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Project of Research N. 6

INNOVATIVE METHODS FOR THE DESIGN OF GEOTECHNICAL SYSTEMS

Coordinated by: Associazione Geotecnica Italiana

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Scientific Report – Activities of the 1st Year

The *Associazione Geotecnica Italiana* (AGI) is actively engaged in the promotion of various activities concerning the seismic geotechnical design. Among these, AGI recently assumed the coordination of the Project of Research n. 6, within the *Reluis* programme, dedicated to innovative methods for the design of geotechnical systems.
This Project of Research encompasses four specific themes, for which the need of further progresses in the geotechnical design is apparent. The themes are:

- 6.1 Deep excavations and tunnels in urban areas;
- 6.2 Tunnels and underground structures in rock;
- 6.3 Slope stability;
- 6.4 Deep foundations.

The main aspects of the aforementioned themes are summarised in the following.

6.1 DEEP EXCAVATIONS AND TUNNELS IN URBAN AREAS

(Research coordinated by S. Aversa)

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 earth mass 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 or 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 for slopes, 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, just like any other numerical analysis, reliable dynamic soil-structure interaction analyses require a number of simplifications and approximations of the problem under examination, 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 and 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 and hypoplasticty, 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, both simple and advanced numerical analyses are carried out finalised both to a better understanding of soil-structure behaviour under seismic loading and to serve as a reference to develop reliable simplified procedures for design to adopt in practice.

2. ACTIVITIES

In the first semester of activity the research was mainly devoted to a survey of the existing technical literature on the seismic behaviour of flexible retaining structures and tunnel linings. The results of this survey were summarised in the progress report compiled at the end of the first semester of activities.
In the second semester, the different research units carried out activities in the following areas:

1 Advanced numerical modelling:
Finite Element formulation of equilibrium and dynamic coupled flow for a two-phase medium. Evaluation of the potential of existing finite element codes for carrying out fully coupled dynamic analyses. Evaluation of changes to existing codes to satisfy the specific needs of the research project. Identification of constitutive models capable of reproducing the main aspects of soil behaviour under cyclic and dynamic actions. Implementation of the selected models in existing Finite Element codes. (Perugia).

2 Analysis of the response of tunnels to seismic loading:
Pseudo-static, simplified dynamic and dynamic analyses of a sample problem in the longitudinal and transverse planes, for different subsoil conditions and two input acceleration time histories. Development of different methods to evaluate actions on the tunnel lining using the pseudo-static approach. Use of free-field one dimensional seismic site response analyses to compute accelerations, shear stresses, and strains at the tunnel depth. Preliminary soil-structure interaction dynamic analyses under plane strain conditions using a commercial code (Plaxis v.8). (Calabria, Molise, Napoli Federico II)

3 Analysis of the response of flexible retaining structures to seismic loading:
Definition of criteria and procedures to select rationally an appropriate value of the seismic coefficient to be used in pseudo-static methods, by simplified dynamic methods (both displacement based methods and decoupled ground response analyses) (Roma La Sapienza, Roma Tor Vergata). Dynamic numerical analyses of simple schemes of flexible retaining structures with commercially available codes (Napoli Parthenope, Padova, Roma La Sapienza,)

4 Physical modelling:
Survey of existing experimental facilities at the Geotechnical Centrifuge Centre of Cambridge University Engineering Department. Preliminary experimental programme to be carried out in the geotechnical centrifuge (Roma Tor Vergata, Molise).

In the selection of input acceleration time histories to be used for numerical analyses at the different levels of complexity, extensive use was made of real ground motions selected from a database compiled by researchers from Roma La Sapienza, in the context of the activities carried out for research line 6.3 - Slope Stability (Lanzo, 2006).

3. RESULTS

3.1 Advanced numerical modelling

Fully coupled dynamic analyses can be approached with the classical theory of mixtures. The governing equations are obtained using: conservation laws (mass, linear momentum and angular momentum) for each component in the mixture, equations describing the interaction between the different components, and constitutive laws for each component. In principle, it is possible to solve the full set of governing equations by FE, but this is seldom done in practice due to the very high non-linearity of the equations and the large number of points required to describe adequately the velocity fields for soil skeleton and water. In many cases, however, the problem can be simplified distinguishing between Extremely Slow Processes (ESP), Low Speed Processes (LSP), Medium Speed Processes (MSP), and High Speed Processes (HSP) up to the limit of Undrained Processes (UP), thus neglecting some terms in the governing equations. The first two cases (ESP and LSP) are drained conditions and quasi-static coupled consolidation, not relevant for this project. MSP are characterised by relatively high frequencies of the applied loads, such that inertia forces cannot be neglected, but, in the governing equations, it is possible to neglect the relative acceleration of the fluid phase. At the upper limit of HSP, UP are characterised by the fact that there is no relative motion between soil skeleton and fluid phase. Conventionally, undrained processes are tackled in total stress with *ad-hoc* constitutive laws and in the assumption that volume strains are solely due to water compressibility. The applicability of the different approximate solutions in one dimensional conditions depends on a number of non-dimensional factors related to the frequency of the applied loads, and to the thickness, stiffness, mass density and permeability of the soil layer. With the typical frequency contents of seismic actions, for soils with permeability in the range $10^{-4}\div10^{-5}$ ms^{-1} or less, such as silts and clays, the UP approach is justified, while for soils with permeability ranging between $10^{-2}\div10^{-4}$ ms^{-1}, the MSP approximation is satisfactory.
Several existing finite element codes were considered for potential use both commercial and freeware or shareware codes, specifically developed for academic use, often with availability of the source code. Some of the codes were developed specifically for geotechnical problems while others are general purpose, but may be integrated by adding new models to the library of constitutive models. In this respect a distinction can be operated between completely closed codes, whose application field cannot be extended beyond the standard version, partially open codes, where it is possible only to extend the library of available constitutive models, and fully open codes, whose capabilities can be extended by adding new elements and constitutive models. It was decided to work on both commercial and open source codes for advanced applications with the twofold objective of: a) explore the possibilities of commercial codes, potentially adopted in industry and ordinary design by non academics and b) maintain the option of implementing the problem of fully coupled dynamic consolidation in both MSP and HSP formulations currently available only for the closed code DYNAflow (Princeton). The commercial codes that were selected are Abaqus and Plaxis, the open source code is FEAP.
Three families of constitutive models were considered for advanced numerical analyses, namely: Bounding Surface plasticity with Radial Mapping (BSRM, Dafalias 1986), Bounding Surface plasticity with multiple Kinematic Hardening surfaces (BSKH, Mroz, 1967), and HypoPlasticity (HP, Kolymbas, 1991). One particular constitutive model was chosen for evaluation from each family: the BSRM model by Tamagnini & D'Elia (1999), the BSKH model by Rouainia & Wood (2000), and the HP model by Masin (2005) including the intergranular strain concept by Niemunis & Herle (1997). Despite their different mathematical structure all three models are based on Critical State Soil Mechanics and can reproduce the main features of the mechanical behaviour of fine-grained soils such as the existence of as virgin compression line, and the existence of a critical state line, which is reached asymptotically for very large deviator strains. Many of the material constants appearing in the mathematical formulation of the models have the same physical meaning and take the same numerical values when the models are calibrated on the same set of experimental data; this guarantees a minimum level of consistency for comparison of results.
The identification of one computational strategy to be adapted to both plasticity based and hypoplastic models was the main problem of this phase of the research; the choice was made to work with explicit adaptive methods for their versatility and robustness both for plasticity and hypoplasticity. In particular, for the purposes of this project a Runge-Kutta-Fehlberg method of the third order with adaptive substepping was developed. In the first year of activity the RFK algorithm has been successfully used to implement the three models above in Abaqus. All three implementations have been diffusely tested in a numerical program including single element tests with imposed displacements at the boundary and mixed boundary conditions,

and preliminary analyses of boundary value problems (UP approach) for static and dynamic loading. The single element tests have proved the high accuracy and robustness of the algorithm under different stress paths and the accuracy of the numerical linearization procedure. They have also permitted to define the optimum range of variation of the perturbation parameters for the numerical linearization, yielding the typical quadratic convergence of Newton method. The preliminary boundary value problems have shown that it is possible to carry out non-linear dynamic analyses with about 10^4 degrees of freedom in a total time less than a few hours.

