in print).

Callisto L, Soccodato FM, Conti R (2008). Analysis of the Seismic Behaviour of Propped Retaining Structures. Proc. 4th Decennial Geotechnical Earthquake Engineering and Soil Dynamics Conference, Sacramento, USA

Callisto L., Soccodato F.M.(2009). Performance-Based Design of Embedded Retaining Walls Subjected to Seismic Loading. Workshop on EC/8. Reluis Ninal Conference, Naples.

Carrubba P., Brusarosco D. (2008). "Static and dynamic behaviour of a full scale geosynthetic reinforced embankment provided with block facing". Proc. The First Pan American Geosynthetics Conference, Cancun, Mexico, 2-5 March 2008, pp. 888-893.

Carrubba P., Brusarosco D. (2007). "Dynamic modelling of earth retaining structures". Proc. 14th ECSMGE, Madrid, 24-27 September, Workshop Special Session ERTC12.

Carrubba P. Pavanello P. (2007). “Equivalent loading for the seismic analysis of sliding structures”. Proc. 14th ECSMGE, Madrid, 24-27 September, Workshop Special Session ERTC12.

Carrubba P., Maugeri M., Moraci N. (2007). “Seismic response analysis of a geosynthetic reinforced embankment”. Proc. 14th ECSMGE, Madrid, 24-27 September, Workshop Special Session ERTC12.

Conti R, Madabhushi SPG, Viggiani GMB (2009). Physical modelling of flexible retaining walls under seismic actions. Submitted to the 7th International Conference on Physical Modelling in Geotechnics, Zurich, Switzerland

Conti R (2008). Centrifuge model tests of flexible retaining walls under seismic actions. European Symposium on Geotechnical Centrifuge Modelling, London, UK

Conti R (2008). Centrifuge Modelling of the Dynamic Response of Cantilever Retaining Walls. Proc. XIX European Young Geotechnical Engineers Conference, Gyor, Hungary

Fabbrocino G., Laorenza C., Ranieri C., Santucci de Magistris F. (2008). Seismic monitoring of structural and geotechnical integrated systems. Proc. of the 2nd Asia-Pacific Workshop on Structural Health Monitoring, Melbourne, Australia.

Fabbrocino G., Laorenza C., Rainieri C., Santucci de Magistris F., Visone C. (2008). Structural and geotechnical seismic monitoring of the new Student House in Campobasso, Italy. International Conference on Performance-Based Design in Earthquake Geotechnical Engineering — from case history to practice — IS Tokyo 2009.

Fabbrocino G., Laorenza C., Rainieri C., Santucci de Magistris F. (2008). Structural and geotechnical seismic monitoring by smart SHM systems. Cansmart 2008, Proc. Int.l Workshop Smart Materials and Structures (G. Akhras Ed.) October 2008, Montreal, Canada 51-60.

Fabbrocino G., Guarini C., Laorenza C., Rainieri C., Santucci de Magistris F. (2009) Il ruolo del calcestruzzo nell’implementazione di sistemi di monitoraggio di opere di sostegno flessibili. Concrete2009 EVOLUZIONE TECNOLOGICA DEL CALCESTRUZZO Tradizione, attualità, prospettive.

Fabbrocino G., Laorenza C., Rainieri C., Santucci de Magistris F., Salzano C., Deseri L. Monitoraggio sismico di una paratia di pali in cemento armato. Anidis 2009, Bologna.

Lanzano, G. 2009. Physical and analytical modelling of tunnels under dynamic loadings. PhD Thesis. University of Naples Federico II.

Santucci de Magistris F. (2008). Beyond EC8: the New Italian Seismic Code. Submitted to The Mining-Geological-Petroleum Engineering Bulletin.

Visone C., Santucci de Magistris F. (2009) Performance-Based approach in seismic design of embedded retaining walls. XVII ICSMGE, Alexandria, Egypt (in print).

Visone C., Santucci de Magistris F. (2009) Mechanical behaviour of the Leighton Buzzard Sand 100/170 under monotonic, cyclic and dynamic loading conditions. Anidis 2009, Bologna.

Visone C., Bilotta E., Santucci de Magistris F. (2008). Remarks on site response analysis by using Plaxis dynamic module. “Plaxis Bulletin” no. 23 pp. 14-18.

Visone C., Bilotta E., Santucci de Magistris F. (2008). One-dimensional ground response as a preliminary tool for dynamic analyses in Earthquake Geotechnical Engineering. Journal of Earthquake Engineering (in print).

Visone C., Santucci de Magistris F. (2008). A review of design methods for retaining structures under seismic loadings. Strategies for reduction of the seismic risk (G. Fabbrocino & F. Santucci de Magistris eds.) pp. 51-64.

Visone C., Santucci de Magistris F., Bilotta E. (2008). Comparative study on frequency and time domain analyses for local site response. Submitted to The Mining-Geological-Petroleum Engineering Bulletin.

Visone C., Santucci de Magistris F., Bilotta E. (2008). Calibrazione dei modelli numerici agli elementi finiti per le analisi dinamiche in ingegneria geotecnica sismica. Incontro Annuale dei Ricercatori di Geotecnica IARG 2008, Catania.

Visone C. (2009). “Performance based design of embedded retaining walls”. Tesi di Dottorato, Università di Napoli Federico II.

Visone C., Santucci de Magistris F. (2008). Nonlinear incremental analysis for the prediction of seismic performances of embedded retaining walls. International Conference on Performance-Based Design in Earthquake Geotechnical Engineering — from case history to practice — IS Tokyo 2009.

Visone C., Santucci de Magistris F. (2007). Aspetti della progettazione in zona sismica delle opere di sostegno flessibili. . Incontro Annuale dei Ricercatori di Geotecnica IARG 2007, Salerno.

Visone C., Santucci de Magistris F. (2007). Some aspects of seismic design methods for flexible earth retaining structures. Workshop of ERTC12 - Evaluation Committee for the Application of EC8 Special Session XIV ECSMGE, Madrid, Patron Ed., Bologna.

SUB-THEME 6.2: TUNNELS AND CAVERNS IN ROCK

1 INTRODUCTION

Underground structures are critical elements when considering transportation systems. They are also critical elements in the more general framework of utility systems (e.g. hydraulic tunnels and hydroelectric caverns, tunnels for transportation of fluids such as petroleum and natural gas, etc.). The importance of these underground structures makes the problem of understanding their behaviour during a seismic event extremely relevant.

The behaviour of underground structures in seismic conditions is significantly different from that of surface structures. While these are substantially designed to sustain the inertial forces due to ground acceleration, the loads acting on underground structures are linked to the state of stress and strain in the ground surrounding them. Thus, the need arises to develop design analysis methods in order to assess the effects on underground structures in seismic conditions.

It is understood that these effects cannot be neglected at the design stage, in particular in nations like Italy where wide areas are characterised by different levels of seismicity. In addition, one may note that there are no rules or general guidelines available, in Italy or in other Countries, for the design of underground structures in seismic conditions. The present research on tunnels and underground caverns in rock in seismic conditions stems from this important need.

2 BACKGROUND AND MOTIVATION

Underground structures are generally thought to be safer than surface structures, given that the ground in which they are built provides protection against earthquake damage. However, these structures cannot be considered to be intrinsically safer than surface structures, as well demonstrated by recent seismic events: in 1995, Kobe (Japan); in 1999, Chi-Chi (Taiwan); in 1999, Kocaeli (Turkey); and in 2004, Niigata (Japan); when a number of tunnels and underground cavities underwent significant damages.

The response of underground structures to a seismic event is governed by the surrounding ground behaviour and not by the inertial characteristics of the structure itself, as the response to such an event is substantially dependent on induced displacements. Thus the design of underground structures in seismic conditions need to consider ground deformations and soil-structure interaction, which imply that the kinematics effects are predominant with respect to the inertial effects.

The presence of the structure is to modify the “free-field” deformations due to the interaction of the ground and the support. In such conditions one is to adopt either closed-form solutions or numerical solutions. The closed-form solutions may be used under simplified geometrical and loading conditions and consider separately the stresses due to the distortion of the tunnel cross section and the out of plane bending and axial deformations. The numerical solutions comprise different design analysis methods, which may comply with a different degree of approximation in relation to the current design stage and the level of understanding of the geological and geotechnical conditions to be considered.

The development of design analysis methods requires a careful understanding of the effects of past earthquakes which have damaged underground structures, so as to be able to underline the most significant factors related to the state of damage. The effects of ground motion is analysed in the following, without considering other types of instability (e.g. earthquakes driven landslides, liquefaction, etc.). In fact, problems of underground structures crossing landslides or active fault zones, although causing relevant damages, are restricted to these zones. Therefore, it is expected that these zones can be localised at the design stage so as to minimise such effects by a careful selection of the tunnel alignment.

3 RESEARCH STRUCTURE

The main purpose of the present research has been to develop design analysis methods for the study of the interaction problems between the lining and the rock mass during severe earthquakes. In view of the present practice in the design of tunnels and underground structures (Barla, 2005), both simplified and advanced numerical methods have been developed. With the intent to compute the state of stress in tunnel linings in seismic conditions, at the design stage, most of the attention has been posed on the development and validation of simplified methods.

A significant problem which has been considered in detail during this research has been the simulation of ground motion, in view of the absence of appropriate measurements of ground acceleration at depth. Considering that a tunnel extends along its centreline, it has been felt important to be able to consider different motions in different sections of the tunnel/cavern. These aspects have been accounted for by using a number of solutions such as proposed by Hisada (Hisada, 1995; Hisada and Bielak, 2003), which allow one to compute the ground motion in a stratified half-space near fault zones.

The complexity of the problem being considered requires in most cases the adoption of advanced numerical methods (mainly: Finite Element Methods - FEM, Finite Difference Methods - FDM, Distinct Element Methods - DEM). This necessity is strictly linked to the three-dimensional conditions characterizing the problem, the complex rock mass behaviour, where in cases the adoption of continuum/equivalent continuum models may become questionable; the complex geometrical and geological characteristics of some applications; etc.. These advanced models need be developed and appropriate rules for its use are to be made available, also in view of the computations to be carried out in order to validate the simplified methods. With the above background in mind, the research structure can be defined in terms of the main topics which have been addressed during the work carried out so far:

1. State of the art of underground structures in seismic conditions.
2. State of the art of design analysis methods.
3. Database, damage scenarios, and development of fragility curves.
4. Definition of seismic input.
5. Design analyses of tunnels and caverns, by analytical and numerical methods, in 2D and 3D conditions.
6. Design analyses of well-documented case studies. Use of advanced codes (GeoELSE) and of commercial codes (FLAC and FLAC3D, Examine3D, Midas-GTS, etc).
7. Development of Early Warning innovative methods for underground monitoring.
8. Development of Guidelines for design analysis of underground structures in static and dynamic conditions.

The above topics have been developed by the three research units forming the Sub-Unit 6.2 (Politecnico di Torino, Department of Structural and Geotechnical Engineering - DISTR; Politecnico di Milano, Department of Structural Engineering - DIS; Eucentre, Pavia - Eucentre). DISTR and DIS mainly worked on the development of design methods for tunnels and underground cavities and on the validation of the adopted procedures, including the analysis of relevant case studies. The main task of Eucentre has been to define the seismic input for design analysis. DISTR has also been providing the documentation material for selected case studies and, moreover, has been working to develop innovative early warning systems for underground monitoring. Due to the limited space available, only the following points are briefly addressed in this review: Database on tunnel performance during earthquakes; Seismic vulnerability assessment and fragility curves; 3. Simplified solution for seismic design of tunnels; Advanced numerical method for seismic design of tunnels; Overview of case studies and applications.

4 MAIN RESULTS

4.1 Database on tunnel performance during earthquakes

A review of the seismic damages suffered by underground structures has been carried out by updating the database currently available (Dowding and Rozen, 1978; Owen and Scholl, 1981; Sharma and Judd, 1991; Power et al., 1998; Wang et al., 2001; etc.). This review shows that most tunnels which underwent damages were located in the vicinity of causative faults. The characteristics of ground motion in these cases can be significantly different from that of the far-field. There is no well defined distance over which a site may be classified as in near-field or far-field. A useful criterion to define the near-field zone is related to the comparison of the source dimension with the source to site distance.

From the bibliographic review of earthquakes-induced damages in underground structures, one can notice a wide variability of data, in terms of geometrical information, rock mass conditions, earthquakes characteristics, strong motion parameters, type of damage, type of support, etc. Therefore, in order to define unambiguously a damage criterion, in this work (see Corigliano, 2007a for details) the damage state has been defined according to Huang et al. (1999), combined with some considerations proposed by ALA (2001) and RTRI (2001). The damage classification includes qualitative and quantitative damage descriptions. In particular, this classification considers four levels of damage (none, slight, moderate and severe damage) grouped into three categories A, B and C, in which the group A includes none and slight damage. Each category comprises the functionality states defined by RTRI (2001). The database collected has been used to assess the effects of the following main factors on tunnel response: overburden depth, predominant rock type, type of internal support, earthquake magnitude, epicentral distance, peak ground acceleration. Figure 3 shows as example a typical plot for the influence of the epicentral distance and surface Peak Ground Acceleration (PGA) on tunnel performance, in terms of damage level: none, slight, moderate, heavy.

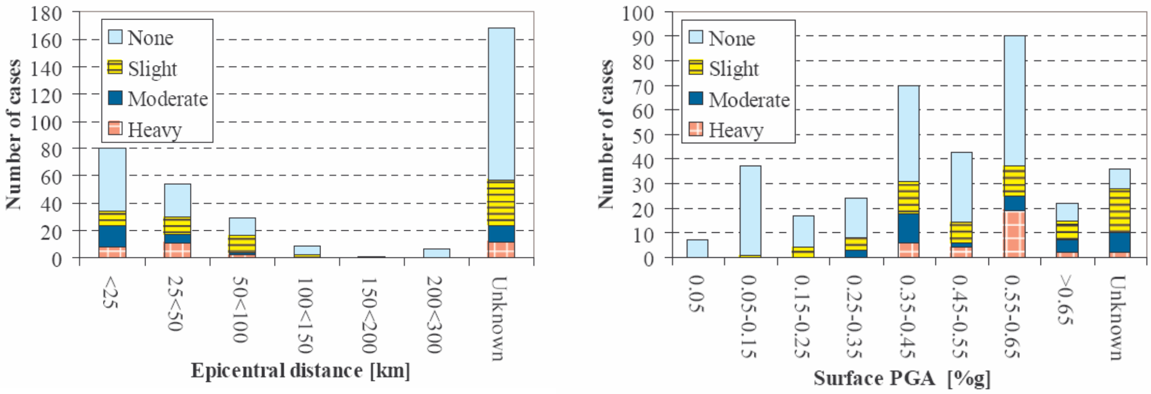


Figure 3. Effects of epicentral distance (left) an Peak Ground Acceleration (PGA) on damage (right).

4.2 Seismic vulnerability assessment and fragility curves

A useful tool for assessing the seismic vulnerability of underground structures is represented by the fragility curves. The seismic vulnerability of a structure can be defined as its susceptibility to be damaged by a ground shaking of a given intensity (Crowley et al., 2006). A fragility curve represents the relation between the probability of achieving a specified level of damage for a prescribed level of seismic hazard. Relatively few families of fragility curves have been derived for underground structures (ATC, 1985; ASCE, 1985; ALA, 2001; FEMA and NIBS, 2004). In general, the Peak Ground Acceleration (PGA) is taken as a measure of ground shaking intensity.

