Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

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NOTATION

\( \alpha \) is the steep of the cycle resistance curve
\( SR \) is the shear stress ratio
\( N \) is the number of cycles
\( N_{eq} \) is the number of equivalent cycles
\( SR_t \) is the threshold value of the shear stress ratio
\( SR_r \) is the reference value of the cyclic resistance curve
\( N_r \) is the reference number of cycles, equals to 15
\( a \) is a parameter of the pore water pressure relationship
\( b \) is a parameter of the pore water pressure relationship
\( c \) is a parameter of the pore water pressure relationship
\( n \) is the number of elements
\( m_j \) is the mass of the generic sub-layer \( j \)
\( c_j \) is the viscous damping coefficient of the generic sub-layer \( j \)
\( k_j \) is the spring stiffness of the generic sub-layer \( j \)
\( r_j \) is the reaction of plastic slider (i.e. the limit shear strength) for a generic sub-layer \( j \)
\( \rho_j \) is the density of the \( j \)-th sub-layer
\( h_j \) is the thickness of the \( j \)-th sub-layer
\( G_j \) is the shear stiffness of the \( j \)-th sub-layer
\( \ddot{u}_a \) is the absolute acceleration
\( \dot{u}_a \) is the absolute velocity
\( u_a \) is the absolute displacement
\( \ddot{u}_g \) is the base ground motion acceleration
\( \dot{u}_g \) is the base ground motion velocity
\( u_g \) is the base ground motion displacement
\( a_{max} \) is the peak ground acceleration of the input motion
\( M \) is the mass matrix
\( C \) is the viscous damping matrix
\( K \) is the stiffness matrix
\( f \) is the vector of applied forces
\( \mathbf{i} \) is the vector with each element equal to zero except for the last that is equal to unity
\( \rho_r \) is the density of the bedrock
\( V_{S,r} \) is the shear wave velocity of the bedrock
\( V_S \) is the shear wave velocity
\( f_{\text{max}} \) is the maximum frequency of the input motion
\( M_s \) is the mass matrix for the sliding part of the dynamic system
\( C_s \) is the viscous damping matrix for the sliding part of the dynamic system
\( K_s \) is the stiffness matrix for the sliding part of the dynamic system
\( \ddot{u} \) is the relative acceleration
\( \ddot{u}_0 \) is the sliding acceleration
\( \ddot{u}_{\text{eq}} \) is the equivalent acceleration
\( \dot{u}_0 \) is the sliding velocity
\( u_0 \) is the relative sliding displacement
\( h_{\text{max}} \) is the maximum sub-layer thickness
\( a_R \) is the constant of the viscous damping matrix which multiplies the mass matrix
\( \beta_R \) is the constant of the viscous damping matrix which multiplies the stiffness matrix
\( \zeta \) is the soil damping ratio
\( \gamma \) is the shear strain
\( \gamma^* \) is the actual shear strain of the spring above the sliding base
\( \zeta_{\text{exp}} \) is the hysteretic damping measured in laboratory tests
\( G_0 \) is the initial shear modulus
\( \gamma_r \) is the reference shear strain of the MKZ model
\( \beta \) is the dimensionless factor of the MKZ model
\( s' \) is the dimensionless factor of the MKZ model
\( \gamma_c \) is the reversal shear strain
\( \tau \) is the shear stress
\( \tau_c \) is the reversal shear stress
\( \gamma_m \) is the maximum shear strain
\( F(\gamma) \) is the generic stress-strain relationship
\( F_{\text{bd}}(\gamma) \) is the back-bone curve
\( F_{\text{ur}}(\gamma) \) is the unloading-reloading curve
\( F'(\gamma_m) \) is the damping reduction factor
\( \zeta_{\text{Max}} \) is the hysteretic damping calculated using the Masing rules
\( p_1 \) is the first non-dimensional parameter of the damping reduction factor \( F'(\gamma_m) \)
\( p_2 \) is the second non-dimensional parameter of the damping reduction factor \( F'(\gamma_m) \)
\( p_3 \) is the third non-dimensional parameter of the damping reduction factor \( F'(\gamma_m) \)
\( G(\gamma_m) \) is the secant modulus corresponding to the maximum shear strain level, \( \gamma_m \)
\( \mathbf{x} \) is the vector of unknown variables
\( x_t \) is the vector of unknown variables at time \( t \)
\( \gamma_N \) is the default coefficient used in the Newmark \( \beta \) method
\( \beta_N \) is the default coefficient used in the Newmark \( \beta \) method
\( q \) is the vector of external applied forces
\( q_{t+1} \) is the vector of external applied forces at time \( t+1 \)
\( A \) is the matrix of the Newmark \( \beta \) method
\( B \) is the matrix of the Newmark \( \beta \) method
\( I \) is the unit matrix
\( 0 \) is the null matrix
\( \Delta \tau_j \) is the shear stress increment of the \( j \)-th soil layer
\( \Delta t \) is the time interval
\( \Delta \gamma_j \) is the incremental value of shear strain
\( u_j^t \) is the displacement at time \( t \)
\( u_j^{t+1} \) is the displacement at time \( t+1 \)
\( \Delta u_j \) is the displacement variation in time interval \( \Delta t \) of the \( j \)-th soil layer
\( \varepsilon_{rel} \) is the maximum value of relative error
\( \varepsilon_{tol} \) is the fixed tolerance value
\( \left[ x_j^{\gamma_{t+1}} \right] \) is the generic matrix calculated at the \( t+1 \) time as \( k \)-th attempt of the \( j \)-th soil layer
\( p' \) is the mean effective stress
\( p_{ref} \) is the reference pressure (100 kPa)
\( K_G \) is the stiffness coefficient
\( IP \) is the plasticity index
\( M_w \) is the moment magnitude
\( M_L \) is the local magnitude
\( I_A \) is the Arias intensity
\( PWP \) is the acronym of pore water pressure
\( PARI \) is the acronym of Port and Airport Research Institute
\( USGS \) is the acronym of United States Geological Survey
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Finally, I extend my deepest gratitude to my family for their patience, understanding and encouragement, and I dedicate this thesis to them.
ABSTRACT

Widespread liquefaction evidences characterized the most recent earthquakes in Italy (Emilia, 2011) and in the world (Canterbury, New Zealand, 2010-2011), resulting in a renewed interest on the characterisation and modelling of pore pressure build-up during seismic events. In particular, it has been highlighted the roughness of some traditional tools for assessing the liquefaction susceptibility as well as the overall complexity of the available numerical models. Therefore, it seems necessary to develop reliable analysis method that comprehensively take into account for the complexity of the mechanical behaviour of soils, even though remaining accessible for professional practice.