3.2 Analysis of the response of tunnels to seismic loading

The sample problem that was selected for the analyses was a circular tunnel with reinforced concrete lining, diameter of 6 m and thickness of 0.3 m, axis at a depth of 15 m in a 30 m thick layer of medium dense gravel, sand or soft clay, overlying a relatively stiff bedrock (V_r = 800 m/s, γ=22 kN/m^3, D_o =0.5%). The values of small strain soil parameters were chosen using literature empirical relationships, linking the initial stiffness and damping ratio to stress, voids ratio and intrinsic properties, such as particle size and plasticity. The variation of shear modulus and damping ratio with shear strain level to use in linear equivalent dynamic analyses, were those by Vucetic & Dobry (1991) for clay (I_P=30%), Seed & Idriss (1970) for sand, and Stokoe (2004) for gravel (D_{50} = 10mm).

Analysis of the transverse section. Four different methods were developed to evaluate the maximum shear stress in the pseudo-static approach, from the equilibrium of a deformable soil column between the surface and a given depth. Both linear and linear equivalent analyses were carried out; in linear analyses the initial shear modulus, G_0, is used, whereas in linear equivalent analyses the stiffness depended on strain level as specified above. In the first method it is assumed that the profile of acceleration with depth is harmonic (first mode of vibration of a homogeneous layer on a rigid bedrock), and the acceleration is equal to a_g at bottom of the layer and Sa_g at the surface. In the second method the acceleration is taken to vary linearly form surface (Sa_g) to bedrock (a_g), as for a layer vibrating in a range of frequencies much lower than its fundamental frequency. Both the third and fourth method express the dynamic equilibrium of a soil column in a manner that is similar to that adopted for the evaluation of liquefaction susceptibility, but introduce different methods to calculate a stress reduction parameter, r_d, which takes into account the soil column deformability, in one case an expression from the literature, in the other a one dimensional seismic site response. In simplified dynamic analyses, accelerations, shear stresses, and strains induced by the seismic waves at the tunnel depth were calculated through a free-field one-dimensional SSR analysis in the frequency domain, (code EERA). The input acceleration time histories were scaled to values of a_g equal to 0.05g, 01.5g, 0.25g and 0.35g, as from the seismic zonation specified by OPCM 3274 (2003).
The FE software Plaxis v8 was used to perform full soil-structure dynamic analyses under plane strain conditions. The code defines the damping tensor as a linear combination of the mass and stiffness tensors (Rayleigh formulation). The combination coefficients were calculated to obtain a constant damping ratio between the first natural frequency of the layer and the fundamental frequency of the seismic signal. The bedrock was taken to be a rigid boundary.
In most cases, pseudo-static methods provided higher average shear strains than simplified dynamic analyses, particularly for non-linear behaviour and for soft deposits. Pseudo-static analyses are more conservative as the peak acceleration and degree of soil non-linearity increase. The free-field dynamic linear analyses in the frequency domain (EERA) and in the time domain (Plaxis) compare favourably in terms of shear strain amplitudes at the tunnel depth under different acceleration time histories. The shear forces, bending moments and thrust in the lining obtained with pseudo-static (methods 3 and 4) and full dynamic linear analyses are in relatively good agreement; the simplified dynamic method provides very similar values of shear force and bending moment as the full dynamic analyses but underestimates the thrust.

Analysis of the longitudinal section. This was carried out by two simplified methods which model the structure as a one-dimensional linear elastic beam, namely a finite difference solution of the dynamic equilibrium by Kawashima (2000) and an approximate solution by Fu *et al*. (2004), which can take into account, in a simplified way, the angle of incidence of seismic waves and the non-synchronism of the motion. Moreover, both methods require an input free-field ground displacement at tunnel axis depth, which was computed using three pseudo-static procedures (PS) and one simplified dynamic method (SD). As before, the pseudo-static methods always predicted higher displacements than SSR. A comparison in terms of computed shear force and bending moment gave indication that in a case where the tunnel is relatively flexible compared to the soil, the two approaches yield essentially the same results at distance from the tunnel ends. However, both approaches need to be further tested with stiffer tunnel structures and against full dynamic interaction analyses.

3.3 Analysis of the response of flexible retaining structures to seismic loading

For retaining structures embedded in a deformable soil, there are reasons to question the assumption that the structure should be designed based on the Mononobe-Okabe theory, which was developed with reference to the limit equilibrium of a rigid soil wedge subjected to an acceleration, expressed through a seismic coefficient which is taken to be constant in space and time.
Steedman & Zeng (1990) extended Coulomb method to a deformable soil wedge (V_s) subjected to the propagation of a sinusoidal wave with an amplitude k_h g, SZ method. The active pressure is expressed using a seismic earth pressure coefficient, K_{aE}, that depends on k_{hmax}, φ', and the ratio of the wall length to the wavelength H/λ. This result can be expressed in terms of the ratio k_h/k_{hmax}, in which k_h is the seismic coefficient corresponding, in the classic MO method, to the earth pressure coefficient computed using SZ method. For a given value of mobilised friction between the soil and the wall, the ratio k_h/k_{hmax} was proven to depend solely on the ratio H/λ.
To apply the method in practice the wavelength associated to the seismic ground motion must be evaluated, for a case where V_s and f are not constant: the cyclic behaviour of the soil is non linear and the input ground motion contains a numebr of sinusoidal signals of varying frequency. It is therefore necessary to carry out parametric studies to define the most appropriate range of values of frequency and stiffness, to evaluate an equivalent range of values of wavelength, for different simple idealised cantilevered and single-propped walls in sand. One-dimensional ground response analyses are being performed, in which a number of Italian acceleration time histories are applied to the bedrock of a deformable layer of varying thickness, stiffness and damping parameters. The analyses are carried ou with equivalent linear model (EERA) and with a non linear hysteretic model (NEERA). The results of the ground response analyses are used to compute values of the seismic coefficients based on the equivalent wavelengths of the seismic motion: in this manner, account is taken of soil deformability and spatial variability of motion along the height of the wall.
Moreover, for a ductile system, temporal variability of motion produce a progressive accumulation of displacements, and the choice of the seismic coefficient can be related to the maximum displacements that the structure can sustain without significant loss of strength. It is therefore planned to apply a displacement method with a large number of input acceleration time histories to express the reduction of the seismic coefficient with the computed displacement. To this purpose, the acceleration time histories may be scaled to the maximum accelerations specified by the OPCM for soil type, and upper bound curves characterised by an exceedence probability of 10% be identified for each seismic zone.
In order to quantify the effects of different factors (e.g.: application of selected input ground motion to the model, correct representation of boundary conditions, constitutive modelling of the soil, non-linear mechanical behaviour of structural elements, presence of interfaces, numerical strategies and algorithms, etc.) in the routine dynamic numerical analyses of retaining structures, a round robin exercise was carried out involving all the research units. Two structural schemes were analysed of one cantilevered retaining wall and one embedded retaining wall with one level of props in dry sand of given mechanical and physical properties, in plane strain conditions.
The analyses were carried out using different codes, different constitutive models and, sometimes, different assumptions. In particular, Roma La Sapienza adopted a Finite Differences Method based code (FLAC 2D) that allowed implementing hysteretic damping, while other units adopted a Finite Element Method based code (Plaxis v.8) with a viscous damping tensor resulting from a linear combination of mass and stiffness tensors (Rayleigh formulation). In all cases the constitutive model for the soil was elasto-plastic; Roma and Parthenope used a Mohr-Coulomb yield criterion and non-associated flow (zero dilatancy), while Padova considered Drucker Prager yield criterion and associated flow. Some discrepancies also exist between the geometries, restraints, interfaces, and seismic input.
A full comparison of the results is currently under way, together with an effort to minimise the existing discrepancies between different analyses, however it is already possible to draw some preliminary conclusions, as follows:
- in all cases the seismic event produces an increase of bending moments and shear forces in the retaining structures both temporary, during the event, and in the long term;
- this increase is connected to a redistribution of the earth pressures on the structure;
- the displacements, both temporary and permanent, experienced by the retaining structures are significant;

- the effects of damping are very important;
- a fixed restraint at the top of a cantilevered wall is meaningless.