As already shown in Figure 3, PGA shows a poor correlation with the damage potential of ground motion. As a consequence, the intensity of ground motion shaking for assessing seismic vulnerability of underground structures has been quantified in terms of PGV at the free surface. Since a direct measurement of PGV at the location where the tunnel is damaged is generally not available, the values of this ground motion parameter can be back-calculated through an attenuation relationship purposely developed to predict PGV in near-fault conditions. Among the different attenuation relationships proposed, that given by Bray & Rodriguez-Marek (2004), which accounts also for site conditions, has been used.

Fragility curves for deep tunnels based on the empirical approach have been developed. The fragility curves are modelled as log-normal statistical distribution functions which give the probability of reaching or exceeding different states of damage for a given level of ground motion intensity. Each fragility curve is characterized by a median value of ground motion (x_{50}) with an associated log-normal standard deviation σ. The functional form of the probability distribution is illustrated in Figure 4 below which gives the Cumulative Distribution Function (CDF) versus the Peak Ground Velocity (PGV).

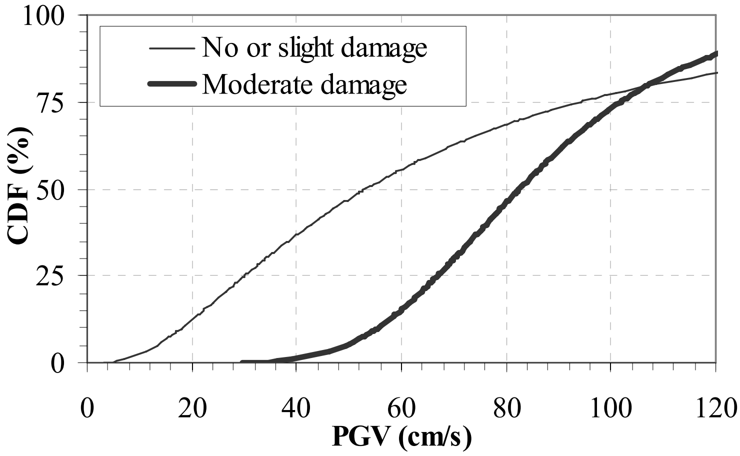


Figure 4. Fragility curves corresponding to states of "*no damage*" or "*slight damage*" and "*moderate damage*".

4.3 Simplified approach for seismic design of tunnels

4.3.1 Analysis of transversal response (ovaling deformations)

A simplified approach for the seismic design of tunnels subjected to strong ground shaking such as occurring at sites located in the vicinity of a causative fault has been formulated. The transversal response (ovaling deformation) has been considered by taking a lined circular

tunnel in plane strain conditions. The earthquake loading has been modelled as a uniform, quasi-static strain field simulating a pure shear deformation (Figure 5).
The relations for displacements, bending moment, thrust and shear forces have been derived following the same approach as used by Einstein and Schwartz (1979), in which however the assumption that the induced internal forces are caused by excavation has been removed and replaced with an imposed, external quasi-static loading distribution. The solution has been obtained for two contact conditions at the structure-rock interface, full-slip and no-slip (Corigliano et al., 2007 b, c; Barla et al., 2008; Corigliano et al., 2009):

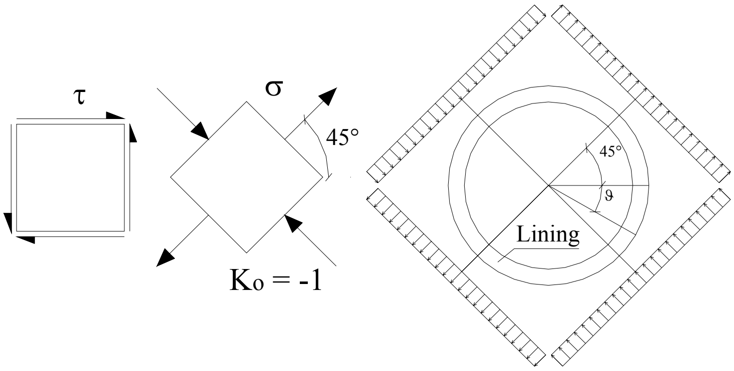


Figure 5. State of stress corresponding to a uniform, pure shear deformation.

4.3.2 Analysis of longitudinal response (axial and bending deformations)

Also considered has been the tunnel response along the longitudinal direction (which involves axial and bending deformations). A FEM model has been purposely developed by subdividing the tunnel into a finite number of beam elements with lumped mass, connected to the surrounding ground by a series of frequency-dependent springs and dashpots in parallel (i.e. a Kelvin-Voigt model, Figure 6) representing the effects of ground deformability and energy dissipation.
Wave scattering or kinematic interaction is not accounted for and thus this model can be ascribed to the class of simplified dynamic methods to analyse underground structures. The seismic excitation is input at the external nodes of the Kelvin-Voigt model through appropriate three-component free-field displacement and velocity time-histories. The synthetic records are generated using the Hisada and Bielak (2003) approach which allows to properly model not only the spatial variability and phase shift of ground motion but also the typical features of near fault ground motion (i.e. directivity and flying step effects).

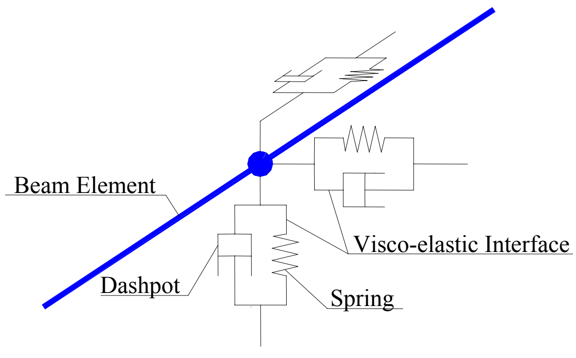


Figure 6. Spatial beam element model connected by means of Kelvin-Voigt elements to the surrounding ground.

4.4 Advanced numerical methods for seismic design of tunnels

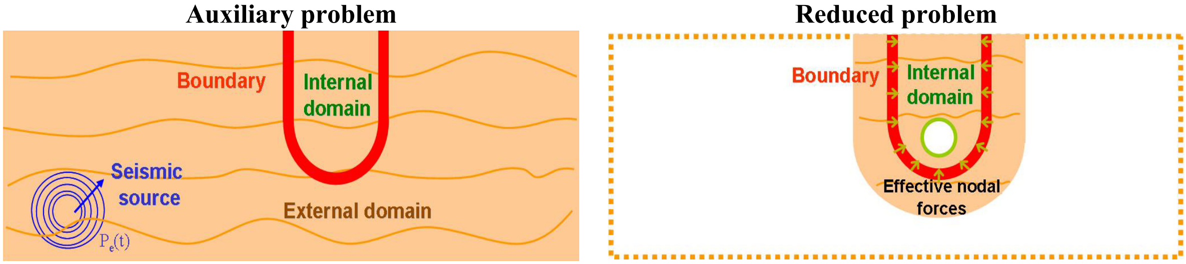
Consideration has been given to the use of advanced numerical methods to perform a complete analysis of the tunnel problem in seismic conditions, which includes the simultaneous consideration of the seismic source, the propagation path, the geological/geotechnical site conditions, and soil-structure interaction. The Domain Reduction Method (Bielak et al., 2003), a powerful sub-structuring approach, has been applied to reduce the computational efforts required by such a large scale numerical problem.

The Domain Reduction Method (DRM) implies the subdivision of the original problem into two simpler problems each one solved in two independent steps (Figure 7). The first problem accounts for earthquake source and propagation path and it is solved with a model that includes both the source and a background structure (*external domain*) from which the structure has been removed and replaced by the same material as the surrounding soil. The second problem simulates with the desired accuracy only a region (*internal domain*) of the domain of interest which includes the structure and the surrounding soil, but not the causative fault.

The input for the solution of the reduced problem is a set of effective nodal forces calculated from solving the auxiliary problem and applied in a narrow strip of elements (see the dark boundary in Figure 7). These forces are equivalent to (and replace the) original seismic forces computed in the first step. A detailed description of the implementation of the DRM for applications with the Spectral Elements Method can be found in Faccioli et al. (2005) and Stupazzini et al. (2006).

The main feature of this approach is the possibility of coupling solutions obtained by different methods in two different domains. In this study the auxiliary problem (*external domain*) has been solved using a 3D model by the semi-analytical method developed by Hisada and Bielak (2003). The reduced problem (*internal domain*) has been instead solved by means of a 2D numerical analysis using the Spectral Element Method (SEM).

DRM STEP I (3D Free-field analysis) **Auxiliary problem** | **DRM STEP II** (2D SEM analysis) **Reduced problem**



a) **b)**

Figure 7. Domain Reduction Method (DRM) for advanced numerical analysis of the tunnel problem.

The above numerical method has been used by taking as example the Serro Montefalco tunnel (railway switch line Caserta - Foggia), in the northern sector of the Southern Apennines, one of the most active seismic regions in Italy. The rock masses include clay-shales, marl and marly limestone and clay and marl intercalated with limestone (Barla et al., 1986). The results obtained from the advanced numerical analyses performed have been compared with those derived with the developed closed-form solution (the maximum shear strain obtained is □ff max = 1.39·10-4).

Figure 8 shows the comparison for thrust force and bending moment, also see Corigliano et al. (2007 b, c). The agreement between the two solutions is satisfactory. In particular, the thrust force computed using the numerical analysis exceeds that calculated by the simplified method by approximately 30%. It is important to remark that the bending moment and the thrust force illustrated in

Figure represent the dynamic increment of the internal forces. The total moment and thrust force are obtained by adding the dynamic increments to the static values as shown in Figure 9.

4.5 Case Studies

In addition to the Serro Montefalco tunnel, briefly mentioned above, two case studies have been analysed (see Barla et al., 2008 a,b). The main interest of these studies has been to validate the approach proposed for design (from the use of simplified analysis methods to more advanced numerical methods) by performing back analyses of tunnels which underwent severe damages during a real earthquake. For this reason the decision was taken to analyse the *Pavoncelli tunnel*, i.e. the base tunnel of the Acquedotto Pugliese which, during the Irpinia earthquake of November 23,1980 suffered severe damages to the tunnel lining (Figure 10). Also, due to the interest in underground caverns, a comprehensive numerical study of the *Venaus Powerhouse*, a major component of the Pont Ventox - Susa hydroelectric scheme (Figure 11) has been carried out. With the geological and geotechnical data available (Barla, 2005; Barla et al., 2008), the analyses have been performed in both static and dynamic conditions by using the FEM method and the Midas-GTS code. Also analyses have been carried out with the Geo-ELSE code and DRM.

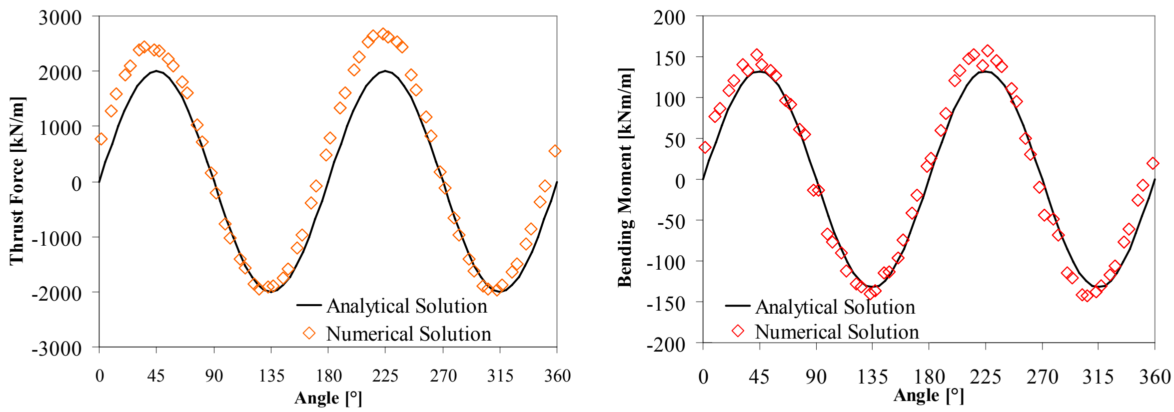
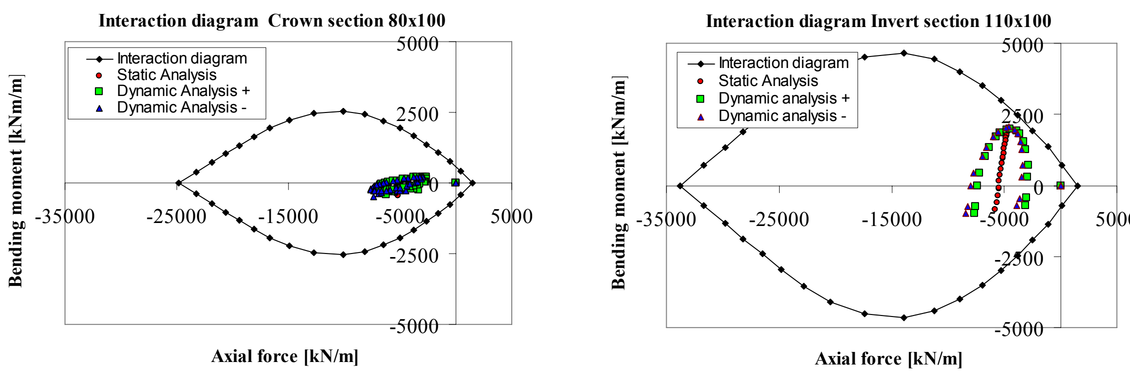
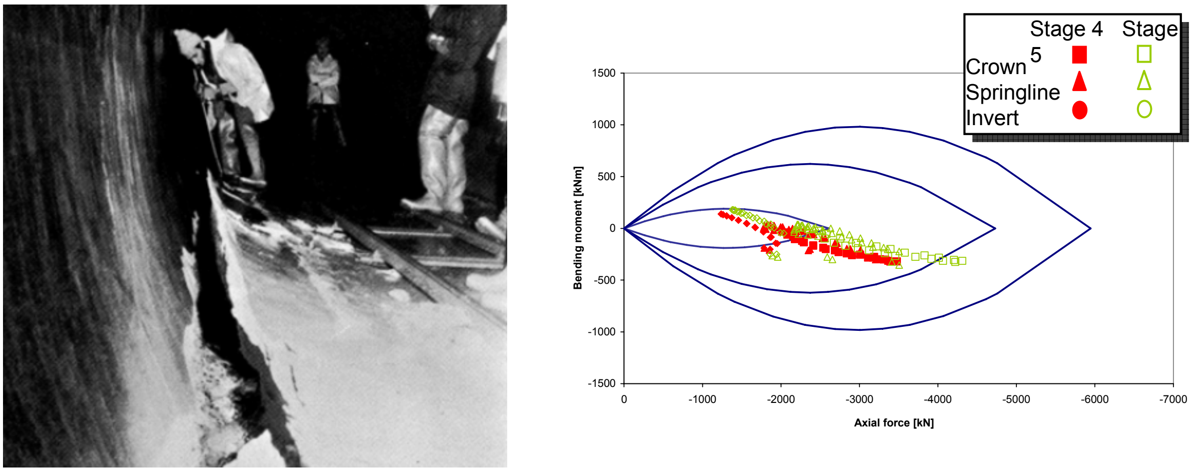


Figure 8. Results of advanced analysis and closed-form solution of the Serro Montefalco tunnel a) thrust force; b) bending moment.