In this work, a simplified constitutive model, incorporated in a one-dimensional code, is proposed. It allows to pursuing a simplified dynamic analysis: it is easy to be applied as the semi-empirical methods and reliable as an advanced numerical approach. The proposed model is based on the results of laboratory undrained cyclic tests. It is easy to calibrate and reasonably approximates the main features of soil behaviour. The numerical formulation of the model has been incorporated in a one-dimensional computer program for seismic soil response, in order to develop a useful tool for engineering practice. In the same code it has been also implemented a numerical routine modelling the pore pressure dissipation thus allowing to simulate both the pore pressure build-up during the seismic event as well as its dissipation.

The upgraded code has been validated using test sites data in order to provide an extensive documentation of the capabilities and limitations of the implemented model in prediction the seismic site response under strong ground motions.

Finally, the developed code has been used to perform specific analyses on a dyke, severely damaged by the 2012 Emilia earthquake sequence. Taking into account the aftershocks that followed the main seismic event, effective stress analyses have been allowed a better understanding of the damage source detected along the embankment.
I. INTRODUCTION

Since the catastrophic failures due to soil liquefaction occurred during Niigata and Good Friday (Alaska) 1964 earthquakes, increasing efforts have been devoted to the understanding and modeling of the phenomenon as well as to the development of effective methods for the prediction the liquefaction potential of soil deposits. In the last 40 years since these earthquakes, liquefaction has been extensively studied extensively by hundreds of researchers around the world. Much has been learned, but the road has not been smooth.

Pioneering studies on liquefaction phenomena were based on measurements of pore pressure build up during undrained cyclic laboratory tests carried out on loose and dense sand. These studies highlighted the dependence of cyclic resistance on factors such as soil density and shear stress amplitude. Subsequent work showed that cyclic resistance measurements are strongly influenced by soil fabric, stress history, overconsolidation ratio and lateral earth pressure coefficient. All these factors can be easily destroyed by sampling disturbance and hardly replicable in reconstituted specimens. For these reasons and thanks to the increasing availability of field data related to liquefaction case histories, these data started to be used to characterize liquefaction resistance in terms of measured in situ test parameters.

The Cone Penetration Test (CPT) and Standard Penetration Test (SPT) are the two most widely used indices for evaluating the liquefaction characteristics of soils. Shear wave velocity (Vs) tests, tend to be used in special situations and thus are used less often than the SPT and CPT in liquefaction evaluations.
Both the approaches, eventually used in conjunction with a simplified dynamic analysis, permit to evaluate the conditions (seismic demand and soil capacity) than can trigger liquefaction but does not allow analyzing the stress-strain evolution of a soil volume. Furthermore, they have some limitations and need some assumptions, such as to convert the irregular earthquake history into an equivalent uniform load. For this reason, in the last fifteen years, a number of constitutive models have been developed to simulate the cyclic loading of non-cohesive soils, with emphasis on cyclic mobility and/or flow liquefaction, basically built on the theory of incremental plasticity. Adequate complexity of constitutive modeling allows efficient prediction for widely different boundary value problems and different loading paths. On the other hand, complex models need specific expertise and often require a great number of geotechnical parameters that are not easily available. Nevertheless, simpler approaches based on empirical charts or simplified dynamic analyses have been also developed over the years. These approaches still remain the most used tools in geotechnical engineering practice since the difficulty to use demanding constitutive models.

In this thesis, a simplified constitutive model aimed at simulating the pore pressure buildup in a soil element is proposed. It has been implemented in a one-dimensional code thus allowing to carry out simplified 1D dynamic analyses in effective stress conditions. The performances of the code are evaluated by analyzing different liquefaction case histories. The comparison between recorded and simulated data highlighted that notwithstanding the simplicity of the proposed approach the code provides reliable results.

**I.1. Definition of the problem**

Dependence of soil response on the level of seismic loading involves rather complex mechanical processes, which may be grouped roughly in two main classes. The first is the degradation of mechanical properties of the material, which is characterized by a decrease in the shear modulus coupled with an increase in energy dissipation; while the second is related to the volumetric-distortional coupling, that takes place at higher stress state and could induce either volumetric strains of the soil skeleton under shear stress in drained conditions, or pore pressure changes in water-saturated soils in undrained conditions, that, in turn may evolve into liquefaction (in the case of loose sandy soils).

While the importance of first class of non-linear effects has been widely recognized and modelled in the most common and commercial codes performing seismic site response (Hashash et al., 2015; Geostudio, 2007; ITASCA, 2011), the modelling of pore pressure
change still remains a challenging issue, due to the limitations of simplified approaches and the specialized knowledge required by advanced constitutive models. The first goal of the present study is to search and develop a constitutive model that is easy to calibrate utilizing the limited data usually available and that can reasonably approximate the main characteristics of soil behaviour. The second goal is to incorporate the model in a simple computer program in order to develop a useful tool functional for engineering practice. An extensive documentation of the capabilities and limitations of the implemented model in predicting the seismic soil response under strong ground motions is finally provided.