3.4 Physical modelling

The experimental programme will consist of 14 tests carried out in the Geotechnical centrifuge of the Cambridge University Engineering Department (CUED) in the period December 1, 2006 – December 31, 2007. These tests will provide the experimental basis to calibrate simplified methods and advanced numerical analyses of the response of flexible retaining structures and tunnels under seismic actions.
The experimental set up for centrifuge modelling of seismic actions currently available at CUED makes use of a Stored Angular Momentum (SAM) actuator. The SAM actuator stores energy in a pair of spinning flywheels driving a reciprocating rod. This energy can be released to a model by means of a fast-acting hydraulic clutch, thus imparting an approximately sinusoidal input motion to the model, with control over amplitude (to a maximum of $\pm$ 2.5 mm), frequency (to a maximum of 50Hz at model scale), horizontal acceleration (to a maximum of $\pm$ 25g at 50 Hz), and duration. Instead, it is not possible to apply to the base of the model real acceleration time histories. Four boxes are available for the experimental programme: one large Equivalent Shear Beam container (670 mm × 250 mm × 430 mm), one small Equivalent Shear Beam container (560 mm × 220 mm × 240 mm), one Laminar Box (500 mm × 250 mm × 300 mm), and one glass sided Plane-Strain box (500 mm × 245 mm × 360 mm). The Equivalent Shear Beam containers and laminar box have deformable boundaries. The plane-strain box is lined with Duxseal to provide an absorbing boundary. For the technical characteristics of Cambridge centrifuge, most dynamic model testing takes place at between 50g and 80g, giving a maximum prototype extent of 54 m × 20 m × 35 m.
The tests will be carried out on three types of structures: circular tunnel, cantiliver wall, pair of retaining walls propped against each other, in dry or saturated sand. The seismic input will consist of trains of sinusoidal waves of different frequency. The same model will be subjected to successive trains of waves of different frequency and increasing amplitude up to failure. The models will be instrumented with strain gauges, displacement transducers and miniaturised pore pressure transducers, depending on the needs of each experiment and availability of transducers at CUED. Tests on tunnels (4 tests) will examine the behaviour of the transverse section in plane strain conditions, with the characteristic dimensions of an underground railway tunnel. The diameter of the tunnel will be constant while the tests will examine different subsoil conditions (dry/saturated sand), different realtive positions of the bedrock (bottom of the container), of the tunnel axis and the ground surface, varying the cover and the thickness of the deformable layer. Tests on flexible retaining structures (10) will be conducted in plane strain conditions on the two structural types mentioned above, with different depths of excavation, subsoil conditions (dry/saturated sand), relative densities (dense/loose), and thickness of the deformable layer.

4. CONFORMITY TO THE PROGRAM

The research carried out in the first year of activity involves to a different extent all the themes included in the project.
The activities connected to advanced numerical modelling are progressing as from schedule.
The activities connected to different methods of analyses of increasing complexity (pseudo-static, simplified dynamic and full dynamic with standard numerical codes) to evaluate the seismic performance of flexible retaining structures and tunnels have been anticipated with respect to the original schedule.
Centrifuge testing has been planned but not yet carried out due to technical problems associated with the refurbishment of Cambridge University geotechnical centrifuge. Laboratory testing has consequently been delayed to the time when the material to be tested in the centrifuge will be chosen.

6.2 TUNNELS AND UNDERGROUND STRUCTURES IN ROCK

(Research coordinated by G. Barla)

1. INTRODUCTION

According to the 1st year programme of activities, the main topics of the work carried out so far are as follows:

- State of the art of underground structures subject to earthquake damage
- Definition of seismic input
- Design analyses of underground structures
- Monitoring of underground structures.

2. STATE OF THE ART OF UNDERGROUND STRUCTURES SUBJECT TO EARTHQUAKE DAMAGE

One of the main activities carried out is the collection of information about the damage conditions suffered by tunnels and underground cavities in rock at depth due to earthquakes. A summary of the work performed so far is reported in a paper by Corigliano et al. (2006), which was presented at the recent First European Conference on Earthquake Engineering and Seismology, Geneva, 3-8 September 2006 (see Enclosure 1). A collection of descriptive forms which are intended to report the different case histories investigated is being prepared (see Enclosure 2 for a typical case: Bolu Tunnel, Turkey).

Underground facilities are usually less vulnerable to earthquakes compared with above-ground structures, however several tunnels worldwide have been heavily damaged by ground shaking. Dowding and Rozen (1978) grouped damages due to earthquakes in underground structures into three main categories:

(1) damage from ground shaking;
(2) damage from fault dislocation;
(3) damage by earthquake-induced ground failures (e.g. liquefaction and landslides).

The main focus of this report is on damage caused in deep tunnels by ground shaking. Tunnels with large overburden stress are usually bored in rock and thus the effects of ground instabilities like liquefaction or landslides are immaterial. Even though fault dislocations are one of the most important causes of tunnel collapse, they are only indirectly associated with seismic loading and the caused damage is concentrated in a relatively small area that can be identified by geological and seismic tectonic studies.
The major factors influencing the seismic response of an underground structure include (Dowding and Rozen, 1978; St.John and Zahrah, 1987):

- shape, size and depth of the structure;
- mechanical properties of the surrounding soil or rock;
- mechanical properties of the structure;
- severity of ground shaking.

Usually, earthquake-related damage in underground structures is mainly concentrated in zones where there is a sharp variation of mechanical or geometrical properties in both the ground and/or in the structure. The level of shaking generally increases as the stiffness of the ground decreases. Tunnels at low depth in soft

soils are seismically more vulnerable than underground structures bored in rock and with large overburden stress. However, occasionally also deep tunnels have been damaged by earthquakes due to either the effects of strong earthquake intensity or the presence of weak zones within the rock mass. Usually tunnels that experience co-seismic damage due to the presence of weak zones have shown problems also during the construction phase. On the contrary, co-seismic damage due to strong ground shaking may be found in tunnels located close to an active fault.
Dowding and Rozen (1978) identified three levels of damage for underground excavations in rock due to ground shaking:

- "no damage", which implies that post earthquake investigations revealed the absence of new cracking or falling rock blocks;
- "minor damage", which implies the fall of rock blocks and formation of new cracks;
- "damage", which implies major fall of rock blocks, severe formation of cracking and closure of the opening.

These authors also proposed a correlation between tunnel damage and peak ground acceleration (PGA) calculated at the free surface immediately above the tunnel through an attenuation law. They suggest that "minor damage" is expected when the value of PGA ranges between 0.19 g and 0.50 g. The corresponding thresholds for peak particle velocity (PGV) range approximately between 20 cm/s and 90 cm/s. The reliability of this correlation depends upon two strictly related aspects of the problem. The first is that peak ground motion parameters are evaluated at the free surface because in-depth measurements of ground motion are usually unavailable. The second is that these parameters are often estimated using attenuation laws which carry a certain level of uncertainty.

The values of PGV suggested by Dowding and Rozen (1978) are typical for near-fault earthquakes and for such events the predictions of PGV made by attenuation relations are even more inaccurate. In relative terms one of the most dependable of such relations is that developed by Bray and Rodriguez-Marek (2004). This relation has been used to correlate the ground motion parameter PGV to the damage thresholds defined by Dowding and Rozen (1978) (see Enclosure 1). For lined tunnels earthquake-related damages may include spalling, cracking, crushing, falling of the concrete lining, bending of reinforcing bars, rising of the invert, rock falling and failure of the tunnel liner. For unlined tunnels in rock, damages may include rock fall, spalling, local opening of joints and obstructing of the opening.

3. DEFINITION OF SEISMIC INPUT

A careful review of the documents collected so far shows that most earthquake-related damaged tunnels worldwide were located in the vicinity of the causative fault. The characteristic of ground motion in the vicinity (i.e. <10-15 km) of a causative fault can be significantly different from that of the far-field. Near-field ground motion is strongly influenced by the rupture mechanism, the direction of rupture propagation relative to the site, and possible permanent ground displacements resulting from the fault slip (Stewart et al., 2001). These latter two factors are usually identified respectively as "rupture-directivity" and "fling step" effects.

Recent earthquakes such as the 1994 Northridge (California), the 1995 Kobe (Japan), the 1999 Kocaeli (Turkey), and the 1999 Chi-Chi (Taiwan) earthquakes, demonstrated the high damage potential of near-field ground motion. In light of this, the study of the dynamic behaviour of underground structures located in the vicinity of seismically active faults requires a careful selection of input time histories. The near-fault zone is usually assumed to be within an epicentral distance of about 20÷60 km from the rupture (Stewart et al., 2001), however only at distances smaller than 10÷15 km a major damage to underground facilities is expected to occur.
For a strike-slip focal mechanism the directivity pulse is oriented along the strike-normal direction whereas the static ground displacement is aligned along the strike-parallel component. On the contrary, for a dip-slip source mechanism the directivity pulse is oriented along a direction normal to the fault dip and has components in both the vertical and the horizontal strike normal directions. The static ground displacement is aligned in this case along the direction parallel to the fault dip and has components in both the vertical and the horizontal strike normal directions (Somerville, 2002).