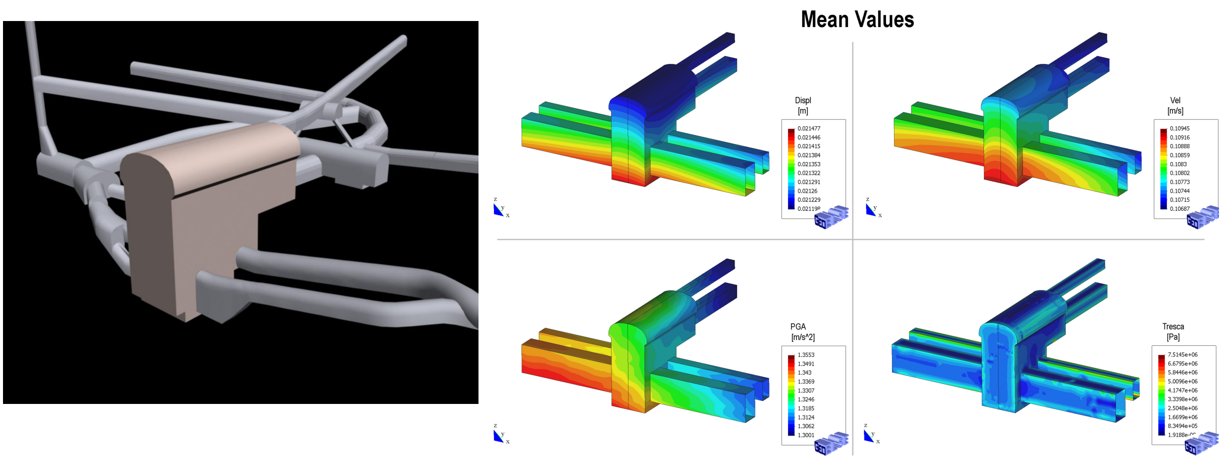
a) b)

Figure 9. Interaction diagram for the cross-section. a) crown and b) invert , with the dynamic stress increment added.



(a) (b)

Figure 10. (a) FEM model, lining detail. (b) Photograph of typical damages occurred during the 23rd November 1980 Irpino-Lucano earthquake.



(a) (b)

Figure 11. (a) Perspective view of the cavern complex. (b) A summary of results in terms of mean values of peak acceleration, velocity and displacement, as well as of the peak maximum shear stress.

5 DISCUSSION

The research programme briefly outlined in this overview has addressed a number of problems and issues dealing with deep tunnels and caverns in seismic conditions, with the main objective to provide appropriate computational methods for design analysis. A comprehensive review of the state of the art on the response of underground structures in seismic conditions has been provided, including an overview of the earthquake induced damages and methods which are presently available in the technical literature. On this basis, the effects of past earthquakes on underground structures have been identified so as to underline the most significant factors responsible for different types of damage: overburden depth, predominant rock type, type of internal support, earthquake magnitude, epicentral distance, and peak ground acceleration.

A closed-form solution for the analysis of the transversal response of the tunnel lining due to ovaling deformations has been developed. The earthquake loading is modelled as a uniform, quasi-static strain field simulating a pure shear deformation. A key parameter for computing stress is the maximum shear strain in free-field conditions. A model based on the Finite Element Method has also been developed to analyse the tunnel response along the longitudinal direction, which involves axial and bending deformations. The tunnel is subdivided into a finite number of beam elements with lumped mass, connected to the surrounding ground by a series of frequency-dependent springs and dashpots in parallel representing the effects of ground deformability and energy dissipation. Advanced numerical methods have been used for a complete analysis of the underground structure in seismic conditions, which includes the simultaneous consideration of the seismic source, the propagation path, the geological/geotechnical site conditions, and soil-structure interaction. The Domain Reduction Method (DRM) which implies the subdivision of the original problem into two simpler problems, each one solved in two independent steps, has been adopted. The first problem accounts for the earthquake source and propagation path and it includes both the source and a background structure (*external domain*) from which the structure has been removed and replaced by the surrounding soil. The second problem simulates the region (*internal domain*) of interest which includes the underground structure and the surrounding soil, but not the causative fault.

Case studies of tunnels have been considered with the main objectives to validate the closed-form solution by means of advanced numerical methods and to perform back analyses of tunnels which underwent severe damages during a real earthquake. Also, the case of an underground cavern, which is characterised for the high level of geological and geotechnical data available, has been taken to assess the most appropriate methods to be used for design of complex structures in seismic conditions.

With the above developments completed, a design process has been defined in line with the Design Norms for Construction (NTC 2008) recently promulgated in Italy (i.e. Preliminary Design, Intermediate Design, Final Design). As underlined also in a paper recently published (Corigliano, Lai and Barla, 2009), the following design procedure can be proposed:

Preliminary Design: a seismic vulnerability assessment of the underground structure can be performed by using Fragility Curves, which provide the designer with a tool for deciding whether a given underground structure need be studied in subsequent design stages with more refined design analyses.

Intermediate Design: a simplified approach is adopted which takes into account the interaction of the underground structure with the surrounding ground and at the same time considers the near-fault ground motion. This implies the use of validated closed form solutions, under the assumption of simplified constitutive laws for the rock mass and the support (continuum, elastic model), with the loading condition represented by the maximum shear strain in free-field conditions.

Final Design: more advanced computational methods are to be used for the design of tunnels, based on numerical methods and non linear constitutive laws for the rock mass and the support (continuum, elastic and elastoplastic models), such as the Finite Element and Finite Difference Methods, with attention being paid to both the transversal and longitudinal responses. Three dimensional complete numerical analyses need be considered for the case of complex structures such as Underground Caverns, depending on the level of complexities and the site conditions to be taken into account.

6 VISION AND DEVELOPMENTS

The research programme on underground structures in seismic conditions has analysed so far the vulnerability of tunnels and caverns, with the rock mass represented as a continuum/equivalent continuum. It is proposed to develop methods for the analysis of underground structures in rock masses represented as a discontinuum. The transition from continuum to discontinuum analysis is very important if intended for design analysis of underground excavations in seismic conditions. In general, this is the case of underground structures excavated in good quality rock mass conditions near to the ground surface or at depth, where the rock mass is better simulated as a discontinuum rather than a continuum/ equivalent continuum.

Analytical and numerical methods for design analysis of discontinuous media need be considered. Given that the methods most frequently used, in two-dimensional (2D) and thee-dimensional (3D) conditions, are for a continuum/equivalent continuum, the bibliographic studies will be concerned with discontinuum. On the one end, consideration will be given to DDA ((Discontinuous Deformation Analysis) and DEM (Discrete Element Method) methods in seismic conditions. On the other end, the attention will be posed on the available analytical solutions, which will be used for validation purposes.

In general, the presence of the discontinuities in a continuous medium is the cause of seismic waves attenuation and delay of the arrival time, due to the onset of both reflected and refracted waves. Also to be accounted for is the presence of waves generated along each discontinuity (Stoneley's waves) which need be assessed in terms of its influence on the overall response of the discontinuous medium. It is important to underline that the study of wave propagation in a jointed medium is totally different from the study of wave propagation in a continuous medium. In fact, a joint comprises two mating surfaces which may undergo shear displacements so that the boundary conditions to be considered imply both the continuity of the joint surface stresses and the discontinuity of displacements. The problem, which is indeed rather complex in two dimensional (2D) conditions, becomes even more difficult if three dimensional (3D) conditions are considered.

7 REFERENCES

American Lifelines Alliance (2001). Seismic fragility formulations for water systems. Part 1 - Guideline and Part 2 -Appendices. ASCE-FEMA.

Applied Technology Council (1982). ATC-13 Earthquake Damage Evaluation Data for California Redwood City.

ASCE (1985). Evaluation of Seismic Fragility and Validation of Probabilistic Models. The Fragility Evaluation/Validation Subgroup Joint Working Group on Seismic PRA of the ASCE Dynamic Analysis Committee.

Barla G. (2005). Sviluppi nell'analisi progettuale delle opera in sotterraneo. Quinta Conferenza "Arrigo Croce". *Rivista Italiana di Geotecnica*, 3, pp. 11-67.

Barla G., Caruso G., Rondini G. (1986). La problematica delle gallerie nel raddoppio della linea ferroviaria Caserta-Foggia. Congresso Internazionale su Grandi Opere Sotterranee, Firenze, June 8-11.

Barla G., Fava A., Peri G. (2008). Design and construction of the Venaus powerhouse cavern in calcschists. Geomechanics and Tunnelling. 5, pp. 399-406.

Bielak J., Loukakis K., Hisada Y., Yoshimura C. (2003). Domain reduction method for three-dimensional earthquake modeling in localized regions. Part. 1: Theory. BSSA. 93 (2), pp. 817-824.

Crowley H., Pinho R., Bommer J.J., Bird J.F. (2006). Development of a displacement based method of earthquake loss assessment. European School for Advanced Studies in Reduction of Seismic Risk, Pavia, Research Report No. ROSE-2006/01.

Dowding C.H., Rozen, A. (1978). Damage to rock tunnels from earthquake shaking, *Journal of the Geotechnical Engineering Division*, 104, pp.175-191.

Einstein H. H. and Schwartz C. W. (1980). Simplified analysis for tunnel support, *Journal of the geotechnical engineering division*, 105, pp. 499-518.

Faccioli, E., Vanini M., Paolucci R., Stupazzini M. (2005). Comment on Domain reduction method for three-dimensional earthquake modelling in localized regions, Part I: Theory by J. Bielak, K.

Loukakis, Y. Hisada, C. Yoshimura, and Part II: Verification and applications by C. Yoshimura, J. Bielak, Y. Hisada, A. Fernández, *Bull. Seism. Soc. Am,* 95 (2), pp. 763-769.
FEMA 1999. HAZUS 99, Earthquake Loss Estimation Methodology. Technical Manual. Federal Emergency Management Agency, Washington D.C.
Hisada Y. (1995). An efficient method for computing Green's functions for a layered half-space with sources and receivers at close depths. *Bulletin of Seismological Society of America*, 85, pp.1080-1093.
Hisada Y., Bielak J. (2003). A theoretical method for computing near-fault ground motions in a layered half-spaces considering static offset due to surface faulting , with physical interpretation of fling step and rupture directivity. *Bulletin of Seismological Society of America*, 93, pp.1154-1168.
Huang T.H., Ho T.Y., Chang C.T., Yao X.L., Chang Q.D., Lee H.C. (1999). Quick investigation and assessment on tunnel structures afte earthquake, and the relevant reinforced methods. Report for the Public Construction Commission, Taipei, Taiwan.
Owen, G. N., Scholl, R. E. (1980). Earthquake engineering of large underground structures. Report prepared for the *Federal Highway Administration and National Science Foundation*, Report FHWA/RD-80/195.
RTRI (2001). Design Code for Tunnels in Urban Areas (in Japanese). Railway Technical Research Institute, Japan.
Sharma S., Judd W.R. (1991). Underground opening damage from earthquakes, *Engineering Geology*, 30, pp. 263-276.
Stupazzini M., Paolucci R., Scandella L., Vanini M. (2006). From the seismic source to the structural response: advanced modelling by the spectral element method. Proceedings of the First European Conference of Earthquake Engineering and Seismology, Genève, September 2006.
Wang W.L., Wang T.T., Su L.L., Lin C.H., Seng C.R. (2001). Assessment of damages in mountain tunnels due to the Taiwan Chi-Chi earthquake. *Tunnelling and Underground Space Technology,*16, pp. 133-150.

8 RELUIS REFERENCES

Barla G., Barla M., Perino A., Principito M. (2008). Soluzioni analitiche e numeriche nella progettazione sismica delle opere in sotterraneo. XII Ciclo di Conferenze di Meccanica e Ingegneria della Rocce. MIR 2008. Patron, Bologna.
Barla G., Mensi E., Perino A. (2008). Analisi degli effetti sismici nelle gallerie profonde: la galleria Pavoncelli dell'Acquedotto Pugliese. XII Ciclo di Conferenze di Meccanica e Ingegneria della Rocce. MIR 2008. Patron, Bologna.
Corigliano M., Lai C.G., Barla. G. (2006). Seismic response of rock tunnels in near-fault conditions. Proceedings of the First European Conference of Earthquake Engineering and Seismology, Genève, September 2006.
Corigliano M. (2007). Seismic response of deep tunnels in near-fault conditions. PhD Thesis. Politecnico di Torino.
Corigliano M., Scandella L., Barla G., Lai C., Paolucci R. (2007a). Seismic analysis of deep tunnels in rock: a case study in Southern Italy". 4th International Conference on Earthquake Geotechnical Engineering, Thessaloniki, Greece, 25-28 June 2007, paper 1616.
Corigliano M., Scandella L., Barla G., Lai C., Paolucci R. (2007b). Aspetti progettuali nell'analisi sismica di gallerie profonde. Convegno dell'Associazione Nazionale di Ingegneria Sismica. Anisid. Pisa, 10-13 Giugno 2007.
Corigliano M., Barla G., Lai C. (2007c). Seismic vulnerability of rock tunnels using fragility curves. 11th International Congress of ISRM. Lisbon, July 8-12, 2007.
Corigliano M., Lai C.G., Barla. G. (2009). Approcci semplificati nella progettazione sismica delle gallerie profonde. *Progettazione Sismica*, 1, pp. 105-133.

SUB-THEME 6.3: SLOPE STABILITY

1 INTRODUCTION

One of the main sources of seismic vulnerability in Italy is represented by the instability of slopes; therefore, this is a subject of great significance, particularly in view of the growing attention that has been recently dedicated to the reduction of seismic hazard.
The response of a slope under seismic loading is determined by the temporal and spatial distribution of the seismic forces in the soil mass, which in turn depend on the characteristics of the seismic input and on the mechanical properties of the soil.
A number of different techniques exist to address this problem, each implying some level of approximation. Experience of the use of advanced numerical analysis is still somewhat limited, and it seems difficult to generalise the results of such complex analyses. An advisable approach would be that of carrying out the analysis of the same problem using a number of approaches characterised by different levels of complexity, in order to assess the reliability and robustness of the different procedures. In the present research project, different research groups were given the task of pursuing the study of the seismic behaviour of slopes using a number of different approaches, and investigating the possibility of using the results of the more advanced analysis as a guidance for a sound and reliable use of the simplest and most common analysis method, that still form the backbone of professional practice.

2 BACKGROUND AND MOTIVATION

The simplest and most common analysis method is the pseudo-static approach in which the earthquake inertia forces are taken to be equivalent to constant static forces. Basically, the ingredients of this method are the amount of the constant forces that simulate the seismic behaviour, and the safety coefficients (or partial factors) that can be deemed adequate for the stability checks to be satisfied. As this approach neglect completely the transient and cyclic nature of the seismic forces, it must be considered as a purely conventional tool, and these input quantities do not have a physical meaning per se, but need to be calibrated against the results of more realistic approaches.
A better representation of the seismic behaviour of a slope can be obtained by the displacement method, originally developed by Newmark (1965) for the seismic analysis of earth dams. This method removes the assumption of temporal invariance of the seismic forces, and evaluates the permanent displacements of the sliding mass, regarded as a rigid body, produced by the acceleration in excess of that sufficient for the complete mobilisation of the soil strength along a sliding surface. Different adaptations of the displacement method do exist, in which the decay of the soil strength parameter or the development of the excess pore water pressures during the earthquake are included in the analysis. An important modification of the method consists in considering explicitly the deformability of the soil, through a decoupled analysis, in which a preliminary analysis is carried out to assess the dynamic response of the slope, and then the resulting acceleration time histories are used to compute the permanent displacements with a rigid block sliding analysis. Although this method inconsistently assumes that there are no permanent displacements in the seismic response analysis, and then uses the analysis results to calculate a seismically induced permanent displacement, it was found to provide a reasonable estimate of seismic displacements for many cases.

More advanced numerical analyses of the seismic behaviour of a slope can indeed be performed, using a coupled approach in which a full description of the dynamic behaviour of the soil is explicitly introduced. While these analysis is conceptually more satisfactory if applied to a well documented case-study, they result somewhat cumbersome in helping the development of simplified procedure, which is specifically the aim of the present research project, which requires the analyses of a large number of simplified slope schemes, subjected to a variety of seismic loading. Therefore, it was felt that the bulk of the results needed for the calibration of the pseudo-static method were to be obtained using the displacement method, more suitable for parametric studies. In applying the displacement method however, several different levels of sophistication were inserted, including two-dimensional amplification effects and the consideration of the deformability o the soil mass. A number of advanced, fully coupled numerical analyses were performed as well, to further corroborate the results obtained.