1.2. Organization of the text

The present research consists of seven different and related chapters. Chapter one reports the description of the problem and the definition of the aim of the study. Chapter two contains a general overview of the gained experience on liquefaction as results of the recent seismic events and the existing analysis methods. Chapter three describes the original and updated formulation of a simplified pore water pressure build up model. The model is based on the results of cyclic laboratory tests; the calibration of the model parameters has been analyzed in details considering both simulated and experimental data. A pore pressure dissipation model based on one-dimensional consolidation theory has been also integrated in the formulation in order to simulate both generation and dissipation of excess pore water pressure. Chapter four details the implementation of the integrated pore water pressure and dissipation model inside a numerical computer program. In Chapter five, the performances of the upgraded code are assessed on well documented case histories in which vertical array records and, sometimes, pore pressure buildup measurements, are available. Both the accuracy and practical usefulness of the code are assessed by comparing calculated predictions and field records. Chapter six presents the case history of a dyke damaged during the seismic sequence occurred in Emilia plain (Italy) in May, 2012. Effective stress analyses have been adopted for studying deeply the source of damage induced by the earthquake, taking into account also the sequence of three seismic events occurred in less than ten minutes. Finally, Chapter VII presents the conclusions and the future perspectives of the research.
I.3. References

II. BACKGROUND

In this chapter, a brief review of the lessons learned from the occurrence of liquefaction due to strong seismic events in Italy and around the world is described. A classification of the available methods to evaluate the site liquefaction susceptibility is therefore presented in the second paragraph highlighting advantages and limitations of the different approaches. Finally, the last paragraph focuses on evaluating the capabilities of one-dimensional, fully nonlinear effective stress procedures in modeling the seismic response of sites where significant pore water pressure buildup is expected.

II.1. Deformation phenomena and liquefaction occurred in recent earthquake

Most of the liquefaction phenomena observed during the last strong earthquakes interested reclaimed lands, landfills and natural or artificial embankments.

One of the most popular cases is the massive liquefaction phenomena occurred during the strong earthquake which struck Kobe city (Japan) in 1995. All the damaged sites correspond to the land reclaimed since 1868 (Figure II.1.1), which confirms that the occurrence of liquefaction is due to both in-situ conditions and applied horizontal acceleration. Loose riverbank sand deposits liquefied causing damages to levees; liquefaction was also observed in inland areas characterised by high ground water levels (Shibata et al., 1996).
Reclamation islands, constructed to expand the Kobe port, were severely damaged even if they were made of gravelly soils, apparently no liquefiable. Indeed, Figure II.1.2 shows the situation of ejected sand and gravel in the southern side of Rokko Island, where it is possible to notice that also cobbles with a grain size larger than 10 cm were ejected.

A wide description of a reclaimed island, Port Island, will be reported in chapter V, with a focus on the vertical seismic array installed at that site.

Figure II.1.2 – Liquefaction of gravel at Rokko Island (Shibata et al., 1996)
River dikes built to control flood also suffered extensive damage consisting of cracking, settlement, lateral spreading and slumping. As shown in Table II.1.1, the damage was observed on six rivers for a total of about 9 km of levees damaged. At most of the sites, sand boils were observed on the ground surface near the dikes (Matsuo, 1996).

Table II.1.1 – Summary of damage to river dykes (Matsuo, 1996)

<table>
<thead>
<tr>
<th>River</th>
<th>Location (Prefecture)</th>
<th>Number of damage sites</th>
<th>Total length of damaged sites (in m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yodo-gawa</td>
<td>Osaka</td>
<td>18</td>
<td>6590</td>
</tr>
<tr>
<td>Kanzaki-gawa</td>
<td>Osaka</td>
<td>1</td>
<td>30</td>
</tr>
<tr>
<td>Ina-gawa</td>
<td>Hyogo</td>
<td>3</td>
<td>610</td>
</tr>
<tr>
<td>Mo-kawa</td>
<td>Hyogo</td>
<td>5</td>
<td>680</td>
</tr>
<tr>
<td>Kako-gawa</td>
<td>Hyogo</td>
<td>3</td>
<td>1180</td>
</tr>
<tr>
<td>Yura-kawa</td>
<td>Kyoto</td>
<td>2</td>
<td>200</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>32</strong></td>
<td><strong>9290</strong></td>
</tr>
</tbody>
</table>

The most severely damaged were Torishima and Nishijima dikes, located along the left and right side, respectively, of the Yodo river, which flows through Osaka city (Figure II.1.1).

Figure II.1.3 reports the cross section of Torishima dyke, before and after the earthquake. The dyke was designed to protect land against the high sea tide of 5.2 m above the mean sea level. The core of the structure was a soil embankment and the river side surface was protected by a concrete parapet wall, 8.1 m height from the sea level.

After the seismic event, the parapet at the top of the levees tilted and settled, sliding about 8 m into the river. The embankment also cracked severely and settled up to 3 m. Sand boils along the fissures were observed on the ground surface along the dike, suggesting that the damage was caused by liquefaction of the loose sand layer under the embankment.

![Figure II.1.3 – Cross section of the Torishima dyke (Shibata et al., 1996)](image-url)
Figure II.1.4 shows the cross section of the failed Nishijima dyke together with the boring data and the soil profile. The embankment has a structure similar to that of Torishima dyke, but in this case no damage to the parapet was observed. Embankment slope failure occurred in the rear slope and cracks were about 20 to 150 cm wide at the crest. Sand boils were observed near the toe, which displaced about 1 m. Also in this case, the liquefaction of the Holocene sand deposit (As2) under the dyke seems to have induced the embankment slope failure (Figure II.1.4).
Liquefaction induced dyke failures also during the 1993 Kushiro-oki earthquake (M= 7.8) which struck the Hokkaido region in the northern of Japan (Figure II.1.5). Along the Kushiro river, very close to the epicenter, dikes were damaged over a length of 10124 km (Sasaki, 2009).

It was found that the failure was induced by liquefaction occurred in the body of the dyke. Indeed, construction of the dyke, using sand fill directly resting on highly compressible peat deposit, resulted in consolidation settlement (2 – 3 m) under the weight of the embankment. This settlement brought the lower part of the sand fill below the water table that existed at the time of the earthquake. The saturated bottom part of the dyke liquefied during the earthquake triggering the failure.
More recently, a long seismic sequence devastated much of the city of Christchurch (New Zealand) and surrounding areas from September 2010 to February 2011 (Figure II.1.7).

Liquefaction during the 4th September 2010 Darfield event (Mw 7.1) affected approximately 10% of the area of Christchurch, while the 22nd February 2011 Christchurch event (Mw 6.2) affected over 50% of the developed land (Figure II.1.8). Including these two events, there were a total of seven events with moment magnitude greater than or equal to 5.5 between the 4th September 2010 and 23rd December 2011 that caused varying degrees of liquefaction in and around Christchurch. Moreover, the Mw 4.7 event that occurred on the 26th December 2010 and Mw 5.0 event of the 16th April 2011 triggered isolated cases of liquefaction (Markham et al., 2014).