3.1 Rupture directivity effects

The term “directivity” refers to the direction of rupture propagation as opposed to the direction of ground displacement. In the aftermath of an earthquake a site may be classified as affected by forward, backward or neutral directivity effects.
Forward-directivity effects occur when the following two conditions are met:

1) the rupture front propagates toward the site,
2) the direction of slip on the fault (rake) is aligned with the site.

The effects of forward-directivity are generated because the velocity of fault rupture is only slightly less than the speed of propagation of shear (transversal) waves in the surrounding rock mass. As the rupture front propagates from the hypocenter, a shear wave front is generated by the accumulation of the shear waves travelling ahead of the rupture front (Somerville et al., 1997). Forward-directivity effects yield a ground motion dominated by a velocity pulse characterized by large amplitudes and short durations (pulse-like motion). These effects are typically long period signals and are best observed as velocity or displacement time histories (Bray and Rodriguez-Marek, 2004). Conversely neutral or backward-directivity effects produce long duration motion of relatively low amplitude.

3.2 Fling step effects

The “fling step” is the static component of the near-fault ground motion and is characterized by a ramp-like step in the displacement time-history and a one-sided pulse in the velocigram. Several authors including Kostadinov and Yamazaki (2001), Faccioli et al. (2004) and Graves (2004) have proposed analytical formulations to model the fling-step effect. They differ for the mathematical functions used to approximate the velocity pulse (e.g. trigonometric, triangular, Gaussian etc.). Every model is described by two parameters: the final fault offset and the duration of the velocity pulse. Furthermore, Kostadinov and Yamazaki (2001) proposed a procedure to remove the static displacement in near-field seismic records in order to estimate the static component of peak ground velocity (PGV). Fling-step effects are independent from epicentre location and they decay rapidly with the distance “d” from the fault (i.e. they attenuate proportionally to $1/d^2$). Furthermore these effects are important only for surface ruptures say less than about 5 km from the fault trace (Graves, 2004).

3.3 Ground motion simulation

For many types of above-ground structures, the seismic action is often represented in the form of either an acceleration or a displacement response spectrum. For underground structures instead, a correct simulation of the response requires the execution of a complete dynamic time-history analysis. To obtain the acceleration time histories many options are available including artificial spectrum-compatible accelerograms, synthetic records generated by a seismological model of the source and real accelerograms (i.e. time histories recorded in real earthquakes). However, for underground structures located in the vicinity of a fault rupture, the ground motion should also reflect the features described above namely directivity effects and fling step. The use of real accelerograms is preferred, however it is influenced by two problems. First the lack of ground motion recordings below the ground surface and, second, the difficulty to adequately scale the time-histories recorded at the free surface. Even though the advent of digital accelerographs has increased the number of near-fault accelerograms, often the use of synthetic records and ground motion simulation is still required given the scarcity of actual recordings.
In the present research project the near-fault time-histories required for studying the seismic response of rock tunnels have been obtained from synthetic records generated using the approach proposed by Hisada and Bielak (2003) which is based on the application to a stratified medium of the method of generalized reflection and transmission coefficients using an extended kinematic source. The method explicitly considers the static offset due to surface faulting and allows one to investigate the effects of fling step and rupture directivity on the computed near-fault ground motions.

4. DESIGN ANALYSES OF UNDERGROUND STRUCTURES

Assessing the seismic response of an underground structure is a problem significantly different from that of a corresponding above-ground facility since the overall mass of the structure is usually small compared with the mass of the surrounding soil and the overall confinement acts as a strong damper of the seismic excitation. Therefore the seismic response of an underground structure is mainly controlled by the response of the surrounding ground and by the imposed ground deformation. The seismic response of underground structures may be assessed using two approaches (Hashash et al., 2001):

- free-field deformation approach;
- soil-structure interaction approach.

The two categories include various sub-methods characterized by different levels of approximation depending on design stage, knowledge of geologic setting, and geotechnical parameters. Concerning the types of analyses they may be grouped into three categories:

- pseudo-static;
- simplified dynamic;
- detailed dynamic analysis.

For engineering purposes underground structures may be assumed to undergo three primary modes of deformation during seismic shaking (Owen and Scholl, 1981):

- compression/extension;
- longitudinal bending;
- ovaling.

Only in detailed dynamic analyses the coupling between the response in the longitudinal direction (i.e. along the tunnel axis) and the one along the tunnel cross-section (i.e. along the transversal direction) is considered. In the following subsections the most important aspects related to the seismic response of underground structures as proposed within the present research project are briefly summarized. Also, a simplified approach in studying the seismic response of rock tunnels which takes into account the interaction of the underground structure with the surrounding ground and at the same time adequately considers the features of near-fault ground motion is proposed as developed during the first year of activities. Different analyses have been used for studying the transversal and longitudinal response. For the former the developed approach is typically pseudo-static whereas for the latter it is a simplified dynamic approach.

4.1 Analysis of transversal response (ovaling deformation)

The seismic response of an underground structure is controlled by the response of the surrounding ground and by the imposed deformation. In this regard, the most critical deformation pattern induced by ground shaking to a tunnel lining is the ovaling of the cross-section. The transversal behaviour is usually studied by analyzing the response of the cross-section to an imposed uniform strain field using the pseudo-static approach (Penzien, 2000). This is done for two reasons. Firstly, because the dimensions of a typical lining cross-section are small compared with the wavelengths of dominant ground motion producing the ovaling. Secondly, because the inertia effects in both the lining and the surrounding ground as produced by dynamic soil-structure interaction effects are relatively small (Penzien, 2000).
The analysis of the transversal response is performed by considering a lined circular tunnel in plane strain conditions. The earthquake loading is modelled as a uniform, quasi-static strain field simulating a pure shear deformation. The relations for displacements, bending moment, thrust and shear forces are 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 derived for two contact conditions at the structure-rock interface: full-slip and no-slip as described in detail in Enclosure 1. The true contact conditions at the ground-structure interface are unknown and the full and no-slip conditions simply represent the two extreme cases in which the real situation is bounded. The full-slip contact condition is usually adopted to obtain the

extreme values of the bending moment and shear in the tunnel lining whereas the no-slip assumption is used to find the maximum values of the thrust acting on the lining (Wang, 1993).
A key parameter for definition of the state of stress in the tunnel lining is the maximum shear strain evaluated in free-field conditions. For shallow tunnels the shear strain profile can be easily obtained by considering a horizontally layered system and using one-dimensional wave propagation theory (Wang, 1993). In near-fault conditions the assumptions of the previous approach are no longer valid (i.e. one-dimensional wave propagation theory with the wave front impinging in the vertical direction). To compute the earthquake-induced shear strain field in the vicinity of a causative fault, the displacement time histories at four points around the cross section of the tunnel were calculated using the Hisada and Bielak (2003) approach.

4.2 Analysis of longitudinal response (axial and bending deformation)

To study the tunnel response along the longitudinal direction (which involves axial and bending deformations) a finite element stick model has been purposely developed by subdividing the tunnel into a finite number of frame 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), representing the effects of ground deformability and energy dissipation (though Sommerfield radiation and material damping) . 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 inputted 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), 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). Also this model has been described in Enclosure 1.

4.3 Case studies

The simplified approach as developed so far, which allows one to consider both the transversal and longitudinal response of deep tunnels subject to earthquake loading need be validated. This is being done by taking typical case studies for which reliable input data are available and by comparing the results obtained by the simplified approach with the results derived by more advanced numerical analyses.
Several analyses involving the coupled effect of seismic source, propagation path, complex geological features and dynamic soil-structure interaction, are being been carried out by using a high-performance spectral element computer code, implemented in parallel architectures. The capabilities of this computer code are enhanced by the implementation of the so-called Domain Reduction Method (DRM), the main feature of which is to provide the exact coupling of the wave propagation solutions in different domains, so that the wave propagation from the seismic source, and the dynamic response of the tunnel can be effectively studied by two numerical models in series, with a significant saving in computer time.
At present, the seismic response of a rock tunnel located in Southern Italy, along the railway line connecting Caserta to Foggia is being considered (see Enclosure 3). As well known, the northern sector of southern Apennines, the Sannio region, is among the most active seismic regions in Italy as it was struck by four large destructive earthquakes in the last three centuries, the latest in 1805. The seismic analyses of the tunnel are carried out by simulating the reactivation of a fault located in close proximity to the tunnel. As a final step, the results obtained from advanced numerical analyses are compared with those determined with simplified methods: The aim is to assess the potentials of simplified methods in the design analysis of deep tunnels in rock with a close view paid to engineering applications and design needs.