3 RESEARCH STRUCTURE

The following steps summarise the organisation of the research project

1) obtain a large and reliable database of input seismic recording that may be used in the parametric study; this was accomplished by re-examining all the available Italian acceleration records and organising the results in a new database for access via the internet;

2) perform the integration of the Italian accelerograms, using a rigid-block displacement approach, for the development of empirical relationships between the displacements and a number of motion parameters;

3) evaluate the relationships between the displacements computed with the displacement approach and the seismic coefficients to use in a pseudo-static approach, still considering an infinitely rigid sliding mass, and linking the pseudo-static seismic coefficient to the requested seismic performances.

4) study the seismic response and evaluate the permanent displacements of idealised slopes to a large number of seismic inputs, in one-dimensional conditions, taking into account in a parametric way the soil deformability and considering the coupling of the propagation through the soil mass and the sliding of the soil mass;

5) carry out a number of two-dimensional analyses of idealised slopes, subjected to a selected number of seismic inputs, accounting for the soil deformability and evaluate the permanent displacement through a decoupled approach;

6) from the above steps (4) and (5) find the corrections to apply to the seismic coefficients found at point (3) to account for soil deformability;

7) perform a limited number of advanced numerical analyses to corroborate the results obtained with the previous approach.

4 MAIN RESULTS

4.1 Database of Italian strong-motion accelerograms

A database of Italian strong motion accelerograms was developed as well as a site and source databank. The database consists in 247 Italian three-component corrected accelerograms obtained from 101 recording sites and related to 89 earthquakes that occurred in the period 1972-2002 (Scasserra et al., 2008d).

Available acceleration time histories were uniformly processed by the same team of seismologists, supervised by Dr. Silva, that is responsible for the PEER (Pacific Earthquake Engineering Center) data processing.

Site conditions at the 101 monitoring stations were identified through both geologic characteristics and values of shear wave velocities.

Velocity measurements are nowadays provided for 51 of the 101 sites. For the remaining sites, the equivalent shear wave velocity in the upper 30 m (V_{S30}), as defined by Eurocode 8, (EN 1998-5) was estimated using a hybrid approach, based on correlations with surface geology (Scasserra et al., 2008c).

A web site was created to disseminate the strong motion database and the source and site databank (Scasserra et al., 2008a). It is called SISMA, that stands for Site of Italian Strong Motion Accelerograms (http://sisma.dsg.uniroma1.it); it can be anticipated that the SISMA dataset will be archived also at the PEER web site. The principal objective of SISMA website is to provide high quality Italian strong motion records with consistent and reliable evaluation of associated seismic parameters.

The structure of SISMA allows records to be located in several ways, depending upon the user interest.

4.2 Empirical relationships for assessing earthquake-induced displacements

Empirical relationships were developed for evaluating permanent co-seismic slope displacements using records from Italian seismic events. Database SISMA was used to this purpose to obtain a homogeneous set of accelerograms with consistent and reliable estimates of the associated seismic parameters. According to the values of shear wave velocity in the subsoil underlying the monitoring station, acceleration time histories on rock or rock-like soils (class A - $V_{S,30} > 800$ m/s) were distinguished from those on stiff soils (class B - $V_{S,30}$ = 360 – 800 m/s) and medium to soft soils (classes C, D and E - $V_{S,30} < 360$ m/s). A total number of 196 free-field horizontal acceleration time histories were selected, these being related to 46 Italian earthquakes with magnitudes in the range 4 to 6.3.

For each accelerogram, the Newmark-type displacements were computed considering values of the critical acceleration a_y = 0.002, 0.005, 0.01, 0.02, 0.05, 0.1, 0.2 g. For a given critical seismic coefficient $k_y = a_y/g$, two values of displacement were determined considering each side of the acceleration time history. The maximum of the two obtained values were assumed as the displacement related to the signal.

The proposed relationships allow permanent slope displacement associated to 50th and 90th percentile to be evaluated as a function of: (1) critical acceleration ratio (a_y/a_{max}); (2) ratio of squared peak ground velocity to peak ground acceleration of input motion and critical acceleration ratio (PGV^2/PGA, a_y/a_{max}); (3) Arias intensity and critical acceleration ratio (I_A, a_y/a_{max}); (4) destructiveness potential factor and critical acceleration ratio (P_D, a_y/a_{max}); (5) mean period and critical acceleration ratio (T_m, a_y/a_{max}) (Madiai, 2009).

The obtained relationships can be used for a preliminary estimate of permanent displacements induced by seismic shaking.

4.3 Evaluation of slope displacements for multi-block sliding mechanisms

In common practice, earthquake-induced displacements are simply evaluated for translational or rotational sliding mechanisms. When the sliding surface is of general shape, permanent displacements can be evaluated either best fitting the sliding surface with circular or logarithmic spiral surfaces or describing the mixed roto-translational mechanism via a multi-block model; the latter provides a better representation of actual sliding mechanism thus yielding more reliable predictions of slope behaviour under seismic conditions.

In the work by Bandini et al. (2008), the transformation rules for slope geometry were selected to meet the following criteria: (1) each block keeps in contact with adjacent blocks and with the slip surface without separation or overlapping; (2) the total mass of the system is constant. If slope displacement is small with respect to the length of the slope, the effect of mass-transfer is negligible and it can be assumed that each block slides as a rigid body along its portion of slip surface without any mass-change. For large displacements the substantial change in the geometry of the slope and the consequent change in soil mass distribution cannot be neglected. In fact, geometrical rearrangement leads to a more stable configuration and is often the main factor yielding the slide eventually to stop.

In the proposed approach, pore pressure build-up in cohesive soils and change in slope geometry were accounted for in the analysis; the Global Limit Equilibrium Method of slices, as proposed by Fredlund and Khran (1977), was used for evaluating the critical seismic coefficient k_y.

The proposed GLE-based multi-block model was applied to parametric studies of several ideal schemes of slope and to studies of landslides reactivated by the Irpinia-earthquake of 1980 ($M = 6.9$).

4.4 Pseudo-static approach and evaluation of equivalent seismic coefficient

Displacement-based sliding block methods are an useful and handle tool for evaluating the response of slopes and earth structures to earthquake loading, having indeed the advantage of providing a quantitative assessment of earthquake-induced displacements using simple analytical procedures. However, the pseudostatic approach is still the most diffused method adopted in common practice. In this method the seismic coefficient designates the horizontal force to be used in the stability analysis, its selection being thus crucial. Starting from Seed (1979), a number of procedures have been proposed in the past in which the pseudostatic approach is calibrated to a particular level of slope performance, which is represented by the earthquake-induced displacement.

Following a similar approach, a procedure has been recently developed by Rampello et al. (2008), in which the horizontal seismic coefficient k and the corresponding safety factor F_S are evaluated using an equivalence with the results of a parametric application of the displacement method, as proposed originally by Newmark (1965). In the procedure, the seismic coefficient is expressed as a function of the maximum acceleration of the slide mass (k_{max}), the ratio of slope resistance to peak demand k_y/k_{max} and the limit displacement d_y considered as tolerable for the slope. Slope stability is satisfied for values of the safety factor $F_S \geq 1.0$. The procedure can be thought of as being related to earthquake magnitudes $M = 4$–6.5, typical of Italian seismic events.

The permanent displacements induced by an acceleration time history can be expressed as a function of the ratio k_y/k_{max}.

The equivalent seismic coefficient can then be defined as a fraction η of the maximum acceleration a_{max} of the slide mass:

$$k = \eta \cdot k_{max} = \eta \cdot \frac{a_{max}}{g} \quad (1)$$

where η decreases as the displacement tolerable for the slope increases. In principle, the application of the displacement method should be performed using the equivalent accelerograms acting in the sliding mass, as obtained by one or bi-dimensional seismic response analyses (Seed and Martin, 1966; Chopra, 1966).

In the proposed approach a rigid soil behaviour was assumed for the slide mass, this implying that, until full mobilisation of shear strength, the acceleration distribution is uniform through the soil mass. Under this assumption, amplification effects are simply taken into account using amplification factors for subsoil profile S_S and ground surface topography S_T, as specified by technical recommendations or building codes (e.g.: EN 1998-5, D.M. 14.01.2008); in eq. (1) it is then $a_{max} = S_S \cdot S_T \cdot a_g$, with a_g being the maximum acceleration at the rigid outcrop.

Permanent displacement were evaluated through a Newmark-type integration of the portion of the accelerograms in excess of k_y. A total of 214 accelerograms were used belonging to 47 events recorded by 58 stations. They were divided in three groups according to the subsoil underlying the monitoring sites, as defined by Eurocode 8 and the Italian building code (EN 1998-5; D.M. 14.01.2008): rock or rock-like soil with shear wave velocity $V_S \geq 800$ m/s (A); dense granular and stiff cohesive soil with $V_S = 360–800$ m/s (B); medium to loose granular and medium stiff to soft cohesive soil with $V_S < 360$ m/s (C, D; E).

For each group of accelerograms, peak accelerations were scaled to values of $a_{max} = 0.05$, 0.15, 0.25 and 0.35 g, limiting the scale factors in the range 0.5 – 2. Earthquake-induced displacements were computed integrating twice the equation of relative motion for translational sliding, using critical acceleration values equal to 10 to 80 % of the maximum acceleration ($k_y/k_{max} = 0.1 – 0.8$). Permanent displacements computed for each subsoil class and for each level of acceleration were plotted as a function of the ratio k_y/k_{max} in a semi-logarithmic scale. An example is shown in Figure 12 for subsoil class B. Computed results were best-fitted using exponential relationships written in the form:

$$d = B \cdot e^{A \frac{k_y}{k_{max}}} \quad (2)$$

Assuming a log-normal distribution around the mean value, the 90th-percentile upper-bound displacements were obtained, their relationships being characterised by the same parameter A of the mean curves and by a value of $B_1 > B$.

At a constant critical acceleration ratio, permanent displacements induced by a given accelerogram, linearly depends on a_{max}. Then it is possible to account for eventual amplification in ground motion, as produced by seismic response, multiplying the coefficient B_1 by the amplification factors S_S and S_T, thus obtaining $B_2 = S_S \cdot S_T \cdot B_1$.

For a given threshold displacement d_y, the corresponding values of η can then be obtained by inverting eq. (2) with $d = d_y$ and $B = B_2$:

$$\eta = \frac{k_y}{k_{max}} = \frac{\ln(d_y/B_2)}{A} \quad (3)$$

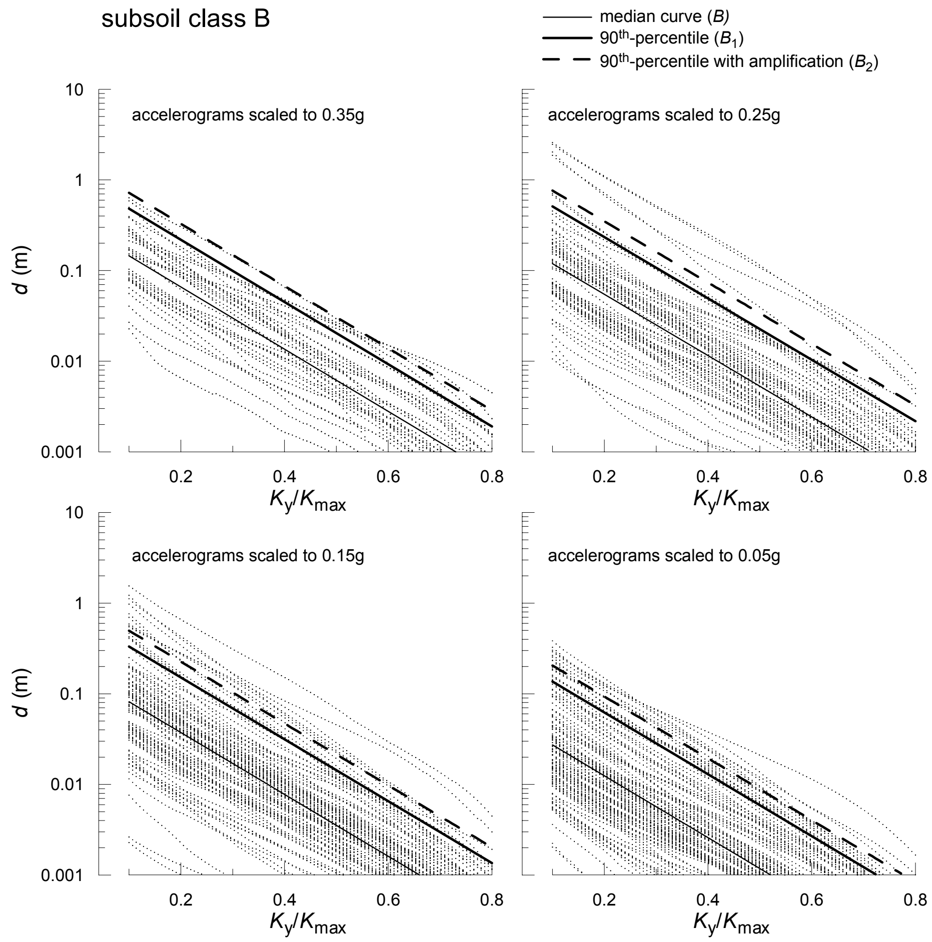


Figure 12. Permanent displacements computed using acceleration time histories monitored on class B subsoil (Rampello et al. 2008).

Values of η are given by Rampello and Callisto (2008) for threshold displacements of 15, 20 and 30 cm, corresponding to levels of damage from moderate to negligible (Idriss, 1985).

Ausilio et al. (2007b) developed a statistical model using a selection of accelerograms from database SISMA (Scasserra et al., 2008a) to compute the Newmark-type displacements. A significant reduction in the scatter of data set was observed once the displacements d were normalised by the product $a_{max} \cdot T_m \cdot D_{5\text{-}95}$ and plotted against the ration k_y/k_{max}. Considering the 90th-percentile upper-bound relationship for non-dimensional displacements, an expression for coefficient η' was obtained by Ausilio et al. (2007b), defined as the ratio a_y/a_g ($= k_y/k_g$):

$$\eta' = \frac{a_y}{a_g} = \frac{1}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{a_g \cdot E[T_m \cdot D_{5-95}]} \right) \right] + 0.237 \tag{5}$$

in which the apex was added to distinguish this definition by that given in eq. (3), and $E[T_m \cdot D_{5\text{-}95}]$ is the expected value of statistic distribution of the product $T_m \cdot D_{5\text{-}95}$ considered as a random variable.

In this case also, ground motion amplification due to site effects can be accounted for using the amplification factors for subsoil profile S_S and ground surface topography S_T, as specified by technical recommendations or building codes (e.g.: EN 1998-5, D.M. 14.01.2008):

$$k = \eta' \cdot k_{\max} = \eta' \cdot \frac{a_{\max}}{g} = \eta' \cdot S_S S_T \frac{a_g}{g} \tag{6}$$

4.5 Influence of the deformability of the soil mass

In the procedures discussed above a rigid soil behaviour was assumed for the slide mass, this implying uniform spatial distribution of acceleration within the slope, with $a(t) = a_g(t)$. More realistic description of seismic performance of slopes can be obtained taking into account the deformable behaviour of the slide mass during ground motion. In this case, slope behaviour can be studied using the de-coupled approach, in which a preliminary analysis is carried out to assess the dynamic response of the slope, and then the resulting acceleration time histories are used to compute the permanent displacements with a rigid-block sliding analysis (Makdisi and Seed, 1978). The seismic response analyses can be carried out in one-dimensional (1D) or two-dimensional (2D) conditions, and the non linear soil behaviour can often be described through the equivalent linear approximation, that is known to yield a reasonable estimate of soil response at moderate levels of seismic intensity.