The damaged area is located inside the Canterbury plain, formed by overlapping alluvial fans produced by glacier-fed rivers from the mountain range in the south of island (Figure II.1.9a). Most of Christchurch was built on floodplains and swamps behind a series of barrier dunes composed of fine-grained beach/dune sand, and lagoons. The Waimakariri river regularly flooded Christchurch before levees construction and river realignment, shortly after the city was established in 1850 (Green at al., 2014). Of particular relevance to liquefaction susceptibility in Christchurch and its environs are the locations of abandoned paleo-channels.
of the rivers. These areas were filled with young loose sandy sediments, which result highly susceptible to liquefaction in combination with shallow groundwater levels (from 1 m to 5 m below ground surface),

One of the worst hit locations was the small town of Kaiapoi north of Christchurch, established on the banks of the Kaiapoi river, a branch of the Waimakariri river (Figure II.1.9a,b). This latter was extensively modified both by natural and human processes, consequently many areas in and around the town were once former river channels (Wotherspoon et al., 2012). Using historical accounts and maps of the region, Wotherspoon et al. (2012) identified areas of land reclamation and old channels that were cut off from the river since the beginning of European settlement in the 1850s. Significant liquefaction damage were observed in these areas after the Darfield event. Significant lateral spreads and sand boils developed in areas of reclamation along the current river path, with fissures up to 2 m deep and 1 m wide (Figure II.1.9c,d). The damaged river banks caused settlement and tilting of structures in the surroundings area, with several residential structures severely damaged. Moreover, in all former channel areas, underground services, roads and railways were severely impacted as a result of cracking and ground movements (Wotherspoon et al., 2012).
Similar evidences have been characterized the seismic sequence that occurred in the Emilia Romagna region in May – June 2012. This sequence of events consisted of two main shocks occurred on May 20 ($M_W=6.1$) and May 29 ($M_W=5.9$), five more shocks with $M_L>5$ and about 2500 smaller earthquakes (Figure II.1.10).

The hit area is located in the south of the Po plain, in the foreland basin of two mountain range constituted by the Alps and the northern Appennine. A complex system of tectonic structures is buried under a thick sedimentary fills, so that the thrusts are generally buried (Fioravante et al., 2013). The main tectonic structure is a buried ridge known as Ferrara folds, which reaches its maximum height, about 120 m below the ground surface, near the city of Ferrara.

The subsoil is characterized by alluvial deposits of different depositional environment, which consists of alternated layers of silty-clayey deposits and sandy soils of channel and levees. Along ancient rivers banks many small villages were establishes in order to safeguard buildings from floods (Figure II.1.11).
Widespread liquefaction phenomena mainly involved the old river bed deposits and levees of rivers and channels (Figure II.1.11). Significant liquefaction effects particularly interested two villages constructed above the abandoned channel of the Reno river, San Carlo e Mirabello. After the 20 May earthquake, typical soil liquefaction evidences were observed: sand boils, surface ruptures and extensional fissures (Figure II.1.12b-e). Wide areas were covered of erupted material, whose thickness was more than 30 cm in many cases. In some locations, such as Sant’Agostino, bulges and fractures were observed at the bottom of artificial channels due to the sand eruption (Figure II.1.12d).
Bertolini and Fioroni (2012) inventoried more than 700 superficial effects thanks to aerial photographs. Seventy percent of the observed effects occurred within town and village boundaries, while the remaining 30% was observed in open agricultural fields and was characterized almost exclusively by sand volcanoes and eruption of wet sands from ground fissures. Figure II.1.13 shows long tension cracks (up to 1 m width and 2 m depth) and eruptions (yellow spots) from wells, cracks and foundations inside the town of San Carlo, which help to understand the extension of the mapped surficial effects.

Moreover, the Agenzia Regionale Prevenzione e Ambiente (ARPA) of the Emilia Romagna region manages an automatic monitoring network composed of 40 stations for monitoring the water levels and the temperatures inside wells, at a rate of one measure per hour. (Marcaccio
and Martinelli, 2012). In concomitance with the May-June 2012 seismic sequence, sudden water uprising phenomena were recorded by some automatic station located in the provinces of Modena and Ferrara (Figure II.1.14).

Water levels increased up to 1.5 m after the mainshock of the 20th May, and then decreased in the days following the earthquake. After the second mainshock of the 29th May, a second increase of the water levels was observed, even if less prominent respect to the first one. These data have been considered useful to understand the relationships between seismic events and the local groundwaters.

*Figure II.1.14 - Monitoring stations location (a) and water-level changes in concomitance with some significant seismic events (b) (Marcaccio and Martinelli, 2012)*
II.2. **Limits of simplified approaches and total stress analysis**

The engineering understanding of the seismic behaviour of soil structures subjected to extensive and damaging liquefaction can be widely improved by taking advantage of the experiences provided by the above mentioned case histories.

A reliable computation of damages due to liquefaction, starts with a correct evaluation of the seismic response of sites potentially prone to develop significant pore water pressures during earthquake shaking. This evaluation in turn can be achieved by adopting approaches characterized by an increasing level of complexity, as shown in table Table II.2.1 where a hierarchical classification of the possible approaches is reported (Silvestri and d’Onofrio, 2014).

The methods of analysis for the liquefaction assessment are listed in table II.2.1 as an increasing function of the refinement and degree of detail of the results, which in turn are proportional to the effort spent in terms of experimental and numerical resources as well as in term of complexity of the constitutive model adopted.