5. MONITORING OF UNDERGROUND STRUCTURES

Infrastructure systems are the most critical part of the asset of a nation: anomalies or collapse can produce loss of human lives and negative consequences on the inner global product (Aktan et al. 1998). In many areas the ageing and maintenance of civil infrastructures introduce new problems for the asset management, whose solution requires technical, scientific and managerial abilities (Aktan et al. 2000). These aspects

caused in the last decades an increasing interest on the subject of structural health monitoring (SHM) from researchers belonging to different areas, not only engineering, but also physics, economics and biology.
SHM encloses the measurement operations of the environmental conditions, the external loads and the structural response for the identification of anomalies or damage which can cut down the serviceability or the safety of a structure (Aktan et al. 2000). A big amount of publications about SHM of bridges and buildings is now available, while the efforts made in developing monitoring systems for underground structures are far to be sufficient: these constructions, in fact, have been considered for a long time to be insensitive to extreme events such as earthquakes and hurricanes. Nevertheless, considering the complexity and the high costs related to underground structures, an efficient monitoring system must be regarded as a fundamental tool of control and mitigation of risks, both during the construction activities and the service life (Bhalla et al. 2005).
In the most general terms, damage can be defined as changes introduced into a system that adversely affects its current or future performance (Sohn et al. 2003): the basic premise of most damage detection methods is that damage will alter the stiffness, mass, or energy dissipation properties of a system, which in turn alter the measured dynamic response of the system. Although the basis for damage detection appears intuitive, its actual application poses many significant technical challenges. The most fundamental challenge is the fact that damage is typically a local phenomenon and may not significantly influence the lower frequency global response of a structure that is normally measured during vibration tests. Moreover, environmental and operational variations, such as varying temperature, moisture, and loading conditions affecting the dynamic response of the structures can often mask subtler structural changes caused by damage.
The damage state of a system can be described as a five-step process, each one related to the following questions:

- is there damage in the system (existence)?
- where is the damage in the system (location)?
- what kind of damage is present (type) ?
- how severe is the damage (extent)?
- how much useful life remains (prognosis)?

Answers to these questions in the order presented represents increasing knowledge of the damage state: the statistical models are used to answer these questions in an unambiguous and quantifiable manner (Sohn et al. 2003, Van der Auweraer & Peeters 2003).
A state-of the art has been produced, focusing its attention especially on sensors and current procedures employed in the case of tunnels. In the first part the basics of structural damage detection have been outlined and the modern probabilistic approach has been briefly presented (Sohn et al. 2003). The second part of the work refers a state-of-the-art of sensors and technologies for structural monitoring, considering both static and dynamic measurements.

6. ENCLOSURES

Enclosure 1: Corigliano M., Lai C., Barla G.: Seismic response of tunnels in near fault conditions. First European Conference on Earthquake Engineering and Sismology. Geneva, 3-8 September 2006, paper n. 998.

Enclosure 2: Example Form for Case Studies: Bolu Tunnel Case Study.

Enclosure 3: Corigliano M., Scandella L., Barla G., Lai C.G., Paolucci R.: Seismic analysis of underground structures: a case study in Southern Italy. Abstract submitted to the 4th International Conference n Earthquake Geotechnical Engineering, Thessaloniki, 2007.

6.3 SLOPE STABILITY

(Research coordinated by S. Rampello)

1. INTRODUCTION

The performance of slopes and earth structures subjected to seismic action can be evaluated through force-based pseudo-static methods, displacement-based sliding block methods, possibly including non linear soil behaviour, and fully coupled effective stress numerical analyses under dynamic loading.
In principle, numerical methods allow the most comprehensive analyses of the response of earth structures to seismic loading. However, reliable numerical analyses require accurate evaluation of soil profile, initial stress state and stress history, pore water pressure conditions, strength and deformation characteristics of the selected soil layers. Moreover, cyclic soil behaviour can be properly described only using advanced constitutive models developed within the framework of bounding surface plasticity or kinematic hardening plasticity, and requiring input parameters not usually measured in field or laboratory testing.
The displacement-based approach provides a compromise between the rather inadequate pseudo-static approach and the more refined numerical analyses; it has indeed the advantage of giving a quantitative assessment of the earthquake-induced displacement using a rather simple analytical procedure.
The displacement analysis can be carried out in its original form, assuming the critical acceleration a_c constant with time and the acceleration time history constant in the soil body, or using the decoupled approach. In the latter, the deformable response of the slope / earth structure is first accounted for through a dynamic response analysis, and the resulting acceleration time history is then used in a rigid sliding block analysis. The ground response analyses can be performed under 1D or 2D conditions, and the non linear soil behaviour is usually described through the equivalent linear method that provides a reasonable estimate of soil response for moderate levels of shearing intensity and provided that no significant excess pore water pressure develop during seismic shaking.
The displacement-based approach has been applied in a number of analyses of natural slopes and ideal or real earth dams. In a few documented case histories, it has been successfully used to back-calculate the measured seismic-induced permanent displacements.
It is worth noting that the displacement analysis is non capable of reproducing the deformation pattern of a slope / earth structure since actual deformations may be spread out over a zone, leading to bulging rather than sliding. Therefore, the computed permanent displacement should be always considered an index of seismic performance.
Despite of the simple analytical procedure required for applications and of their better prediction capability with respect to the pseudo-static methods, displacement-based methods are not commonly used in engineering practise because they require to represent the seismic action by appropriate acceleration time histories and then requiring, as a consequence, a proper knowledge of site seismicity. This is why the pseudo-static methods are the most widespread in engineering practice for the analysis of slope stability under seismic conditions.
In the pseudo-static approach, the earth mass is assumed to behave as a rigid-plastic material and to be in a state of limit equilibrium under the action of inertia and static forces. The inadequacy of this approach in predicting the performance of a slope subjected to earthquake loading has been recognised from a long time. The pseudo-static inertia force is in fact considered constant while the earthquake loading is typically a transient action characterised by abrupt changes in modulus and sign. As a consequence, during the earthquake, the ratio of the resisting to the driving forces may drop below unity for a short period of time and in limited portions of the slope only and this may induce some movement without causing a complete collapse of the slope. The static equivalent force is proportional to the weight of the potential sliding mass through a seismic coefficient K of horizontal and vertical components K_h and K_v, respectively. The horizontal seismic coefficient is usually expressed as a fraction of the maximum site acceleration.

The choice of the seismic coefficient K is crucial in the pseudo-static approach in that a single static force, constant with time in modulus, direction and sign, should represent the effects induced by the seismic action that is variable and of short duration.
In principle, the seismic coefficient should depend on the characteristics of both design acceleration time history and slope. In fact, two different accelerograms characterised by the same maximum acceleration but by different duration, predominant period and intensity, induce different effects on a given slope this implying different seismic coefficients to be used in the two cases. Moreover, assuming the seismic coefficient to be a fraction of the maximum acceleration at the site, implies acceptation of permanent seismic-induced displacement even though the pseudo-static analysis is satisfied. Then, the pseudo-static approach should be linked to the displacement-based approach and the seismic coefficient be defined selecting threshold values of earthquake-induced displacements.