When a seismic response analysis is performed, the maximum acceleration $a_{\max}$ in eq. (1) should be intended as the maximum equivalent acceleration $a_{(eq)max}$.

A parametric study was carried out by Ausilio et al. (2007a) in which 1D seismic response analyses were carried for soil columns of height H = 5 – 30 m, these representing the depths of the sliding surface, and bedrock depths of 5 to 60 m. The 1D equivalent linear visco-elastic analyses were carried selecting 124 Italian accelerograms from database SISMA (Scasserra et al., 2008a) and assuming 21 different subsoil profiles, representing subsoil classes as identified by Eurocode 8 (EN 1998-5) and by Italian Building Code (D.M. 14.01.2008).

The Authors recognised that the ratio $\alpha = a_{eq(max)}/a_s$, plotted in Figure 13 against T_s/T_m, could be interpolated by a single relationship irrespective of the subsoil profile:

$$\alpha = \frac{a_{(eq)\max}}{a_s} = \frac{a_{(eq)\max}}{a_g \cdot S_{NL}} = 0.4199 \cdot \left(\frac{T_s}{T_m}\right)^{-0.815} \tag{7}$$

The ratio between the wavelengths characteristics of the seismic event and the thickness of soil column decreases with the ratio T_s/T_m; as a consequence, the inertial forces reduce substantially yielding to equivalent accelerations lower than those computed at the ground surface. Then, values of $a_{(eq)max}/a_s$ lower than unity are due to vertical incoherence of ground motion, implicitly accounted for into $a_{(eq)max}$.

Using the results of the analyses mentioned above, Ausilio et al. (2007b) included the influence of soil deformability in the estimate of the equivalent seismic coefficient. In this case, the maximum acceleration $a_{\max}$ in eq. (1) is properly intended as the maximum equivalent acceleration $a_{(eq)max}$ of the slide mass, expressed in the form: $a_{(eq)max} = \alpha \cdot a_s = \alpha \cdot S_{NL} \cdot a_g$. To maintain a simplified and conservative character of the procedure, the Authors assumed a constant value of α = 0.74 computed for T_s/T_m = 0.5. Under these assumption, starting from eq. (5) they proposed a reduction factor η'' that include ground motion amplification produced by response analysis:

$$\eta'' = \frac{a_y}{a_g} = \frac{0.74 \cdot S_{NL}}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{0.74 \cdot S_{NL} \cdot a_g \cdot E[T_m \cdot D_{5-95}]}\right)\right] + 0.237 \tag{8}$$

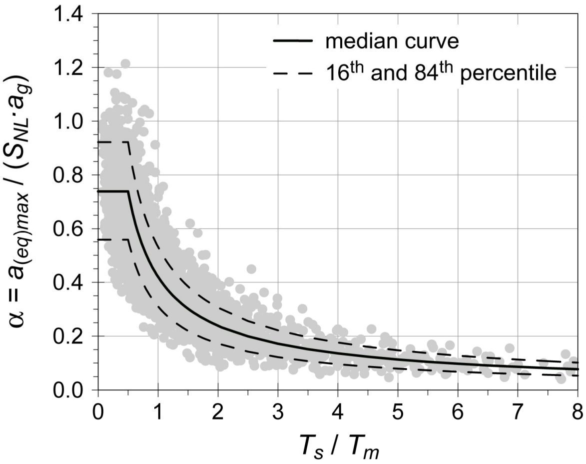


Figure 13. Ratio of $a_{(eq)max}/a_s$ versus the ratio between the fundamental period of slope and the mean period of input motion (Ausilio et al. 2007a).

In this case, the equivalent seismic coefficient can be written in the form:

$$k = \eta'' \cdot k_{\max} = \eta'' \cdot \frac{a_{\max}}{g} = \eta'' \cdot S_T \frac{a_g}{g} \tag{9}$$

Two-dimensional ground response analyses were also carried out to evaluate the influence of soil deformability on the seismic response of slopes and earth dams. The analyses were performed assuming the soil to behave as a linear elastic material, and assuming rotational sliding mechanisms.

The dynamic linear-elastic seismic response analyses were carried out investigating the influence of some key factors, such as the slope height and inclination, the soil stiffness E' and the frequency content of the seismic input.

Time histories of the equivalent seismic coefficient $k_{hS}(t) = a_{eq}(t)/g$, related to specified soil volumes, were evaluated as mentioned above for two-dimensional conditions.

In general, the maximum values of the equivalent seismic coefficient k_{hS} computed for given soil volumes were seen to be substantially lower than the ratio PGA/g irrespective of slope geometry and of depth and length of the sliding surfaces, at least for values of E' typical of soil deposits.

Influence of spatial incoherence of ground motion was also studied for two earth dams through coupled, effective stress dynamic analyses. For Camastra earth dam seven accelerograms, representative of possible seismic scenarios associated to different return periods T_R, were selected from the European Strong Ground Motion Database (Bilotta et al., 2008). For El Infiernillo earth dam, analyses were carried out using accelerograms from three earthquakes actually occurred at the dam site. The occurrence of asynchronous motion was detected plotting the contours of total stresses and instantaneous accelerations (Sica et al., 2008; Sica and Pagano, 2009).

The ratio $a_{hS,equiv}(t)/PGA$ computed in the two dams for all the analysed surfaces and seismic events decreases with increasing size of slide mass, as quantified by the maximum depth of the surface and by its length. Also, computed values of k_{hS} increase with peak input acceleration, but with a decreasing rate. This occurrence can be attributed to plastic soil behaviour within the dam embankment for high intensity earthquakes.

Sliding mechanisms parallel to the ground surface were also studied using the decoupled approach, for several idealised clayey slopes and a number of different seismic inputs. In the analyses, performed with the finite element method using the code QUAKE, an equivalent linear visco-elastic soil model was used in which the laws for decay of the secant shear modulus and for the increase of the damping ratio were taken from Vucetic and Dobry (1991) and a plasticity index $I_P = 25\%$.

The ground surface of the analysed slopes had a constant inclination $\alpha = 10°$ and 20° with a vertical distance from a horizontal bedrock varying from 15 to about 120 m. The computed displacements were related to the soil stiffness, to the size and location of the sliding mass within the slope, and to the properties of the seismic input, to support the prediction of earthquake-induced displacements for this category of sliding mechanisms (Rampello et al., 2009).

It was shown that for soft soil deposits the use of equivalent accelerograms globally results in lower displacements than those computed using the input accelerograms. For stiff soils, larger displacements than the ones computed using the input accelerogram can be obtained only for shallow and short sliding mechanisms, that is for small volumes of the sliding mass.

4.6 Advanced numerical analyses

Coupled effective stress FE analyses were carried out to study the seismic behaviour of an ideal slope, in a clayey soil deposit, with a height $H = 20$ m. The two-surface elasto-plastic constitutive model MSS, that is capable of describing the main features of soil behaviour under cyclic loading, was used in the analyses. Two sets of analyses were carried out, the first referring to a soft clay deposit, while the second to a stiff and overconsolidated clay deposit. A parametric study was performed for evaluating the influence of the overconsolidation ratio, the bedrock depth, the peak acceleration of input ground motion and the geometry of the mesh.

The main advantages (Amorosi et al., 2009b) of coupled effective stress FE dynamic analyses of a slope are represented by the possibility of: *i*) describing with accuracy shear stiffness decay with shear strain and the related hysteretic damping; *ii*) limiting the amount of the fictitious viscous damping usually added in numerical analyses (1-2 %); *iii*) predicting plastic strain accumulation and excess pore water pressure build-up during earthquake loading; *iv*) describing the dissipation of the excess pore water pressures after the end of the seismic action; (*v*) describing the evolution of strain and displacement fields during and after the earthquake.

5 DISCUSSION

The objective of developing a simplified procedure for a reliable evaluation of the seismic performance of a slope was pursued by means of a number of tools, each referring essentially to a parametric evaluation of the permanent displacements of simplified slope schemes. Firstly, a database of Italian strong motion database was developed together with a site and source databank, to provide high quality Italian strong motion records with consistent and reliable associated seismic parameters.

Using this database, a number of empirical relationships were developed for a preliminary estimate of earthquake-induced slope displacements, and a code was developed for evaluating slope displacements induced by seismic shaking along sliding surfaces of general shape. In this code, the mixed roto-translational mechanism is described via a multi-block model and account is taken for pore water pressure build-up and mass transfer occurring during motion. The code was checked by back-analysing the landslides reactivated by Irpinia earthquake of 1980.

Then, a parametric Newark integration of the whole database was performed with the specific intent to evaluate, for a given seismic performance, the seismic coefficient to use in a pseudo-static calculation. This study was timely, as it was possible to include some results in a new version of the Italian building code (D.M. 14.01.2008).
The influence of soil deformability on slope response to earthquake loading was studied through one and two-dimensional equivalent linear visco-elastic analyses and with a limited number of two-dimensional fully coupled stress dynamic analyses. The analyses considered simple idealised slopes and a few real earth dams.
The soil deformability brings about two somewhat contrasting effects, namely the amplification of the base motion and the non uniform spatial distribution of the inertia forces. The latter effect may lead to a substantial reduction of these forces, and can prevail when the wave length of seismic motion become comparable or smaller than the size of the sliding mass, and this may happen in large deformable volumes of soil, and for high-frequency seismic signals. Conversely, ground motion amplification prevails for small and stiff sliding masses excited by low-frequency seismic signals, and near the top of natural slopes or the crest of earth embankments, because of multiple reflection of the seismic waves.
To sum up, it may be stated that most of the research objectives were attained and that the main factors affecting the slope response to earthquake loading were recognised, their influence being isolated and evaluated.

6 VISIONS AND DEVELOPMENTS

The present research activity was mainly addressed to the study of the seismic behaviour of natural slopes. Ideal schemes of slopes were studied and typical geometries were considered for the sliding mechanisms and the depth of the bedrock, while the stiffness and strength properties were typical of natural soils. A number of well documented case histories were also back-analysed. The main characters of the seismic behaviour of the slopes were highlighted, and simplified procedures for the study of their seismic stability were proposed, including a relationship between the pseudo-static seismic coefficients and the required seismic performance, quantified by the final permanent displacements.
This research activity finds a natural extension in the study of the seismic behaviour of artificial earth structure, and specifically of earth dams. These structure are typically made of compacted soil, characterised by a large stiffness, and may produce significant amplification of the inertial forces, because of the multiple reflection and focalisation of seismic waves near the crest. The research should distinguish between different types of earth dams: homogeneous, zoned with vertical impervious nucleus, and zoned with inclined impervious nucleus. The influence of the bedrock depth needs also be investigated, as in many cases this depth is not well documented and a degree of subjectivity does exist on the choice of the boundary to which the seismic motion should be applied.
Similarly to the case of the natural slopes, this study may be carried out using methods of analyses of increasing complexity, including the simple pseudo-static analysis. However, the singularity and importance of the earth dams calls for a larger role of the advanced numerical analyses. A number of centrifuge tests might b also devised, in which the above different categories could be reproduced to provide the experimental evidence for the calibration of the analyses. The results of this research might provide guidance for the design of new earth dams, but would also assist in the seismic evaluation of the many existing earth dams that were designed without accounting for earthquake loading.

7 MAIN REFERENCES

Ambraseys N.N., Menu J.M. (1988). "Earthquake-induced ground displacement". *Earthquake Engineering and Soil Dynamics*, 16: 985-1006.

Ambraseys N.N., Simpson K.A., Bommer J.J. (1996). "Prediction of horizontal response spectra in Europe". *Earthquake Engineering and Structural Dynamics*, 25(4): 371-400.

Aubry D., Hujeux J.C., Lassoudiere F., Meimon Y. (1982). "A double memory model with multiple mechanism for cyclic soil behaviour". *International Symposium on Numerical Models in Geomechanics*, Zurigh, 3-13

Aubry D., Modaressi A. (1996). "GEFDYN, Manuel Scientifique". Ecole Centrale Paris, http://www.mssmat.ecp.fr/sols/logiciels/gefdyn/notice_line/index.htm.

Bardet J.P., Ichii K. and Lin C.H. (2000). "EERA - A computer program for Equivalent-linear Earthquake site Response Analyses of layered soils deposits". User Manual, 2000.

Bray J.D., Rathje E.M., Augello A.J., Merry S.M. (1998). "Simplified seismic design procedure for geosynthetic-lined, solid-wasteland fills". *Geosynthetics International*, 5: 203–235.

Cai Z., Bathurst R.J. (1996). "Deterministic sliding block methods for estimating seismic displacements of earth structures". *Soil Dynamics and Earthquake Engineering*, 15(4): 255-268.

Chan A.H.C. (1995). "User Manual for DIANA SWANDYNE II". School of Engineering, University of Birmingham, UK.

Chopra A.K. (1966). "Earthquake effects on dams". Ph.D. Thesis, University of California, Berkley.

Cotecchia V., Del Prete M. (1984). "The reactivation of large flows in the parts of Southern Italy affected by the earthquake of November 1980, with reference to the evolutive mechanism". *Proc. of 4th Int. Symposium on Landslide*, Toronto, 1-6.

Crespellani T., Madiai C., Maugeri M. (1996). "Analisi di stabilità di un pendio in condizioni sismiche e post-sismiche". *Rivista Italiana di Geotecnica*, 1: 50-61.

Crespellani T., Madiai C., Vannucchi G. (1998). "Earthquake destructiveness potential factor and slope stability". *Géotechnique*, 48(3): 411- 419.

Decreto Ministeriale 14.1.2008. del Ministero delle Infrastrutture. "Nuove norme tecniche per le costruzioni". S.O. n. 30 alla G.U. del 4.2.2008, No. 29.

Del Prete M., Trisorio Liuzzi G. (1981). "Risultati dello studio preliminare della frana di Calitri (AV) mobilitata dal terremoto del 23 novembre 1980". *Geol. Appl. e Idrogeol.*, 1981, 21, 153-165.

EN 1998-5 (2002) - Eurocode 8: Design of Structures for Earthquake Resistance - Part 5: Foundations, retaining structures and geotechnical aspects. *CEN European Committee for Standardization*, Bruxelles, Belgium.

Fredlund D.G., Krahn J. (1977). "Comparison of slope stability methods of analysis". *Canadian Geotechnical Journal*, 14: 429-439.

Hutchinson J.N., Del Prete M. (1985). "Landslide at Calitri, Southern Apennine, reactivated by the earthquake of 23rd November 1980". *Geol. Appl. e Idrogeol.*, 20(1): 9-38.

Hynes-Griffin M.E. and Franklin A.G. (1984). „Rationalizing the Seismic Coefficient Method". *Miscellaneous Paper GL-84-13, Department of the Army, Waterways Experiment Station*, Vicksburg, MS.

Idriss I.M. (1985). "Evaluating seismic risk in engineering practice". *Proc. of the 11th Int. Con. of Soil Mechanics and Foundation Engineering*, S. Francisco, 1: 255-320.

Jibson R.W. (1993). "Predicting earthquake-induced landslide displacements using Newmark's sliding block analysis". *Transportation Research Record*, 1411: 9–17.

Jibson R.W. (2007). "Regression models for estimating coseismic landslide displacement". *Engineering Geology*, 91(2-4): 209-218.

Jibson R.W., Harp E.L., Michael J.M. (2000). "A method for producing digital probabilistic seismic landslide hazard maps". Engineering Geology, 58: 271–289.

Kavvadas M., Amorosi A. (2000). "A constitutive model for structured soils". *Geotéchnique*, 50 : 263-273.