<table>
<thead>
<tr>
<th>Analysis method</th>
<th>Reference input motion</th>
<th>Constitutive model</th>
<th>Investigation and tests</th>
<th>Typical output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Screening criteria</td>
<td>Magnitude and distance</td>
<td>-</td>
<td>-</td>
<td>Liquefaction yes/no</td>
</tr>
<tr>
<td>Semi-empirical methods</td>
<td>Maximum acceleration $a_{\text{max}}$ Magnitude</td>
<td>Rigid - plastic</td>
<td>Ordinary investigation in-situ and laboratory Shear wave velocity measures, VS</td>
<td>Safety factor</td>
</tr>
<tr>
<td>Simplified dynamic</td>
<td>Accelerograms $a(t)$</td>
<td>Equivalent linear/ non-linear</td>
<td>As above plus cyclic and dynamic laboratory tests</td>
<td>Accelerogram Displacement Total stress Strain Safety factor profile</td>
</tr>
<tr>
<td>Advanced dynamic</td>
<td>Polyphase medium approximate coupled</td>
<td>Polyphase medium Stricly coupled</td>
<td>As above plus Excess pore water pressure Effective stress</td>
<td></td>
</tr>
</tbody>
</table>

As regard to the semi-empirical methods, there are basically two different approaches for evaluating the liquefaction potential of saturated soils: (1) evaluation based on the performance of soil deposits in previous earthquakes and extrapolation of such data to new conditions; and (2) evaluation based on comparison of the stresses induced in the ground by
the earthquake with the stress conditions causing liquefaction of the soils as measured by laboratory tests.

In both cases, the safety against liquefaction to a generic depth is reviewed in terms of safety factor defined as the ratio between the available resistance to liquefaction (state of critical effort associated with liquefaction condition or occurrence of large plastic deformation) and stress induced by the seismic action. Both stress parameters are normalized with respect to the vertical effective stress acting at the considered depth, so as to define a cyclic resistance ratio, CRR, and a cyclic stress ratio, CSR. If the first semi-empirical method is adopted the cyclic resistance ratio, CRR, is estimated using the results of in-situ tests (CPT penetration resistance, SPT blow count, or shear wave velocity, Vs); otherwise a cyclic resistance curve obtained from laboratory tests (simple shear test, torsional shear test, cyclic triaxial test), provides the proper CRR.

The definition of seismic demand has inherent the concept of converting the irregular seismic action into an equivalent number of cycles of uniform shear stress, as introduced by Seed and Idriss (1971). Indeed, the cyclic stress ratio, CSR is computed as

$$ CSR = \frac{\tau_{eq}}{\sigma'_{v0}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_{v}} r_d $$

Where $a_{max}$ is the peak acceleration at the ground level for the design earthquake; $g$ is the gravity acceleration; $\sigma_v$ and $\sigma'_{v}$ are respectively the total vertical stress and the vertical effective stress to the considered depth; $r_d$ is a reductive coefficient of the seismic action, taking into account the deformability of the soil. In the described approaches, the cyclic stress ratio, CSR, is the normalized amplitude, $\tau_{eq}$, of an equivalent uniform history of shear stress, in which the earthquake is converted. The uniform load is defined equivalent because, in principle, should produce the same pore pressure build-up expected at the site by the irregular seismic action.

In order to apply the semi empirical approach based on the cyclic resistance measured from laboratory tests it is necessary to define also the number of cycles, $N_{eq}$, of the uniform equivalent load. A wide range of conversion procedures is available in literature (Seed et al., 1975; Annaki and Lee, 1977; Biondi, 2002; Green and Terri, 2005). These procedures, however, are rather complex and their results strictly depend on the adopted conversion curve and on the techniques for choosing and counting the stress cycles that significantly affect the pore pressure build-up (Biondi et al., 2012).

As a consequence, a number of empirical relationships for the assessment of $N_{eq}$, bypassing the conversion procedure, were developed over the years. These models relate $N_{eq}$ to the magnitude of selected design earthquake, to site-source distance and, finally, to parameters
describing the energy and frequency content and the significant duration of the imposed ground motion (Figure II.2.1).

![Graph showing the number of equivalent uniform cycles versus earthquake magnitude, $M_w$ (Idriss and Boulanger, 2004)](image)

Figure II.2.1 - Number of equivalent uniform cycles versus earthquake magnitude, $M_w$ (Idriss and Boulanger, 2004)

In the case of the simplified dynamic approach, a safety factor can still be computed along a vertical soil profile but, in this last case, the seismic response of the soil column is taken into account. The stress-strain behavior is usually modeled as linear equivalent visco-elastic, independent from the failure condition typically modeled with a rigid-plastic approach. The seismic demand is expressed as

$$CSR = 0.65 \frac{\tau_{max}}{\sigma'_{vq}}$$  \hspace{1cm} (II.2.2)

where $\tau_{max}$ is the peak value of the shear stress history computed by a seismic response analysis in total stresses (Figure II.2.2).

![Graph showing the definition of equivalent shear stress](image)

Figure II.2.2 - Definition of equivalent shear stress (mod. after Seed and Idriss, 1971)
Finally, the advanced dynamic methods allow the predictions of the dynamic behavior of the soil volume in terms of stress-strain magnitudes. Within this level of approach the analysis methods are classifiable based on:
- The different method adopted to compute the stresses induced by the earthquake in the soil, typically related to the way of treating the non-linearity of the stress-strain relationship (linear analysis, equivalent linear, non-linear);
- The different constitutive model adopted: e.g. Hypoelastic models (such as the hyperbolic), elasto-plastic with isotropic hardening (e.g. Cam Clay type) or kinematic, elasto-visco-plastic, and so on;
- The different geometric discretization adoptable, which leads to the finite element method (FEM), the finite difference (FDM), or distinct elements (DEM).

It is worth noting that only coupled non linear analyses allow the prediction of effects related to pore pressure buildup; on the other hand, such rigorous approach require a high number of constitutive parameters to be experimentally measured by non conventional apparatuses; furthermore their results are strongly influenced by the adopted earthquake scenario. On the other hand, it is possible to evaluate the excess pore pressure induced by the earthquake at every depth and time instantaneous within the analysed domain.

The advanced effective stress analyses can be further divided into two categories based on the way the interaction between the two phases is modelled: (1) approximate methods, in which the interaction is expressed through simple empirical correlations; and (2) strictly coupled methods, also called rigorous methods in effective stresses, in which the interaction between phases is modelled through a system of differential equations describing the theory of consolidation generalized to the dynamic field (Sica, 2000).