2. ACTIVITIES

In the first semester of activity, documented case-histories of the main earthquake-induced landslides occurred in Italy and in the world have been summarised in a state of the art report. Among the Italian landslides, two case histories were selected: the slide of Andretta and that of Calitri, both occurred during the Irpinia earthquake of 1980. The main geological, geotechnical and seismic characteristics of these slides, together with a description of the observed failure mechanisms have been summarised to allow further studies to be carried out in this research. Tthe Andretta and Calitri landslides will constitute a benchmark for calibrating and evaluating the prediction capability of force-based, pseudo-static methods and displacement-based, sliding block methods of analysis on real case histories.
In the second semester of activity, the research was mainly devoted to the definition of criteria and procedures for selecting the seismic coefficient to be used in the pseudo-static methods, based on the equivalence between the results of pseudo-static and displacement-based methods.
In this way, the reduction η to be applied to $K_{max} = a_{max}/g$ (where a_{max} is the maximum acceleration at the site) to obtain the equivalent seismic coefficient $K_{eq} = \eta \cdot K_{max}$ may be evaluated taking into account the earthquake effects on the slope, through the specification of threshold values of displacement, and the earthquake characteristics, through analyses of ground response. Specifically, the reduction of the seismic coefficient to be used in the pseudo-static analyses could be linked to the slope ductility, i.e. to the slope capacity of undergoing specified thresholds of displacement without attaining general collapse, and to the slope deformability, that induce asynchronous slope movements thus producing a reduction of the equivalent seismic action to be used in the pseudo-static approach, in which rigid soil behaviour is assumed.
Preliminarly to these activities, a database of accelerograms of seismic events occurred in Italy was prepared, in order to obtain the input acceleration time histories to be used in the analyses of geotechnical systems such as slopes, earth structures, retaining structures, shallow and deep foundations, underground excavations. The database, includes 214 free-field records of the NS and WE horizontal components, obtained from 47 seismic events. The selected accelerograms refer to events of magnitude M_L = 4.0 – 6.6 and epicentral distance in the range of 1 to 87 km. The acceleration time histories have been divided in three groups depending on the soil type at the registration site: rock (V_s > 800 m/s), stiff soil (V_s = 360-800 m/s) and soft soil (V_s < 360 m/s). The database contains 74 records in rock, 98 on stiff soil and 42 on soft soil. Acceleration time histories recorded at epicentral distances greater than 100 km, relevant to seismic event of magnitude M_L< 4.0 have been excluded from the database in that characterised by negligible values of peak ground acceleration (< 0.05 g).
The displacement method has been applied in its original form (critical acceleration a_c constant with time and a(t) constant within the soil body) for expressing the reduction coefficient η as a function of the computed displacement d. To this purpose, for each soil type the accelerograms were scaled to the maximum accelerations specified by the O.P.C.M. for soil type A (rock or rock-like soil): a_g = 0.35g, 0.25g, 0.15g and 0.05g. Records to be scaled to each value of a_g were those with values of a_{max} = 0.05 – 2 a_g. Four sets of different accelerograms were thus obtained for the four seismic zones. Application of the displacement method to the four sets of accelerograms allowed the relations $\eta = \eta(d)$ to be obtained for each subsoil condition (rock, stiff and soft soil) and seismic zone. Upper bound equations characterised by an exceedance probability of 10% were used to this purposes. Similar evaluations were also carried out using empirical relationships proposed in the literature for evaluating permanent displacements induced by earthquake loading conditions.

The influence of soil deformability in reducing the equivalent static force to be used in the pseudo-static approach was taken into account evaluating an equivalent acceleration time history relevant to the potential sliding mass, through one-dimensional or two-dimensional ground response analyses. In a simplified procedure proposed after 1D ground response analyses on 21 soil profiles, the maximum acceleration of the equivalent accelerogram was expressed as a function of the ratio between the fundamental period of the subsoil T_s and the mean period of the acceleration time history T_m.
Finally, advanced constitutive models, in which the soil is assumed to behave as an elastic-plastic medium with isotropic and kinematic hardening, were implemented into two codes: one used for research purposes (Swandyne II) and the other commercially available (Plaxis V.8). Control analyses were carried out simulating cyclic triaxial or simple shear tests on a soil element, or considering simple boundary value problems for which there are available solutions.
Ideal slopes of given geometry and mechanical characteristics were also analysed for selecting the cases to be studied in the centrifuge.

3. RESULTS

Application of the displacement method to the database of accelerograms of seismic events occurred in Italy permitted to relate the reduction factor η to the computed displacement *d* for each kind of subsoil and for each seismic zone; reference was made to upper bound relationships characterised by an exceedance probability of 10%. The η-*d* relationships can be expressed in the form:

$$\eta = \frac{K_y}{K_{max}} = \frac{ln(d/B)}{A}$$

A and B being best fitting constants. It has been shown that the relationship depends on a_{max}; specifically, it is characterised by the same value of A and by values of B increasing with a_{max}. The upper bound curves were then obtained accounting for the maximum acceleration at the site, as specified in the O.P.C.M.:

$$a_{max} = S \cdot S_T \cdot a_g$$

Similar results were obtained using empirical relationships published in the literature for evaluating the earthquake induced displacements. Also, a complete equivalence between the pseudostatic and the displacement analyses was shown for the case of the infinite slope, demonstrating that the reduction factor is a function of both the earthquake and the slope characteristics.
Apart from the differences related to the use of different relationships for computing the earthquake induced displacements, the studies mentioned above showed that a significant reduction of the equivalent static force to be used in the pseudo-static methods can be obtained depending on the adopted threshold values of displacement.
A further reduction of the static equivalent force was evaluated accounting for soil deformability in the ground response analysis. Reference was made in this case to the maximum acceleration, $a_{eq,max}$ of the equivalent accelerogram acting in the potential sliding mass.
Values of $a_{eq,max}$ were normalised with respect to the maximum acceleration at the site $a_{max} = S \cdot a_g$ and were plotted against the ratio between the subsoil fundamental period T_s and the average period of the acceleration time histories T_m. Four different approaches were adopted for evaluating the coefficient of soil amplification S. Substantial reduction of the equivalent maximum acceleration were obtained with increasing values of T_s/T_m and with increasing soil deformability. This occurrence can be attributed to asynchronous movement of the potential sliding mass during earthquake loading.
Although results obtained from the different Unities of Research must be compared in a greater detail and be rendered compatible, it is agreed that it should be abandoned the idea of constant values of the seismic coefficient irrespective of the subsoil conditions, earthquake characteristics and adopted values of threshold displacement. Upper bound relationships between the reduction factor η to be applied to $K_{eq,max}$ and the displacement *d* could be used as an alternative, $K_{eq,max}$ being corrected to account for the soil deformability. Specifically, $K_{eq,max}$ can be obtained taking into account the coupling between the ground response of the potential sliding mass and the frequency content of the input seismic action.

4. CONFORMITY TO THE PROGRAM

The research carried out in the first year of activity mainly involve themes No.3 and No.4 relative to comparison between methods of analysis of increasing complexity for evaluating the seismic performance of slopes and earth structures.

These activities have been anticipated with respect to the time table proposed initially since the centrifuge set up for carrying out dynamic test is not yet completed. Laboratory testing has also been delayed to the time when the material to be tested in the centrifuge will be chosen.

6.4 DEEP FOUNDATIONS

(Research coordinated by A. L. Simonelli)

1. INTRODUCTION

The main objective of the project is to individuate elements to be introduced in the technical code, regarding the seismic design of deep foundations, with particular attention to the effects of kinematic interaction.
In order to achieve the final goal, the following activities should be faced and the relative objectives obtained:
- organization of the actual knowledge on the seismic behaviour of piles, and in particular on the kinematic interaction phenomenon;
- classification of computational procedures, on the basis of the degree of complexity (physical-mathematical modelling and relevant algorithm);
- definition of a wide range of reference cases, differing for characteristic parameters of the exciting waveform, soil layering conditions, geometry of the foundation system (from the single pile to the pile group, with a wide variability of the meaningful geometrical parameters);
- extensive pseudo-static and dynamic analyses of such reference cases, by means of the selected computational procedures, with the aim of individuating reference types of “performance”;
- identification of computational tools, different for degree of complexity and for the evaluation of the kinematic interaction bending effects (the same goal should be achieved for the inertial effects too);
- planning and execution of some laboratory tests on physical models, with the aim of corroborating the results acquired through the numerical analyses.

According to the planned timetable, during the first year the following goals have been achieved:
(a) literature review and state of art report on the seismic behaviour of piles and kinematic interaction;
(b) identification of several reference numerical models.

Further, some preliminary dynamic analyses have been carried out, with the aim of properly testing different numerical models under examination.