Lanzo G., Scasserra G. (2009). "Calibration of EC8 site-dependent acceleration response spectra using strong-motion Italian records". In *Quali prospettive per l'Eurocodice 8 alla luce delle esperienze italiane*, Workshop Progetto Reluis, Napoli (in press).

Makdisi F.I., Seed H.B. (1978). "Simplified procedure for estimating dam and embankment earthquake-induced deformations". *J. Geotech. Eng.*, ASCE 104: 849–867.
Martino S., Scarascia Mugnozza G. (2005). "The role of seismic triggering in the Calitri landslide (Italy): historical reconstruction and dynamic analysis". *Soil Dynamics and Earthquake Engineering*, 25: 933-950.
Matsui T., Ohara H., Ito T. (1980). "Cyclic stress-strain history and shear characteristics of clay". *Journal of Geotechnical Engineering*, ASCE, 103(7): 757-768.
Newmark N.M. (1965). "Effect of earthquakes on dams and embankments". *The Rankine Lecture, Geotèchnique*, 15(2): 139-160.
PLAXIS 2D (2003). Reference Manual, Version 8.
Rampello S., Silvestri F., Viggiani G. (1994). "The dependence of G_0 on stress state and history in cohesive soils". Invited paper. In *Pre-Failure Deformation Characteristics of Geomaterials*, 1st IS-Hokkaido, Sapporo, Balkema, 2: 1155-1160.
Rathje E.M., Abrahamson N.A., Bray J.D. (1998). "Simplified frequency content estimates of earthquake ground motions". *Journal of Geotech. and Geoenvironm. Engng.*, ASCE, 124(2): 150-159.
Sarma S.K., Chlimintzas G.O. (2001). "Co-seismic and post seismic displacement of slopes". XV *ICSMEG TC4, Satellite Conference on "Lesson learned from recent strong earthquakes"*, Istanbul, Turkey, 183-188.
Seed H.B. (1979). "Considerations in the earthquake-resistant design of earth and rockfill dams". *Geotéchnique*, 29: 215–263.
Seed H.B., Martin G.R. (1966). "The seismic coefficient in earth dam design". *J. Soil Mech. and Found. Div.*, ASCE, 92(3): 25–58.
Sica S., Pagano L., Modaressi A. (2008). "Influence of past loading history on the seismic response of earth dams". *Computers and Geotechnics*, 35: 61-85.
Stewart J.P., Blake T.F., Hollingsworth R.A. (2003). "A screen analysis procedure for seismic slope stability". *Earthquake Spectra*, 19(3): 697–712.
Vucetic M., Dobry R. (1991). "Effects of the soil plasticity on cyclic response". *Journal of Geotechnical Engineering*, ASCE, 117(1): 89-107.
Whitman R.V., Liao S. (1984). "Seismic design of gravity retaining walls. *Proc. 8th WCEE,* San Francisco, 3: 533-540.

8 RELUIS REFERENCES

Amorosi A., Boldini D., Elia G. (2008). "Modellazione della risposta sismica locale mediante analisi agli elementi finiti". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.
Amorosi A., Boldini D., Elia G. (2009). "Finite element modelling of seismic ground response". *Journal of Earthquake Engineering*, submitted for publication.
Amorosi A., Boldini D., Elia G., Lollino P., Sasso M. (2007). "Sull'analisi della risposta sismica locale mediante codici di calcolo numerici". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2007, Salerno.
Amorosi A., Boldini D., Germano V. (2007). "A generalised backward euler scheme for the integration of a mixed isotropic-kinematic hardening model for clays", *International Journal of Numerical and Analytical Methods in Geomechanics*, Published online in Wiley Inter-Science (www.interscience.wiley.com). DOI: 10.1002/nag.663.
Amorosi A., Elia G. (2008). "Analisi dinamica accoppiata della diga Marana Capacciotti", *Rivista Italiana di Geotecnica,* 52(4): 78-96.
Amorosi A., Elia G., Chan A.H.C., Kavvadas M.J. (2008). "Fully coupled dynamic analysis of a real earth dam overlaying a stiff natural clayey deposit using an advanced constitutive model". In *12th Int. Conf. of International Association for Computer Methods and Advances in Geomechanics (IACMAG)*, Goa, India.

Ausilio E., Costanzo A., Silvestri F., Tropeano G. (2008). "Prediction of seismic slope displacements by dynamic stick-slip analyses". In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, vol. 1: 475-484.

Ausilio E., Costanzo A., Silvestri F., Tropeano G. (2009). "Evaluation of seismic displacements of a natural slope by simplified methods and dynamic analyses". In *Performance-Based Design in Earthquake Geotechnical Engineering - from case history to practice*, IS -Tokyo, in press.

Ausilio E., Silvestri F., Troncone A., Tropeano G. (2007a). "Seismic displacement analysis of homogeneous slopes: a review of existing simplified methods with reference to italian seismicity". In *Earthquake Geotechnical Engineering*, *4th International Conference*, Thessaloniki, Greece, Paper No. 1614.

Ausilio E., Silvestri F., Tropeano G. (2007b). "Simplified relationships for estimating seismic slope stability". In *ETC12 Workshop on Geotechnical Aspects of EC8*, ISSMGE, Madrid, Spain.

Bandini V., Cascone E. (2008). "Un modello multiblocco per l'analisi sismica di pendii in terreni coesivi". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Bandini V., Cascone E., Biondi G. (2008). "A GLE multi-block model for the evaluation of seismic displacements of slopes". In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, vol. 1: 485-492.

Bilotta E., Pagano L., Sica S. (2008). "Sincronismo del moto sismico in due dighe in terra". *Rivista Italiana di Geotecnica*, 52(4): 182-206.

Bilotta E., Pagano L., Sica S. (2009). "Effects of asynchronism on seismic performance of earth dams". In *Performance-Based Design in Earthquake Geotechnical Engineering - from case history to practice*, IS -Tokyo, in press.

Bilotta E., Pagano L., Siervo L. (2007). "Determinazione del coefficiente sismico equivalente". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2007, Salerno.

Bilotta E., Sica S., Pagano L. (2008). "Asincronismo del moto sismico in dighe in terra: evidenze sperimentali ed analisi". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Biondi G., Cascone E., Maugeri M. (2006). "Equivalenza tra analisi pseudo-statica e metodo degli spostamenti". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2006, Pisa.

Biondi G., Cascone E., Maugeri M. (2008). "Evaluation of pseudo-static coefficients according to performance-based criteria of rigid retaining walls". In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, vol. 1: 501-508.

Biondi G., Cascone E., Rampello S. (2007). "Performance-based pseudo-static analysis of slopes". In *Earthquake Geotechnical Engineering*, *4th International Conference*, Thessaloniki, Greece, Paper No. 1645: 1-12.

Callisto L., Rampello S. e Fargnoli P. (2008). "Studio del comportamento sismico di una diga in terra con metodi disaccoppiati". *Rivista Italiana di Geotecnica*, 52 (4): 38-53.

Costanzo A., Ausilio E., Tropeano G., Silvestri F. (2008). "Valutazione degli spostamenti di pendii omogenei sotto azione sismica attraverso analisi pseudo-statiche e dinamiche con complessità crescente". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Elia G., Amorosi A., Chan A.H.C., Kavvadas M. (2009). "Fully coupled dynamic analysis of an earth dam". *Géotechnique*, submitted for publication.

Fargnoli P., Rampello S. e Callisto L. (2007). "Analisi di stabilità dei pendii: valutazione del coefficiente sismico equivalente". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2007, Salerno.

Fargnoli P., Rampello S. e Callisto L. (2008). "Analisi della risposta sismica di pendii in condizioni bidimensionali". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Lollino P., Santaloia F., Amorosi A., Cotecchia F. (2009). "The role of post-excavation pore pressure equalization in the delayed failure of natural stiff clay slopes: the case of the Lucera landslide". *Géotechnique*, submitted for publication.

Madiai C. (2008). "Correlazioni tra spostamento atteso del blocco di Newmark e parametri del moto sismico valide per il territorio italiano". *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Madiai C. (2009). “Correlazioni tra parametri del moto sismico e spostamenti attesi del blocco di Newmark”. *Rivista Italiana di Geotecnica* (in press).

Pagano L., Mancuso C., Sica S. (2008). “Prove in sito sulla diga di Camastra: tecniche sperimentali e risultati”. *Rivista Italiana di Geotecnica*, 52(3): 9-27.

Pagano L., Mancuso C., Sica S. (2009). “Characterization of soil properties of an earth dam by in situ tests”. In *Performance-Based Design in Earthquake Geotechnical Engineering - from case history to practice*, IS -Tokyo, in press.

Pagano L., Sica S., Coico P. (2009). “A Study to evaluate the seismic performance of road embankments. *Soils and Foundations* (accepted for publication).

Rampello S., Callisto L. (2008). “Stabilità dei Pendii in condizioni sismiche”. In *Opere Geotecniche in condizioni sismiche*. XII Ciclo di Conferenze di Meccanica e Ingegneria delle Rocce, 241-271.

Rampello S., Callisto L., Fargnoli P. (2008). “Evaluation of seismic coefficients for slope stability analysis using a displacement-based approach”. Panel paper. In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, in press.

Rampello S., Callisto L., Fargnoli P. (2009). “Two-dimensional seismic response analysis to evaluate permanent slope displacements”. In *Performance-Based Design in Earthquake Geotechnical Engineering - from case history to practice*, IS -Tokyo, in press.

Rampello S., Cascone E., Grosso N. (2009). “Evaluation of the seismic response of a homogeneous earth dam”. *Soil Dynamics and Earthquake Engineering*, 29: 782-798.

Scasserra G., Lanzo G., Stewart J.P., D’Elia B. (2008a). “SISMA (Site of Italian Strong Motion Accelerograms): a web-database of ground motion recordings for engineering applications”. In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, vol. 2: 1649-1656.

Scasserra G., Stewart J.P., Bazzuro P., Lanzo G., Mollaioli F. (2008b). “Comparison of empirical ground motion models for global shallow crustal earthquakes to italian strong motion data”. *Bulletin of the Seismological Society of America* (accepted for publication).

Scasserra G., Stewart J.P., Kayen R., Lanzo G. (2008c). “Site characterization of Italian strong motion recording stations”. In *Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA)*, vol. 1: 338-345.

Scasserra G., Stewart J.P., Lanzo G. and Kayen, R. (2008d). “Database for earthquake strong motion studies in Italy”. *Journal of Earthquake Engineering* (accepted for publication).

Sica S. & Pagano L. (2009). “Performance-Based Analysis of earth dams: procedures and application to a sample case”. *Soils and Foundations* (accepted for publication).

Sica S., Pagano L., Vinale F (2008). “Interpretazione dei segnali sismici registrati sulla diga di Camastra”. *Rivista Italiana di Geotecnica*, 52(4): 167-181.

Tropeano G., Ausilio E., Costanzo A., Silvestri F. (2008). “Previsioni di spostamenti sismici di pendii omogenei con diverse procedure semplificate”. *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2008, Catania.

Tropeano G., Ausilio E., Costanzo A., Silvestri F. (2009). “Valutazione della stabilità sismica di pendii naturali mediante un approccio semplificato agli spostamenti”. In *L'Ingegneria Sismica in Italia*, XIII Convegno Nazionale, Bologna (in press).

Tropeano G., Ausilio E., Silvestri F., Troncone A. (2007). “Analisi sismica di pendii omogenei: revisione di un metodo semplificato esistente con riferimento alla sismicità italiana”. *Incontro Annuale dei Ricercatori di Geotecnica*, IARG 2007, Salerno.

SUB-THEME 6.4: DEEP FOUNDATIONS

1 INTRODUCTION

This research deals with the behaviour of deep foundations under earthquake excitations, with particular reference to the kinematic interaction phenomenon between the soil and the pile invested by the seismic waves.
During strong earthquakes foundation piles tend to significantly modify soil deformations, since they oppose to the seismic motion of the subsoil. Further because of the interplay between soil and piles the motion at the base of the superstructure can significantly deviate from the free-field motion, and the piles be are subjected to additional bending, axial and shearing stresses. The bending moments, usually referred to as “kinematic” ones, may be very important even in the absence of the superstructure.
In the past the kinematic interaction between soil and piles has been studied by many researchers. In spite of the big effort on such a topic, the complex problem still need to be investigated and well consolidated in the research field. As a matter of fact kinematic interaction has been rarely accounted for in practical design. Modern seismic codes have, however, acknowledged the importance of kinematic interaction and demand piles to be designed also accounting for soil deformations arising from the passage of seismic waves. Eurocode 8 (EN 1998-5), for example, suggests that kinematic effects should be taken into account when all the following conditions simultaneously exist: 1) seismicity of the area is moderate or high (specifying that moderate or high-seismicity areas are characterized by a peak ground acceleration $a_g \cdot S > 0.1g$, where a_g is the design ground acceleration on type A subsoil and S is the soil factor); 2) subsoil type is D or worse, characterized by sharply different shear moduli between consecutive layers; 3) the importance of the superstructure is of III or IV class (e.g. schools, hospitals, fire stations, power plants, etc). The recent Italian building code (D.M. 14.1.2008) provides quite similar indications concerning the kinematic bending moments in piles.
In this context the present ReLUIS research line 6.4 has been planned, involving five Italian research groups and collaborations with other national and international experts and foreign seismic research structures. The main final goal of research Line 6.4 was to find out innovative elements for a new regulation on seismic design of piled foundations with emphasis on kinematic interaction. The evidence collected from previous studies and the results of the present research, even far from providing a definitive answer to the question, have allowed the clear individuation of the factors controlling the phenomenon at hand, the evaluation of pile response for reference soil-pile configurations, some important indications for pile seismic design, and mainly what has still to be done for this issue.

2 BACKGROUND AND MOTIVATION

2.1 Background on analytical aspects

The kinematic interaction phenomenon has been studied by many researchers by means of different approaches and analytical tools. The methods for the analysis of kinematic soil-pile interaction may be classified into three groups: <u>continuum approaches</u> (FEM, BEM), <u>Winkler methods</u> (BDWF), <u>simplified formulations</u>.

In continuum approaches soil, pile and superstructure are modelled as a whole. The soil-pile geometry typically is modelled in 3-D and discretized by F.E.M. or B.E.M. techniques (Kaynia 1996, Kimura e Zhang, 2000; Zhang et al. 2000; Zhang e Kimura 2002; Wu e Finn, 1997; Bentley & El Naggar, 2000; Finn e Fujita, 2002). Soil is usually modelled by elastoplastic models. A few works in literature adopt models of the so-called Advanced Plasticity to properly reproduce soil response under cyclic loading conditions, and some work describes the possibility to simulate gap at the pile-soil interface under strong motion events (Maheshwari et al., 2004). Among numerical approaches, the finite element method (Wu and Finn, 1997; Cai et al., 2000; Kimura and Zhang, 2000; Maheshwari et al., 2004) provides a powerful and versatile technique, since some important effects such as soil nonlinearity and heterogeneity may be directly accounted for. Nevertheless, this method is generally very expensive from a computational viewpoint, since it requires suitable boundaries conditions being introduced to simulate the radiation damping effect. Worth mentioning is the simplified continuum approach developed by Wu & Finn (1997) in which vertically propagating shear waves are modelled disregarding seismic induced deformations in the vertical direction and along the normal to the direction of shaking (quasi 3-D approach).
A more attractive approach is represented by the boundary element technique (Kaynia and Kausel, 1982; Mamoon and Banerjee, 1990; Cairo and Dente, 2007). It only needs the discretization of the interfaces and permits the condition of wave propagation towards infinity to be automatically satisfied. The ReLUIS research group from the University of Calabria developed a code called SASP (Seismic Analysis of Single Piles) based on BEM solution for the analysis of single piles and pile groups subjected to vertical loading, under static and dynamic conditions. The method makes use of the closed-form stiffness matrices derived by Kausel & Roësset (1981) to simulate the response of layered soils. The analysis is performed in the frequency domain under the assumption of soil linearity.
The methods based on the Winkler foundation model assume that the pile is modelled as a linearly elastic beam, with length L and diameter d, discretized into segments connected to the surrounding soil by springs and dashpots, which provide the interaction forces in the lateral direction (Figure 14). Springs represent soil stiffness and dashpots soil damping due to radiation and hysteretic energy dissipation. As a first approximation, the spring stiffness k may be considered to be frequency-independent and expressed as a multiple of the local soil Young's modulus Es (Kavvadas and Gazetas, 1993). The dashpot coefficient c represents both material and radiation damping. The latter one may be computed using the analogy with one-dimensional wave propagation in an elastic prismatic rod of semi-infinite extent (Gazetas and Dobry, 1984).