The rigorous analyses are generally carried out to solve two-dimensional problems (rarely three-dimensional) using numerical codes based on finite elements or finite differences and constitutive models based on the incremental plasticity theory. For this reason, at the moment, their application is generally limited to structures with strategic relevance (e.g. earth dams) which can justify the use of such methods of analysis and sophisticated investigations.

Also 1D effective stress analyses are included in the category of the advanced dynamic analyses. Even if they allow modeling simple geometry several one-dimensional codes adopt advanced constitutive models, e.g. Cyclic 1D (Elgamal et al., 2006). In the following a focus on the existing 1D codes working in effective stresses will be presented, highlighting limits and peculiarities.
II.2.1. Existing models for 1D effective stress analysis

Table II.2.2 reports some of the most popular 1D codes performing effective stress analyses. Moreover, Table II.2.3 reports several commercial two-dimensional codes that can be adopted also for one-dimensional analyses.

As described in the previous paragraph, the pore water pressure variation during seismic loading can be modeled in two different ways: by adopting (1) semi-empirical pore water pressure, pwp generation relationships used in combination with models working in total stress; and (2) by effective-stress models whereby the pwp variation is computed as the difference between total stresses (or loads) and effective stresses, computed through an advanced constitutive model.

Given the enhanced capability of actual hardware resources, the number of codes adopting a fully coupled formulation to model pore pressure buildup is rapidly increasing, as shown in Table II.2.2. Nevertheless, the evaluation of material parameters for these models requires significant expertise and detailed site-specific soil properties (Matasovic and Hashash, 2012), often not available.

Table II.2.2 - 1D codes for effective stress analyses

<table>
<thead>
<tr>
<th>Dynamic analysis</th>
<th>Code</th>
<th>Reference</th>
<th>Constitutive model</th>
<th>Pwp generation model</th>
<th>Pwp dissipation /redistribution model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SUMDES</td>
<td>Li et al. (1992)</td>
<td>Bounding surface hypoplasticity model</td>
<td>Bounding surface hypoplasticity model</td>
<td>Biot theory</td>
</tr>
</tbody>
</table>
Examples of plasticity-based constitutive laws include Prevost (1989), Beaty and Byrne (1998), and Elgamal et al. (2004). These advanced constitutive models are capable of simulating complex soil behavior under a variety of loading conditions. Key elements of these models include yield surfaces, flow rules, and hardening (or softening) laws.

As stated in the Introduction chapter, the aim of the thesis is to develop a simplified pore water pressure model to be implemented into a computer code, aimed at performing 1D effective stress analysis. The attention will then be concentrated on codes that simulate the pore pressure buildup by adopting empirical relationship, based on the results of cyclic undrained laboratory tests, and allow to carry out an approximate coupled effective stress analyses.

Table II.2.3 – Most popular 2D codes for effective stress analyses

<table>
<thead>
<tr>
<th>Dynamic analysis</th>
<th>Code</th>
<th>Reference</th>
<th>Constitutive model</th>
<th>Pwp model</th>
</tr>
</thead>
<tbody>
<tr>
<td>coupled</td>
<td>FLAC 2D</td>
<td>ITASCA (2005)</td>
<td>Sigmoidal/Hardin and Drenovich (1972)</td>
<td>Martin et al. (1975)</td>
</tr>
<tr>
<td>Strictly coupled</td>
<td>GEFDYN</td>
<td>Aubry and Modaressi (1996)</td>
<td>Multiyield surface plasticity ECP model (Aubry et al., 1982)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FLIP</td>
<td>Iai et al. (1992)</td>
<td>Strain space multiple mechanism model</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PLAXIS 2D</td>
<td>Brinkgreve et al. (2013)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In this class of models, pore water pressure buildup is computed based on semi-empirical models. At the beginning of shaking (i.e., at time t= 0), the stress-strain behavior is modeled as in total stress conditions because pwp is zero. As shaking progresses, pwp is generated and cyclic degradation (of clay microstructure) starts. Subsequently, the effects of pwp generation and, in some models, of cyclic degradation are included by modeling the degradation of soil strength and stiffness.

Pore pressure generation models can generally be categorized into stress-based, strain-based, and energy-based models which can be applied in one-, two-, and three-dimensional analyses. Whereas initial models were primarily based on the results of cyclic stress-controlled tests, further studies showed a better correlation between pwp buildup and the shear strain level (Dobry et al., 1985) or the energy dissipated within the soil deposit (Green et al. 2000).
An extensive description of the pore water pressure models is reported in Biondi et al. (2010). A number of pwp generation models have been developed, starting with Martin and Seed (1978) and Martin et al. (1975). The Martin et al. (1975) pwp generation model was used in DESRA-2 (Lee and Finn 1978). An other example of the semi-empirical pwp model for saturated sand is the Dobry et al. (1985) model. This model was based on the results of strain-controlled cyclic direct simple shear and cyclic triaxial tests. The model was later modified by Vucetic (1986) and further by Matasovic (1993), that accurately modeled pwp induced degradation of shear modulus and shear stress. The Vucetic (1986) modification of this pwp model has been successfully incorporated in DESRAMOD (Vucetic 1986), and the Matasovic (1993) modification has been incorporated in D-MOD (Matasovic and Vucetic 1995), D-MOD2000 (Matasovic and Ordonez 2007), and DEEPSOIL (Hashash 2009).

The major drawback of all the described pwp generation models is the requirement of an equivalent number of cycles to represent earthquake shaking. Even if a great number of empirical relationships have been developed to define the $N_{eq}$, they all cannot take into account for the complexity of the seismic action. Also for this reason Polito et al. (2008) introduced an energy-based model (GMP model) for the generation of pwp based on a large number of laboratory tests, which does not require the development of an equivalent number of cycles. This model has been implemented in DEEPSOIL (Hashash 2009) combined with the degradation index framework introduced by Matasovic and Vucetic (1993). With the exception of the modified Dobry et al. (1985) model, as implemented in D-MOD2000, there is limited information to guide the user in selecting the appropriate pwp model parameters (Matasovic and Hashash, 2012).

II.3. References


Booker J.R., Rahman M.S., Seed H.B. (1976). GADFLEA - A computer program for the analysis of pore pressure generation and dissipation during cyclic or earthquake loading. Earthquake Engineering center, University of California, California, USA.