2. ACTIVITIES

As regards the activity (a), at present there are numerous scientific contributions on the kinematic interaction for deep foundations, mainly on behalf of few research groups: however the results are still rather fragmentary and a common and consolidated knowledge does not yet exist. Furthermore, even in the "static" field some uncertainties still remain on the real interaction between the soils and the piled foundations. Therefore it has been necessary to organise the global knowledge on this topic.
The report on the state of the knowledge mainly put in evidence: the various proposed and consolidated levels of analysis, and the significance of the achieved results; the validity of the simplified hypotheses usually adopted in the different analyses of the soil-structure interaction (e.g.: the hypothesis of foundation fixed to the base); the conservativeness of the results that come out from the adoption of the simplified hypotheses. This study finally indicates the guidelines for the subsequent numerical analysis.
All the 5 Research Units (RU) involved in the project have contributed to activity (a).
As regards activity (b), all the different numerical models proposed in the scientific literature (including those already adopted by the RU) have been evaluated. Some reference numerical procedures have been identified and grouped on the basis of the degree of complexity of both the physical-mathematical model and the computational algorithm.
The report on the reference numerical modelling constitutes a tool for the attainment of the objectives of the research phases that will follow. In particular, since the reference numerical models have been almost

defined, the proper tools of analysis (computational codes) have been identified and are going to be acquired, with the aim of carrying out the numerical analyses scheduled for the second and third year of research.
The activity (b) has been carried out mainly by 3 RU (Univ. of Basilicata, Univ. of Catania and Second Univ. of Napoli) with reference to pseudo-static horizontal loads on piles and piled foundations (simulating inertia forces), and by two RU (Univ. of Sannio and Univ. of Calabria) with reference to kinematic interaction.
Three official meetings among all the RU have been organised during the first year, while a number of informal meetings between researchers of different RU took place often during the year.
Further, inside Reluis activities the Sannio RU has organised a seminar at Benevento, inviting prof. George Mylonakis, from Patras University (Greece). Prof. Mylonakis gave a lecture on “Seismic SSI effects for Surface & Deep Foundations: Theory, Numerical Analysis, Case Histories”; many researchers throughout Italy attended the seminar, besides all the members of the RU of Reluis 6.4 research line.

3. RESULTS

3.1. Literature review

3.1.1. Vertically loaded piled foundations

Simple numerical codes for the analysis of soil-structure interaction in the case of vertically loaded piled foundations are available. The Boundary Element Method provides the most widespread technique for the analysis of pile groups and piled rafts, even if a number of contributions on the use of the Finite Element Method can be found particularly in very recent publications.
A number of well-documented case histories of full-scale vertically loaded piled foundations have been published in literature. The results obtained by comparing the available procedures of analysis to the experimental evidence support the conclusion that satisfactory predictions may be obtained by the existing codes, provided that the real problem is properly modelled.
Traditional design methods for piled foundations still require that piles have to carry the total structural load. Neglecting the contribution of the raft is unduly conservative, but it is adopted as a common practice by engineers in many countries and prescribed by the majority of existing codes, including Italian regulations. On the other hand, recently innovative foundations in which the contact between raft and soil is fully considered (*piled raft*) are widely studied. A number of reports have been published on the use of piles as settlement reducers and some applications have been reported. A new code has been recently approved in Italy (September 2005) which allows to take into account the load sharing between piles and raft.

3.1.2. Horizontally loaded piles and piled foundations

3.1.2.1. Experimental evidence on single pile behaviour
A single pile under horizontal static load behaves essentially as a flexible element: the horizontal displacements y of the pile (and its surrounding soil) generally vanish at depth ≅ 10 d (d = pile diameter). This implies that: (i) the full length L of the pile is rarely significant; (ii) mechanical properties of top soil layers govern the pile-soil system behaviour. The location of the maximum bending moment depends on boundary conditions at the pile head. Two limit conditions are usually examined: (a) fixed and (b) free head. In the former case, the maximum bending moment occurs at the pile head, while in the latter it occurs along the pile shaft, at a depth equal to a few diameters, depending on pile-soil relative stiffness. The ultimate horizontal capacity of the pile-soil system is governed by the structural capacity of the pile section The yield bending moment of the pile section is thus a key parameter to be accounted for in the analyses.

3.1.2.2. Experimental evidence on pile group behaviour
In addition to the factors previously described, pile group behaviour is governed by some further parameters, such as: (i) mutual pile-soil-pile interaction; (ii) stiffness of the structure (raft/beam) connecting pile heads; (iii) raft (beam)-soil interaction. The experimental evidence available in the literature is relative to free-standing piles connected by rigid rafts; as a consequence, it is focused only on mutual pile-soil-pile interaction, whose main features can be summarised as follows:
- pile-soil-pile interaction reduces the stiffness of the overall system;

- piles in a group behave differently according to their position.

3.1.3. Soil-structure interaction under seismic loads

3.1.3.1. Foundation input motion and kinematic interaction

In linear systems, seismic soil-structure interaction can be split in two distinct phenomena: kinematic and inertial interaction. The first (*kinematic interaction)* is due to the presence of the foundation that causes foundation displacements to deviate from the free-field motion of the soil; the second (inertial interaction) is due to the superstructure shaking that induces additional deformations to the supporting soil.
The analyses are thus conveniently performed in a multistep approach, known as substructure method, consisting of the following steps:

- evaluation of the foundation input motion, i. e. the resulting motion of the foundation from kinematic interaction, in the absence of the superstructure;
- computation of the dynamic impedances at the foundation level;
- evaluation of the response of the superstructure supported on the dynamic impedances and subjected to the foundation input motion.

Pile damage due to seismic shaking has been observed after numerous strong motion events (see, Loma Prieta in 1989 and Kobe in 1995). Identified or suspected causes of failure in most of the cases are:

1) large pile movements due to liquefaction phenomena and lateral soil spreading;
2) excessive forces (bending and shear) coming from the superstructure;
3) pile bending due to deformation induced by the passage of seismic waves through the soil.

Pile loading due to the causes listed at item 1 and 3 are imposed directly along the pile shaft, and develop regardless of the presence of a superstructure.
Analytical and field evidences do confirm that at the interface of two layers having different shear moduli, soil shear strain is discontinuous and the associated curvature (which is the derivative of strain) is infinite. At the same time pile curvature is finite, specially if pile is assumed to behave elastically. The different response of the pile and the surrounding soil, generates additional stresses in the pile (*kinematic interaction*). Kinematic bending moments mainly depend on:

- sharp stiffness contrast between two consecutive soil layers;
- the boundary conditions at the head of the piles;
- the proximity of the excitation frequency to the fundamental natural frequency of the soil deposit;
- the depth of the soil layer interface, respect to the active length of the pile.

The kinematic response of piles has received less research attention as compared to its inertial counterpart and little attention by engineers in practice.

3.1.3.2. Analysis methods for studying pile kinematic interaction

The kinematic interaction has been studied by numerous researchers throughout the world. Closed-form expressions have been derived for computing the maximum steady-state bending moment at the interface between two layers.
The different approaches proposed in the literature can be grouped in:

1) <u>simplified methods</u> assuming that the pile follows the free-field soil response; this approach (Margason, 1975, and NEHRP, 1997) can be applied only to homogeneous soils.
2) <u>Winkler models</u> in which the pile is connected to the soil through continuously-distributed springs and dashpots (Flores-Berrones & Whitman, 1982; Kavvadas & Gazetas, 1993). In these methods, hereafter called Beam on Dynamic Winkler Foundation (BDWF), the springs represent soil stiffness while the dashpots represent soil damping due to radiation and hysteretic energy dissipation in the medium. The wave-induced motion in the soil represents the input excitation of the pile-soil system. An explicit solution has been given by Dobry & O'Rourke (1983) for determining kinematic bending moments at an interface separating two soil layers of different stiffness.
   An alternative formulation has been proposed by Nikolaou et al. (2001). It refers to harmonic steady-state bending moments developing in a cylindrical pile at the interface between two layers of different stiffness, under resonant conditions.
   Mylonakis (1999) developed a model analogous to the Dobry & O'Rourke (1983) one, which incorporates the geometric characteristics (thickness) of the soil layers and the dynamic nature of the excitation.
3) <u>numerical methods based on a continuum approach</u> where the soil, pile and superstructure are modelled as a whole. The geometry is 3-D and typically discretized by F.E.M. techniques. Soil should be modelled by elastoplastic models of the so-called Advanced Plasticity to be able to reproduce soil response under ciclyc loading conditions. A simplified continuum approach is developed by Wu & Finn (1997) that

simulate vertically propagating shear waves with disregarding seismic induced deformations in the vertical direction and normal to the direction of shaking (quasi 3-D approach).