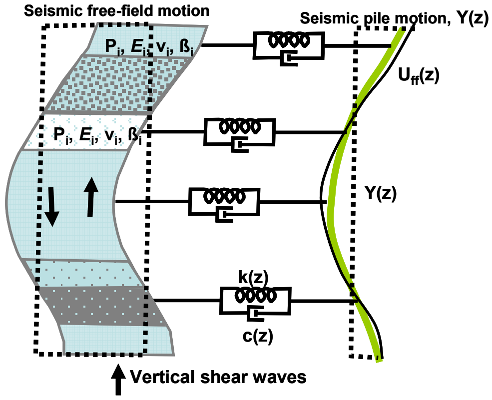


Figure 14. Beam on Dynamic Winkler Foundation model (BDWF).

Some results obtained using the Winkler approach (BDWF), as proposed by Mylonakis et al. (1997), proved to be quite accurate, computationally time saving and suitable for performing extensive parametric analyses (to investigate the role of both the variables of the system and the earthquake motion characteristics). They further allow nonlinear behaviour of the soil to be easily incorporated if solution is envisaged in the time domain (Boulanger et al., 1999; El Naggar et al., 2005; Maheshwari and Watanabe, 2006; Cairo et al., 2008).
Simplified formulations are closed-form expressions (Dobry and O'Rourke, 1983; Nikolaou and Gazetas, 1997; Mylonakis, 2001; Nikolaou et al., 2001) available in literature for approximately computing the maximum steady-state bending moment at the interface between two layers. These approaches have been derived by modelling the pile as a beam on a Winkler foundation and are based on the following simplified assumptions: each soil layer is homogeneous, isotropic and linearly elastic; the soil is subjected to a uniform static shear stress field; the pile behaves as a linear-elastic semi-infinite beam; the embedded length of the pile in each layer is greater than the so-called "active length".

2.2 Background on experimental activity

Experimental activities on pile models have been conducted in the past, mainly by shaking table apparatus (1*g*), with the aim to investigate seismic behaviour of soil-pile systems. Most of the experimental tests focused on liquefaction problems (Towhata et al., 1999; Dungca et al, 2006; Yao et al, 2004). Significant tests have also been carried out for understanding soil-pile-superstructure interaction (SSPSI) in cohesive soils and in non-liquefiable sand.
In particular Meymand (1998) carried out a series of 1*g* tests on model piles in soft clay, with the main goals of providing insight into a variety of SSPSI topics, and generating a data set with which to calibrate an advanced SSPSI analysis tool being developed at U.C. Berkeley. The tests on single piles were seen to respond with components of inertial and kinematic interaction, though the inertial components produced upper bound bending moments. These results suggest that developing pile demands from consideration of inertial loading only may be acceptable for cases where site stiffness contrasts or ground failure (lateral spreading) do not exert significant soil loads or deformations on the piles.
Recently Tokimatsu and Suzuki (2009) performed a huge experimental study utilising the big shaking table at E-Defense, Japan. The tests were conducted on soil-pile-structure models with dry and saturated sands for examining and quantifying the effects of inertial and kinematic forces. These main findings can be outlined: *i)* if the natural period of the structure is less than that of the ground, the kinematic force tends to be in phase with the inertial force, increasing the stress in piles; *ii)* if the natural period of the structure is greater than that of the ground, the kinematic and inertial forces tend to be out of phase, restraining the pile stress from increasing; *iii)* the maximum pile stress may be estimated by applying both the inertial and kinematic forces on the pile at the same time, if the natural period of the structure is less than that of the ground; it may be estimated as the square root of the sum of the squares of the two moments estimated by applying the inertial and kinematic forces on the pile separately, if the natural period of the structure is greater than that of the ground.

3 RESEARCH STRUCTURE

The general objective of the project was to reach a level of knowledge suitable for identifying elements and suggestions to be introduced in the national normative on the seismic behaviour of deep foundations with special regard to kinematical interaction. At this aim the following partial objectives were envisaged:

(a) elaboration of the state of art on the kinematical interaction;
(b) evaluation of the acceptability of the hypothesis of foundation fixed to the base;
(c) selection of the criteria for identifying the cases in which it is necessary to take into account the kinematical interaction for the calculation of the stresses induced on piles;
(d) definition of computational procedures of the stresses due to the kinematical interaction, characterized by different degrees of complexity;
(e) evaluation of the feasibility of an experimental testing activity on physical models, with the aim of validating the results acquired through the numerical research.

The following five research Units (RU) were involved: University of Sannio (alias SANNIO), University of Basilicata (alias UNIBAS); University of Calabria (alias UNICAL); University of Catania (alias UNICT) and Second University of Napoli (alias SUN). Further, other RU from the University Parthenope (I), the University of Bristol (UK) and the University of Patras (G) effectively collaborated to the research.

The above objectives have been pursued through the following research activities:

(a) the layout of state of the art report;
(b) the identification of different computational procedures, with different degree of complexity of both the physical-mathematical model and the computational algorithm;
(c) the identification of a wide range of sample cases, characterised by different characteristic parameters of the exciting waveform, stratigraphical features and geometry of the foundation system (from the single pile to the group of n-piles, with a wide variability of the main geometrical features);
(d) the execution of the numerical analyses of the sample cases, by means of the different computational procedures previously selected;
(e) the interpretation and synthesis of the results, with the identification of typical behaviours, for the discrete range of values of the significant model parameters;
(f) a first experimentation on a physical model, by means of a shaking table apparatus.
(g) the identification of elements to be introduced in the seismic normative.

The following three documents have been produced, for illustrating the whole research activities:

A. Report on the activity and results of each Research Unit;
B. Publication of all the studied cases, which have been effective for the identification of typical behaviours;
C. Elements for a technical normative on the project of deep foundations in seismic areas.

4 MAIN RESULTS

During the first year the research Units involved in Line 6.4 have carried out a wide study on the knowledge available on this specific topic. Literature on kinematic interaction was found to be often fragmentary and sometimes conflicting and incoherent. The second year of this project has been mainly dedicated to the development of "reference" numerical models for the dynamic analysis of kinematic interaction effects for piled foundations. Analyses have been carried out by models with increasing degree of complexity: P-y curves, BDWF (*beam on dynamic Winkler foundation*) models (both linear and non-linear), 3D models based on the finite element method and the boundary element method. Non-linear soil response has been simulated at a very simple level by imposing G=G(γ) and D=D((γ) material curves (G = shear modulus, D = soil damping ratio, γ= shear strain) or at a more sophisticated level by means of models capable to consider hysteretic soil behaviour.

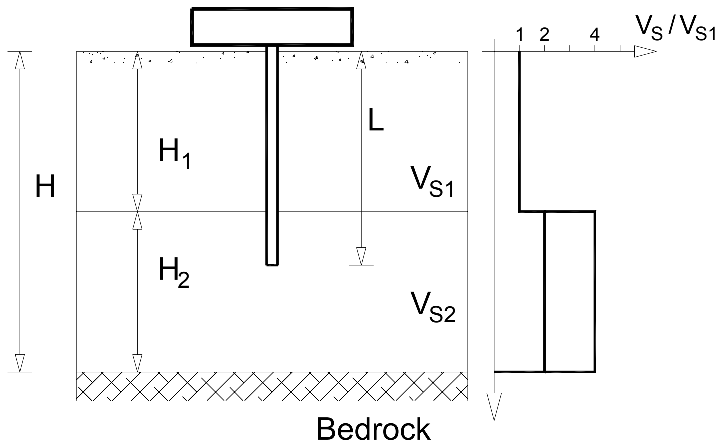


Figure 15. Foundation scheme adopted as basis by the five research Units.

Several particular soil-pile configurations have been studied by all the RU ('general cases'). In addition each RU has independently developed the study of particular cases. Most of the research has been dedicated to the response of piled foundations (single piles as well as pile groups) embedded in an ideal subsoil consisting of two layers separated by an interface at depth H_1 and underlain by a rigid base at depth H (see Figure 15). While UNISANNIO, for example, has performed a parametric study to assess the influence of the depth H of the bedrock, SUN has investigated the role of the interface depth H_1 on the kinematic response of both single piles and pile groups. At the same time the effect of non linear soil response has been investigated by UNICT , by means of a pseudostatic approach based on the P-y curve method, and by UNISANNIO and UNICAL by performing linear equivalent analyses. In addition to this a computer code capable to account for the non-linear soil response has been developed by UNICAL (Cairo et al. 2008). This is based on a dynamic Winkler model (BNWF = beam on a Non linear Winkler Foundation) and represent an enhancement of the original version developed by Conte and Dente in 1988. One out of the key features of this code is the use of P-y curves based on the classical relationship by Ramberg-Osgood.
Summarising, elastic analyses have been performed for single piles and for pile groups; the bending moments have been investigated both at the interface between soil layers and at the pile head; the results by the different analysis tools have been compared, both considering the linear and non linear soil behaviour. The detailed illustration of the results is published in the Final Scientific Reports supplied by the Research Units; here only a list of the numerous case studied by means of elastic analysis is given in Table 1.
In the third year an experimental research has been carried out by means of the shaking table at BLADE (Bristol Laboratory for Advanced Dynamics Engineering) of the University of Bristol, thanks to the support of the Universities of Sannio and Calabria (Italy), and the collaboration with the same University of Bristol (UK) and the University of Patras (Greece). Laboratory tests were carried out on a pile model, for three configurations of subsoil (made of different granular materials and even rubber) pluviated inside a special container (*shear stack*, Figure 16). The shear stack was installed on the earthquake simulator, and subjected to three different Italian earthquake. Five different pile boundary conditions were tested (pile head free and fixed against rotation, with or without a superstructure). The repeatability of the test was checked, and the effects of input motion frequency content, soil stiffness contrast, and interaction between inertial and kinematic effects were successfully investigated.

Table 1. Linear elastic analyses.

RU	Method of analysis - computer code	H (m)	H1 (m)	L (m)	V_{S1} (m/s)	V_{S2} / V_{S1}	D(%)	No. of seismic input motions*	Foun-dation type
SANNIO	BDWF-SPIAB	30-(21)[1]	5-10-15-19	20	100	2-3-4	2 10	17	Single pile
		30-(21)[2]	5-15	20	150	2-2.67-4	10	17	Single pile
	3D Continuum dyn. analysis VERSAT P3D	30	5-10-15-19	20	100	4	10	3	Single pile
	3D Continuum dyn. analysis ABAQUS	30	15	20	100	4	2	19	Single pile
UNICT	Pseudostatic P-y transfer curve method	30	15	30	$E_P/E_1$5 000	0.58-3		3	Single pile Pile in a group
	3D Continuum dyn. analysis SAP	30 24	15 8	20 12	100 80	2-3-4 2-4	2-10 10	1 3	Single pile
UNIBAS	BDWF	30	5-10-15-17-19	20	50-100	2-4	10	1	Single pile
		30	15	20	100	2	10	16	Single pile
SUN	3D Continuum dyn. anal. VERSAT P3D	30	5-10-12-15-17-19 10	20 5-9-10-20	50-100 100	2-4 4	10	3	Single pile and pile groups
		30	5	6-10-20	100	4	10	4	Single pile
UNICAL	3D Continuum dyn. anal. SASP	30	15	20	100	2-3-4	10	3	Single pile
	BDWF	30	5-10-15-19	20	100 (150)[3]	2-3-4	10	17	Single pile

[1]For H=21 m the analyses have been performed only for H_1=15 m and D=10%
[2]For H=21 m the analyses have been performed only for H_1=15 m
[3]For V_{S1}=150m/s, V_{S2}/V_{S1} ratios were 2-2.67 and 4
* For the list of the utilised accelerograms, see the document B produced by the L.R. 6.4

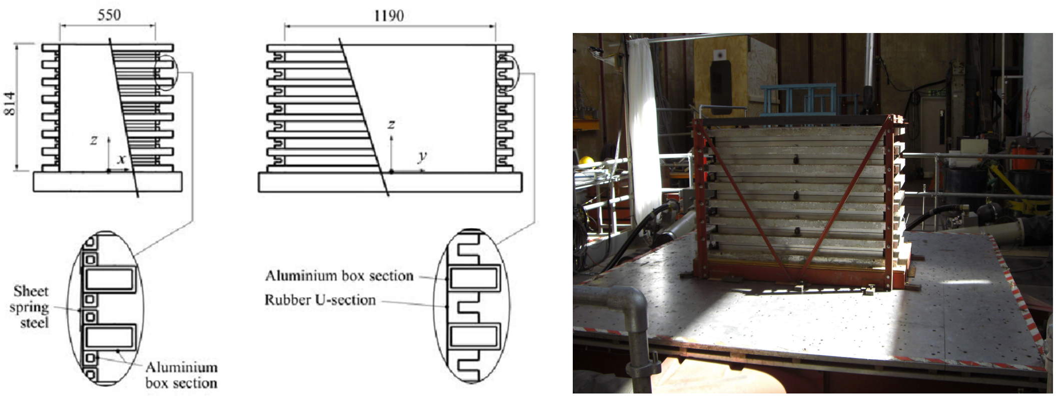


Figure 16. Shear Stack at BLADE (Bristol, UK).

5 DISCUSSION

The main objective of the research project was the evaluation of the kinematic interaction effects on pile foundations, with the final aim to find out suggestions for recent Codes. In order to achieve the goals, analytical approaches of different complexity levels have been applied. The wide set of Italian strong motion accelerograms analysed and collected in the database SISMA have been utilised as input motion.

First extensive analyses have been performed by the BDWF (Beam on Dynamic Winkler Foundations) methods, which are the most common tools utilised in the research field too, for investigating the linear response of the soil-pile systems. The same approach has been utilised for investigating the effect of soil non linearity on kinematic interaction effects. An important upgrading has been achieved in the application of BDWF methods, defining the proper range of soil parameter (stiffness and damping) values for analysing the linear response at low strain levels, or the non linear (or the simpler equivalent linear) response of soils at higher strain levels. The results obtained by different BDWF tools utilised by the involved research Units are in good agreement, and appear to provide good indications for a practical, even if conservative, pile design.

The extensive analyses confirmed the significant role of the following variables on the amount of the induced pile bending moments:

- stiffness contrast between two consecutive soil layers;
- depth of the interface between layers with stiffness contrast, with respect to the active length of the pile;
- boundary conditions at the head of the piles;
- whole subsoil stiffness (subsoil classification according to recent seismic Codes);

Their role has been also enhanced by the results of the experimental tests performed by the shaking table apparatus at BLADE of the University of Bristol.