III. DEVELOPMENT OF A SIMPLIFIED MODEL

In this chapter, the original and updated formulation of a simplified pore water pressure build up model is described. The model is based on the results of cyclic laboratory tests; the calibration of the model parameters has been analysed in details considering both simulated and experimental data. A pore pressure dissipation model based on monodimensional consolidation theory has been also integrated in the formulation in order to simulate both generation and dissipation of excess pore water pressure.

III.1. Formulation of cyclic strength and pwp buildup

The proposed model is formulated within the framework of the endochronic theory. This Theory is based on the concept of intrinsic time, firstly introduced in the early 1950’s. Valanis (1971) applied this concept to describe the inelastic, viscoplastic behaviour of metals. The theory is called endochronic (endo: within, chronic: related with time) because stress is defined with respect to an intrinsic time scale, which is a material property and depends on the deformation history.

Bažant and Krizek (1976) developed an endochronic constitutive law for soils under quasi-static and cyclic loads. This model is based on the postulate that exists a series of internal state variables and intrinsic material functions that express, on a macroscopic level, the microstructural changes that take place in the soils during load application and their build up along a stress or strain path.
Finn and Bathia (1982) applied endochronic theory to develop a direct relationship between the pore pressure buildup in a cyclic laboratory test and the dynamic response parameters of the sand-water system. In this case, the endochronic model can be considered as ‘black-box’ approach that allows to express important parameters of dynamic soil response such as pore pressure build up as monotonically increasing functions of suitable variables. It does not attempt to model the actual physical process involved and therefore can be considered as useful prospect for eliminating the need to measure complex mechanical parameters of soil behaviour in undrained cyclic conditions.

Bathia (1982) verified the endochronic formulation on both strain-controlled and stress-controlled simple shear tests carried out on Ottawa sand, Crystal Silica sand and Toyoura sand. Since in cyclic tests stress-controlled conditions are still the most widely used laboratory procedure for evaluating the liquefaction potential, only for this type of test the analytical formulation of the model will be described in the following.

III.1.1. Analytical formulation

Pore water pressure ratio, \( r_u = u / \sigma'_0 \), is a function of the shear stress ratio, SR, and the number of cycles, \( N \), as clearly shown in Figure III.1.1a where data measured by Silver and Park (1976) on Crystal Silica sand (Dr=60%) are reported.

\[ r_u = f(SR, N) \]  

(Figure III.1.1 – Porewater pressure ratio vs number of cycles related to stress-controlled cyclic triaxial tests on Crystal Silica sand (mod. after Silver and Park, 1976))
Starting from this evidence Finn and Bathia (1982), proposed an alternative to N in describing the stress history applied to the sample: the length, $\eta$, of the stress path corresponding to N cycles of SR. The variable, $\eta$, is a monotonically increasing and continuous variable since, for definition, an increment in the length of the stress path equates an increment of shear stress, as suggested by Ivšić (2006)

$$d\eta = \frac{|d\tau|}{\sigma'_o} \tag{III.1.2}$$

Following this definition the stress path length corresponding to one cycle at a given shear stress level, SR, is four times SR thus the length of the stress path at the end of the Nth cycle of a stress controlled test will be

$$\eta = 4 \cdot N \cdot SR \tag{III.1.3}$$

Based on equation (III.1:3) the total length of the stress path was calculated for all the data of Figure III.1.1a and the pore water pressure ratio was then plotted as a function of $\eta$ in Figure III.1.1b.

The pore pressure ratio in a constant stress test may now be expressed as

$$ru = g(SR, \eta) \tag{III.1.4}$$

SR and $\eta$ are related each other through a further variable introduced by Finn and Bathia (1982), known as damage parameter, $\kappa$, which synthesizes all the parameters defining the stress history, including variable stress amplitudes and number of cycles. For definition, the damage parameter is a transformation of $\eta$, if a transformation $T$ exists so that

$$\kappa = T \cdot \eta \tag{III.1.5}$$

The parameter $\kappa$ is called damage parameter because the increase of stress path length induce pore pressure build up and weaken the resistance of sand.

The transformation $T$ should be a function of shear stress ratio, SR, expressing that at equal stress path length the damage increases as SR increases (Figure III.1.1c).

Porewater pressure ratio can be then expressed as function of the damage parameter, $\kappa$, as

$$ru = h(\kappa) \tag{III.1.6}$$

The equation III.1.6 represents the time history of excess pore pressure, $ru$, and expresses a strict one-to-one relationship between damage parameter and porewater pressure ratio.

Several authors proposed different relationships for the transformation $T$ and the function $h$ summarized in Table III.1.1.
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### Table

<table>
<thead>
<tr>
<th>Authors</th>
<th>Damage parameter $\kappa = T \cdot \eta$</th>
<th>Excess pore pressure ratio $ru = h(\kappa)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finn and Bathia (1982)</td>
<td>$\kappa = e^{SR} \cdot \eta$</td>
<td>$ru = \frac{F \kappa + G}{H \kappa + I} \cdot \kappa$</td>
</tr>
<tr>
<td>Ivšić (2006)</td>
<td>$\kappa = \frac{\eta}{SR} \cdot \left(\frac{SR}{SR_t} - 1\right)^a$</td>
<td>$ru = \begin{cases} 0.5 \cdot \kappa^{0.4} &amp; \text{for } \kappa \leq 2.5 \ (1.01 \cdot \kappa - 0.72) / \kappa &amp; \text{for } \kappa \geq 2.5 \end{cases}$ or $ru = \kappa / (1 + \kappa)$</td>
</tr>
<tr>
<td>Park et al. (2014)</td>
<td>$\kappa = (\eta / SR) \cdot \left(\frac{SR - SR_t}{SR - SR_t}\right)^a$</td>
<td>$ru = \frac{2}{\pi} \operatorname{arcsen} \left[ \left( \frac{\kappa}{\kappa_L} \right)^{1/3} \right]$</td>
</tr>
<tr>
<td>This study</td>
<td>$\kappa = (\eta / SR) \cdot \left(\frac{SR - SR_t}{SR - SR_t}\right)^a$</td>
<td>$ru = a \left( \frac{\kappa}{\kappa_L} \right)^b + c \left( \frac{\kappa}{\kappa_L} \right)^4$</td>
</tr>
</tbody>
</table>

#### III.1.1. Damage parameter and cyclic strength

As regard to the transformation $T$, Finn and Bathia (1982) adopted an exponential relationship of the stress ratio, SR, where $\lambda$ is an experimentally determined constant. Ivšić (2006) proposed a different function for the transformation $T$, to better take into account for some peculiar features of the cyclic response of sand. As a matter of fact, laboratory tests show that the number of cycles required to produce liquefaction decreases with increasing amplitude of the cyclic stress ratio, SR, and with decreasing relative density. The relationship among relative density, cyclic stress ratio and number of cycles can be expressed graphically by the cyclic resistance curve (Figure III.1.1c).