3.2. Numerical simulation

3.2.1. Horizontally loaded piles and piled foundations

3.2.1.1. Available procedures: soil modelling and numerical methods

The procedures may be classified as belonging to three broad categories, depending on the model assumed to simulate soil behaviour and the numerical method adopted to solve the governing equations :

(A) soil modelled as a Winkler medium. The independent springs can be defined by means of appropriate non linear p-y curves, derived from the experimental evidence. They allow to incorporate several features, such as soil stiffness varying with depth and soil nonlinearity. (Castelli, 2006)

(B) soil modelled as a continuum, solutions obtained by means of either (B1) BEM or (B2) FEM approaches. Both BEM and FEM approaches have been initially used assuming the soil as an elastic linear homogeneous continuum. This assumption can be modified to account for soil layers with different stiffness, soil collapse, soil non linearity even in the elastic range; to this aim, FEM procedures prove to be extremely versatile, even in some commercial codes, such as for instance ABAQUS and PLAXIS.

3.2.1.2. Selection of numerical procedures to perform parametric analyses

In the framework of this research project, it appeared appropriate to investigate the potentials of procedures belonging to the three different categories previously defined. To this purpose, two original procedures have been and are being implemented, each respectively belonging to category (A) and (B1), and correspondingly referred to as code (A) and code (B1), while a commercial FEM code (B2, in this document) has been selected to perform numerical analyses falling in category (B2).
In this section a brief description of the features of code (B1), whose development is still in progress, is subsequently given, while codes (A) and (B2) and some preliminary results obtained through their use will be described in the following section.
Code (B1) is a BEM code, modified to incorporate a soil model accounting for soil collapse, for soil layers with different stiffness and strength, and to account for yielding of the pile section. Both the pile and the soil layers are assumed to be elasto-plastic; it is envisaged to assume a more complex moment – rotation relationship up to failure. The code is aimed at analysing both a single pile and a pile group.

3.2.1.3. Some preliminary numerical results

3.2.1.3.1. Results by Code (A)

Code (A) assumes the soil to behave as a Winkler medium, adopting hyperbolic functions as p-y curves, modified through the assumption of the “p-multiplier” (Brown et al., 1988) to account for group effects, with the aim of simulating the “shadowing effect”. Reductions factors are determined back-analysing full scale load tests performed both on single piles and pile groups. Preliminary results seem to support that this procedure can not only reproduce satisfactorily the response of a single pile response, but can also capture the essential features of pile group behaviour.

3.2.1.3.2. Results by Code (B2)

Code (B2) is the FEM commercial code ABAQUS ver. 6.3. Parametric 3D analyses were carried out for a single pile under static horizontal loading resting on a granular soil with mechanical properties varying with depth, and assuming both free and fixed head conditions.
The soil was assumed to be elasto-plastic, obeying Mohr-Coulomb failure criterion. In the linear elastic range, the Young’s modulus E was assumed to vary with depth according to a bi-linear law. Pile diameter d was assumed to be 1 m, while its Young’s modulus E_P was set equal to 25 GPa. Furthermore, it was assumed that the installation procedure induces no change in the in situ soil stress conditions (*wished in place assumption*), which should be appropriate for a bored pile.
The objectives of the research are: 1. To explore the influence on the load deflection response of factors like the pile slenderness L/d and the pile-soil relative stiffness; 2. To study the influence of axial forces applied at pile head on the response of a lateral loaded single pile; 3. To provide benchmark solutions against which simpler analyses may be compared; 4. To define an equivalent static horizontal force capable to reproduce the maximum bending moment in the pile evaluated by dynamic analyses.
A preliminary study has been performed to assess the influence of the mesh refinement in both the

horizontal and the vertical directions. The analyses were performed under displacement control. The lateral (vertical) loading was simulated by applying uniform increments of horizontal (vertical) displacements to the nodes belonging to the pile head. Preliminary results of this study show that:

1. The load deflection response of a single pile is not affected by the pile slenderness, while it is strongly affected by the pile-soil relative stiffness;
2. An (initial) axial load distribution does not exert a significance influence on the lateral response of a single pile, provided that the material response of the pile is linear elastic;
3. An axial load primarily affects the bending moment corresponding to the formation of a plastic hinge and, thus, the extent (load or deflection) at which the load deflection curves evaluated by adopting a linear elastic model for the pile become unrealistic;
4. The comparisons vs. numerical solutions evaluated by the BEM method appears to be promising, even if they have been limited to a small number of cases where the soil stiffness varies smoothly with depth.

3.2.2. Soil-structure interaction under seismic loads

The most suitable analysis methods, which are going to be adopted in the next two years of activity, can be grouped in:

1) simplified BDWF methods as those developed by Kavvadas & Gazetas (1993), Mylonakis (1997) and Nikolau et al. (2001). This kind of approach has been generally adopted for parametric analyses aimed at defining guidelines for several seismic normatives in the world. It should be noted that these methods are limited by the assumptions of linearity in the soil and the pile, perfect bonding at the pile-soil interface, and seismic excitation due to exclusively vertically-propagating S waves. Accordingly, some of the obtained results may not hold in cases deviating significantly from the assumed ones.
   In addition, another method originally developed by Conte & Dente (1988) for the seismic analysis of piles in layered soils, will be reviewed. In this method, the pile is modelled as a linearly elastic beam connected to the surrounding soil by non linear springs and dashpots, which provide the interaction forces in the lateral direction. The stiffness of the springs is related to the soil shear modulus updated to the current strain level induced by the seismic motion, according to a non linear hysteretic Ramberg-Osgood type model. The radiation damping coefficient is evaluated using simple analytical expression available in the literature. The non linear response of the soil in free-field condition is achieved by a numerical technique based on the characteristics method, under the assumption of shear wave one-dimensional propagation (Conte & Dente, 1987). The method accounts for important effects such as heterogeneity and non linearity of the soil, energy dissipation due to material and radiation damping. In addition, the presence of a superstructure, idealized as a number of masses connected by elastic beams, can be considered to analyse directly the response of the superstructure-pile-soil system in time domain.
2) numerical methods based on a continuum approach and spatial discretization by FEM or BEM techniques. These analyses, more heavy from a computational point of view, are going to be performed by ABAQUS 3-D and VERSAT-3D (Wu & Finn 1997) on soil-pile-superstructure sample combinations with the aim to test the results of simplified approaches. In addition a hybrid boundary element formulation is going to be developed for the analysis of single piles and pile groups under vertically and obliquely incident seismic waves, starting from the analogous approach for the analysis of single piles and pile groups subjected to vertical loading, under static and dynamic conditions (Cairo et al., 2005, and Cairo & Conte, 2006).

3.2.2.1. Some preliminary numerical results

During the first year of activity, numerical analyses have carried out by means of a BDWF method (*Beam on Dynamic Winkler Foundation*) developed by Mylonakis (1997), licensed by the literature study as a fruitful and cost-effective approach to extensively study kinematic interaction of piles.

The first set of analyses refers to a fixed-head pile in a bi-layered subsoil. Soil shear stiffness contrast was changed from 16 to 4 which implies shear wave velocity ratio varying from 4 to 2; the velocity values have been chosen in order to have type D subsoils, according to the classification given by EC8 (or OPCM 3274/2003). Italian accelerograms were scaled in magnitude to provide a rock peak acceleration consistent with the seismic zone taken into account. The computed kinematic pile bending moments at the interface were systematically compared to the results given by the predictive formulas suggested in literature (Dobry&O'Rourke, 1983; Nikolaou et al., 2001) and to the yielding moments of the adopted pile section for different concrete reinforcement conditions. The results show that kinematic moments can be very high and well above the yielding moment of the section. This statement evidences the crucial matter of preserving the hypothesis of pile elastic behaviour. Recent developments still confined to the research field (Gazetas, 2006) move towards assuming plastic behaviour of piles.

4. CONFORMITY TO THE PROGRAM

According to the research program, task 1 and 2 activities have been performed during the first year. In particular the literature review has been finalised to the state of art report, and the reference numerical models to be implemented for the second and third year extensive analysis have been individuated.
Further, some preliminary dynamic analyses have been anticipated with respect to the proposed time table. Finally, preliminary contacts with the University of Bristol have been taken in order to plan the laboratory research; according to them, the experimental testing activity should be performed at the beginning of the third year.