The main conclusion that can be drawn by these applications is that kinematic interaction effects can be significant in particular conditions, but not dramatic as it could appear on the basis of the oversimplified formulas proposed in the past to evaluate the maximum bending moments. In fact, the amount of the effects does not depend only on the soil-pile configuration (i.e. the variables listed before); it has been demonstrated that the effects of the interaction strongly depend on the coupling between the input motion frequency content and the subsoil main frequency; hence this new variable has to be accounted for in analysing the induced pile bending moments. From the previous considerations, it can be derived that the kinematic effects can not be properly determined on the basis of the pile-soil system and the maximum expected input motion acceleration, unless it is accepted that conservative criteria would be adopted in the design. As regards soil non linearity effect, it appeared to be both beneficial or detrimental for pile bending response: the influence of non linearity on the variation of the relative stiffness between consecutive soil layers plays a fundamental role. Observations and suggestions on these aspects have been summarised in “Product C” document produced by ReLUIS L.R. 6.4 at the end of the Project; an interesting proposal for Code design procedure has been outlined in Cairo et al. (2009).

Important indications relative to the asynchronous effects induced at the pile head by kinematic interaction and superstructure inertial actions have been provided by both numerical analyses and the laboratory tests performed at the BLADE on a single pile model with a simple oscillator resting on it.

Several applications of more advanced numerical approaches (based on FEM and BEM techniques) confirm the results obtained by BDWF methods, if the simple assumptions there adopted are still saved (linear or non linear behaviour of the soil, linear behaviour of the pile).

In conclusion, it may be stated that most of the research objectives have been attained, and that the performed studies also represent a valid reference for addressing the research in the next future.

6 VISIONS AND DEVELOPMENTS

The research activity that has been performed to date has been mainly addressed to the study of the seismic behaviour of piles, according to the most consolidated tools for studying soil-structure interaction under earthquake excitations. Then the obtained results have been substantially validated by several more advanced numerical analyses and by some experimental tests on model piles. The research has certainly provided good results, since they allow to state some important conclusions relative to the main factors influencing pile bending response, in the hypothesis usually assumed for the soil-pile system.
First attempts have been made to investigate more complex soil-pile configurations, including soil plasticity and pile yielding. In the next future, the role of soil plasticity, and its beneficial or detrimental effects on pile bending moments have to be accurately studied, through more refined numerical analyses implementing advanced soil constitutive models. At the same time it would be interesting to investigate the response of the soil-pile system after the yielding of the structural element, and its effects on the superstructure behaviour, in order to consider the eventual removal of the pile elastic response required in most recent seismic Codes.
The behaviour at the pile head also need to be more deeply investigated, in order to confirm or not the delay between the maximum kinematic and inertial induced effects; this aspect has to be confirmed also through further specifically planned experimental tests. The results on this question could have relevant effects on pile design, since actually all the Code impose to simply sum the maximum effects, probably producing over-conservative design.
In conclusion, the three-year research activity has provided significant results, with effective suggestions for the Codes, and encourages further and more addressed activities both in the theoretical and in the experimental research field.

7 MAIN REFERENCES

Bentley, K.J., and El Naggar, M.H. 2000. Numerical analysis of kinematic response of single piles. *Canadian Geotechnical Journal,* 37: 1368–1382.

Cai, Y.X., Gould, P.L., and Desai, C.S. (2000). Nonlinear analysis of 3D seismic interaction of soil–pile–structure system and application.Engineering Structures, 22(2): 191–199.

Cairo, R., Dente, G. (2007b). Kinematic interaction analysis of piles in layered soils. XIV European Conference on Soil Mechanics and Geotechnical Engineering, ISSMGE-ERTC 12 Workshop "Geotechnical Aspects of EC8", Madrid, Pàtron Editore, Bologna, cd-rom, paper n. 13

Conte, E., Dente, G. (1988). Effetti dissipativi nella risposta sismica del palo singolo. Deformazioni dei terreni ed interazione terreno-struttura in condizioni di esercizio, Monselice, 2, 19-38.

D.M. 14/01/2008 del Ministero delle Infrastrutture. Norme Tecniche per le Costruzioni. Gazzetta Ufficiale n. 29 del 4/02/2008

Dobry R. and O'Rourke M.J., 1983, Discussion on 'Seismic response of end-bearing piles' by Flores-Berrones R.,Whit-man R.V.', J. Geotech. Engng. Div., ASCE, 109, 778-781.

Dungca J.R., Kuwano J., Takahashi A., Saruwatari T., Izawa J., Suzuki H., Tokimatsu K., 2006 Shaking table tests on the lateral response of a pile buried in liquefied sand. Soil Dynamics and Earthquake Engineering 26 (2006) 287–295

EC8-5. 2003. Foundations, retaining structures and geotechnical aspects. CEN European Committee for Standardization, Bruxelles, Belgium, December 2003, Final Draft.

Finn W.D.L., Fujita N. "Piles in liquefiable soils: seismic analysis and design issues", Soil Dynamics and Earthquake Engineering, n.22, 731-742, 2002

Gazetas G., Dobry R. (1984). Horizontal response of piles in layered soils. Journal of Geotechnical Engineering, ASCE, Vol.110, no.1, pp 20-40.

Kanya, A. 1997. Earthquake-induced forces in piles in layered media. Geotechnical Special Publication 70, ASCE 75-95.

Kausel, E., Roësset, J.M. (1981). Stiffness matrices for layered soils. Bull. Seism. Soc. Amer., 71, 6, 1743-1761.

Kavvadas M, and Gazetas G. 1993. Kinematic seismic response and bending of free-head piles in layered soil. Geotechnique, Vol.43, no.2, 1993, pp 207-222.

Kaynia, A. M. & Kausel, E. (1982). Dynamic behaviour of pile groups. Proc. 2nd Int. Co& Numer. Meth. In Offshore Piling, Austin, 509-532.

Kimura M. & Zhang F. 2000 "Seismic evaluation of pile foundations with three different methods based on three-dimensional elasto-plastic finite element analysis", Soils and Foundations, 40, 5, 113-132

Maheshwari B.K., Truma K.Z., El Naggar M.H. Gould P.L. 2004 "Three-dimensional finite element non linear dynamic analysis of pile groups for lateral transient and seismic excitations", Canadian Geotechnical Journal , 41, 118-133

Mamoon, S.M., Banerjee, P.K., 1990. Response of piles and pile groups to travelling SH waves. Earthq. Engng. Struct. Dyn., 19 (4), 597-610.

Mylonakis G., Nikolaou A., Gazetas G. (1997). Soil-pile-bridge seismic interaction: kinematic and inertial effects. Part I: soft soil. Earthquake Engineering Structural Dynamics, 27, (3), 337-359.

Mylonakis, G. (2001). Simplified model for seismic pile bending at soil layer interfaces. Soils and Foundations, vol. 41, n. 4, 47-58.

Nikolaou S., Mylonakis G., Gazetas G., Tazoh T. 2001. Kinematic pile bending during earthquakes: analysis and field measurements. Géotechnique, 51 (5), 425-440

Nikolaou, A.S., and Gazetas, G. 1997. Seismic design procedure for kinematically loaded piles. In Proceedings of the 14th International Conference Soil Mechanics and Foundation Engineering. Hamburg, Special volume, ISSMFE TC4 Earthquake geotechnical engineering, pp.253-260.

Tokimatsu K, Suzuki H. 2009, Seismic soil-pile-structure interaction based on large shaking table tests. in Int. Conf. on Performance-Based Design in Earthquake Geotechnical Engineering, IS-Tokyo

Towhata I, Vargas-Monge W., Orense R.P., Yao M. Shaking table tests on subgrade reaction of pipe embedded in sandy liquefied subsoil. Soil Dynamics and Earthquake Engineering 18 (1999) 347–361

Wu, G., and Finn, W.D.L. 1997a. Dynamic elastic analysis of pile foundations using finite element method in the frequency domain. Canadian Geotechnical Journal, 34:34-43.
Wu, G., and Finn, W.D.L. 1997b. Dynamic nonlinear analysis of pile foundations using finite element method in the time domain. Canadian Geotechnical Journal, 34:44-52.
Yao S., Kobayashi K., Yoshida N., Matsuo H., Interactive behavior of soil–pile superstructure system in transient state to liquefaction by means of large shake table tests. Soil Dynamics and Earthquake Engineering 24 (2004) 397–409
Zhang F. & Kimura M. 2002 "Numerical prediction of the dynamic behaviour of a RC group-pile foundation", Soils and Foundations, 42, 3, 77-92
Zhang F., Kimura M., Nakai T., Hoshikawa T. 2000 "Mechanical behaviour of pile foundations subjected to cyclic lateral loading up to the ultimate state", Soils and Foundations, 40, 5, 1-17

8 RELUIS REFERENCES

Ardita D., Armenia F., Motta E., Raciti E. 2008. Interazione cinematica di un palo singolo in terreno eterogeneo: risultati preliminari. IARG 2008, Università degli Studi di Catania.
Cairo R. 2007. L'interazione cinematica nelle fondazioni su pali. IARG 2007, Università degli Studi di Salerno.
Cairo R. 2008. Interazione cinematica non lineare palo-terreno. IARG 2008, Università degli Studi di Catania.
Cairo R., Conte E., Cosimo V.A., Dente G. 2009. Analisi dell'interazione cinematica palo-terreno. ANIDIS, Bologna, in press.
Cairo R., Conte E., Dente G. 2008. Nonlinear seismic response of single piles. MERCEA'08, Reggio Calabria, AIP, N.Y., 1, 602-609.
Cairo R., Conte E., Dente G., Sica S., Simonelli A.L., 2009. Soil-Pile Kinematic Interaction: New Perspectives for EC8 Improvement. Workshop RELUIS_UNI SC8: "Quali prospettive per l'Eurocodice 8 alla luce delle esperienze italiane"; Napoli, 3 aprile 2009, (in print)
Cairo R., Dente G. 2007a. Un metodo per l'analisi dell'interazione cinematica palo-terreno nei depositi orizzontalmente stratificati. ANIDIS, Pisa, memoria n. 378.
Cairo R., Dente G. 2007b. Kinematic interaction analysis of piles in layered soils. XIV European Conference on Soil Mechanics and Geotechnical Engineering, ISSMGE-ERTC 12 Workshop "Geotechnical Aspects of EC8", Madrid, Pàtron Editore, Bologna, cd-rom, paper n. 13.
Castelli F., Maugeri M. 2007. A pseudo-static analysis for the evaluation of the lateral behaviour of pile groups. Proc. 4th Int. Conf. Earthquake Geotechnical Engineering, Thessaloniki, Greece, June 25-28, 2007, Paper no.1399.
Castelli F., Maugeri M. 2007. Numerical analysis for the dynamic response of a single pile. Proc. XIV European Conf. on Soil Mechanics and Geotechnical Engineering, ERCT 12 Workshop, Madrid
Castelli F., Maugeri M., Mylonakis G. 2008. Numerical analysis of kinematic soil-pile interaction. Proc. MERCEA 2008, Reggio Calabria, vol. 1, 618-625.
de Sanctis L., Maiorano R.S.M. 2007. Effetti della costituzione del sottosuolo sui momenti dell'interazione cinematica di pali in gruppo e pali isolati. IARG, Salerno
de Sanctis L., Maiorano R.S.M., Mandolini A., Aversa S. 2008. Un criterio per la valutazione dei momenti di interazione cinematica nei pali di fondazione indotti da sollecitazioni sismiche. IARG, Catania
de Sanctis L., Maiorano R.S.M. 2009. Analisi degli effetti della componente cinematica dell'interazione dinamica palo-terreno in un sottosuolo ideale a due strati. ANIDIS, Bologna,
de Sanctis L., Maiorano R.S.M. 2009. Earhquake induced kinematic bending moments of fixed and free-head piles. Proc. of the Int. Conf. on Performance Based Desing in Earth. Geotech. Engineering. IS Tokyo 2009, Tsukuba, Japan
Di Laora R., de Sanctis L., Mandolini A. 2009. Modifica del segnale sismico alla base di una struttura prodotta dalla presenza dei pali. ANIDIS, Bologna

Di Laora R., Mandolini A. 2009. Some remarks on the kinematic vs. inertial interaction for piled foundations. Proc. of the Int. Conf. on Performance Based Desing in Earth. Geotech. Engineering. IS Tokyo 2009, Tsukuba, Japan

Dihoru L., Bhattacharya S., Taylor C.A., Muir Wood D., Moccia F., Simonelli A.L., Mylonakis G. 2009. Experimental modelling of kinematic bending moments of piles in layered soils. Int. Conf. Performance Based Design in Earthquake Geotechnical Engineering, IS-Tokyo 2009

Grassi F., Massimino M.R. 2008. Primi risultati su uno studio di interazione cinematica palo-terreno mediante approccio FEM. IARG, Catania.

Grassi F., Massimino M.R. 2009. Evaluation of kinematic bending moments in a pile foundation using the finite element approach. 7th Int. Conf. Earthquake Resistant Engineering Structures, Cyprus.

Maiorano, R.M.S, de Sanctis L., Aversa S., Mandolini A. 2009. Kinematic response analysis of piled foundations under seismic excitation. Canadian Geotechnical Journal, 46(5): 571-584

Maugeri M., Castelli F. 2007. Seismic retrofitting of the piled foundation of a reinforced concrete building. Proc. 2nd Greece - Japan Workshop on Seismic Design, Observation and Retrofit of Foundations, Shin-juku-ku, Tokyo, pp.320-334.

Maugeri M., Castelli F., Massimino M.R. 2007. The contribution of geotechnical engineering to safer design of earthquake resistant building foundation. Proc. Special Workshop on "How can earthquake geotechnical engineering contribute to safer design of structures to resist earthquake?". 4th Int. Conf. Earthquake Geotechnical Engineering, Thessaloniki, Greece, pp.511-540.

Maugeri M., Motta E., Raciti E., Ardita D., Armenia F. 2009. Kinematic interaction of a single pile in heterogeneous soil. 7th Int. Conf. Earthquake Resistant Engineering Structures, Cyprus.

Maugeri M., Motta E., Raciti E., Ardita D., Armenia F. 2009. Seismic response of single piles due to kinematic interaction. TC4 Earthquake Geotechnical Engineering Satellite Conference, 2-3 October 2009, Egypt.

Moccia F., Sica S., Simonelli A.L. 2009. Kinematic interaction: a parametric study on single piles, Int. Conf. on Performance-Based Design in Earthquake Geotechnical Engineering, IS-Tokyo 2009

Moccia F., Simonelli A.L. 2008. Interazione cinematica palo-terreno: sperimentazione su tavola vibrante. IARG 2008, Università degli Studi di Catania.

Sica S., Mylonakis G., Simonelli A.L. 2007. Kinematic bending of piles: analysis vs. code provisions. 4th Int. Conf. on Earthquake Engineering, Thessaloniki

Sica S., Mylonakis G., Simonelli A.L. 2009. Kinematic pile bending in layered soils: linear vs. equivalent-linear analysis. Int. Conf. on Performance-Based Design in Earthquake Geotechnical Engineering, IS-Tokyo 2009

Sica S., Simonelli A.L. 2007 2007. Interazione cinematica palo-terreno: ricerca e normative. IARG 2007, Università degli Studi di Salerno.

Sica S., Simonelli A.L. 2008. Analisi dell'interazione cinematica palo-terreno. IARG 2008, Università degli Studi di Catania.

Simonelli A.L., Sica S. 2007. Interazione cinematica palo-terreno: analisi ed indicazioni di normativa, ANIDIS, Pisa.

Simonelli A.L., Sica S. 2008. Fondazioni profonde otto azioni sismiche. In Opere geotecniche in condizioni sismiche. XII Cliclo di Conferenze in Meccanica ed Ingegneria delle Rocce. Casa Ed. Pàtron Bologna.

Simonelli A.L., Sica S. 2008. Soil-pile kinematic interaction. Panel paper. MERCEA'08, Reggio Calabria, Italy (in print).