The curve in Figure III.1.1c tends to an asymptotic value of SR as the number of cycles increases; it means that exists a limit value of cyclic stress ratio below which the failure will never be reached. This threshold stress ratio, $SR_t$, is the value below which excess pore pressure are not induced and it is analogous to the volumetric threshold strain, $\gamma_v$ (Vucetic, 1994) observed in strain controlled cyclic tests.

The damage parameter should take into account that for stress levels below $SR_t$ pore pressure build up does not occur. Ivšić (2006) computed the net stress path length, being the difference between the total length, $\eta$ and that characterized by a stress ratio $SR \leq SR_t$, $\eta' = \eta - \eta_v$.

(III.1.7)

In case of cyclic stress-controlled test, $\eta'$ can be expressed as follows:

$\eta' = 4 \cdot N \cdot SR - 4 \cdot N \cdot SR_t = 4 \cdot N (SR - SR_t)$  (III.1.8)
Based on the net length of stress path introduced by Ivšić (2006) it is possible to define the
damage parameter as:

\[ \kappa = \eta \cdot (SR - SR_t)^m, \quad m \geq 0 \]  \hspace{1cm} (III.1.9)

Where \( m \) is a positive exponent which affects both smaller and larger testing shear stress.

Rewriting equation (III.1.9) including equation (III.1.8) and (III.1.3), the damage parameter can
be written as follows \( \kappa = (\eta / SR) \cdot (SR - SR_t)^\alpha \)  \hspace{1cm} (III.1.10)

Where \( \alpha \) is \((1+m)\).

This formulation of the damage parameter is that adopted by Park et al. (2014) and also in this
study (Table III.1.1).

The above expression of the damage parameter differs from that proposed by Ivšić (2006) since
he suggested to divide \( k \) in the equation (III.1.10) by \( SR_\alpha \) with \( SR_\alpha \neq 0 \) (Table III.1.1). The damage
function proposed by Ivsic (2006) was disregarded in the present study since it give rise to some
numerical problems as \( SR_\alpha \) values approach zero.

The parameters \( SR_\alpha \) and \( \alpha \) define the cyclic strength of the soil and can be evaluated from the
cyclic resistance curve obtained from a stress-controlled cyclic test. In detail, the parameter \( \alpha \)
describes the steepness of the cyclic resistance curve, that is defined adopting the equation
proposed by Park and Ahn, 2013, rewriting as

\[ \left( \frac{SR - SR_t}{SR_t - SR_t} \right) = \left( \frac{N_r}{N} \right)^{\frac{1}{\alpha}} \]  \hspace{1cm} (III.1.11)

where \((N_r, SR_t)\) is a reference point on the cyclic resistance curve. A reference number of cycles
equal to 15 is considered in this study (that is the number of equivalent cycles corresponding to
a magnitude of 7.5 if the relationship proposed by Idriss (1999) is considered).

The physical meaning of the damage parameter can be easily deduced from the expression
(III.1.10), whereas it is the product of two quantities: the first, \( \eta / SR \), is proportional to the
number of cycles (see equation III.1.3), while the second, \( (SR - SR_t)^\alpha \), takes into account the
amplitude of the applied stress exceeding the threshold value. The damage parameter is
therefore an integral parameter that corresponds to the colored areas reported in Figure III.1.2a
for two different values of stress ratio. If the points \((SR,N)\) below to a cyclic resistance curve the
two areas are equivalent since they all corresponds to a unique value of damage parameter,
that can be computed substituting equation (III.1.11) in the equation (III.1.10)

\[ \kappa_\alpha = 4 \cdot N_r (SR_t - SR_t)^\alpha \]  \hspace{1cm} (III.1.12)

Moreover, due to the functional relationship between the damage parameter and the pore
pressure ratio (equation III.1.6), it is possible to design isodamage lines in the \( SR - N \) plane,
which corresponds to different value of pore pressure ratio (Figure III.1.2b). Substantially, during a stress-controlled cyclic test, the damage parameter increases with the number of cycles until reaching the liquefaction condition expressed by the cyclic resistance curve corresponding to $k = k_L$

Figure III.1.2 - Graphical representation of the damage parameter (a) and isodamage curves in the N-SR plane (b)

It is then possible to define the damage ratio, being the ratio between the damage, $k$, corresponding to a generic point along the stress path and that evaluated at liquefaction, $k_L$, as follows

$$\frac{k}{k_L} = \frac{4 \cdot N (SR - SR_l)^\alpha}{4 \cdot N_l (SR - SR_l)^\alpha}$$ \hfill (III.1.13)

A unique relationship exists between the number of cycles at liquefaction, $N_L$ and the stress ratio, $SR$, the cyclic resistance curve thus, expressing $SR$ as a function of $N_L$, through equation (III.1.11), it can be found that the damage ratio is equal to the ratio between the generic number of cycles and the number of cycles to liquefaction, as

$$\frac{k}{k_L} = \frac{4 \cdot N \cdot \frac{N_l}{N} (SR - SR_l)^\alpha}{4 \cdot N_l (SR - SR_l)^\alpha} = \frac{N}{N_L}$$ \hfill (III.1.14)

### III.1.1.2. Excess pore pressure ratio

The analytical functions modeling the pore pressure ratio dependency on the number of uniform cycles are all based on the best fitting of the experimental data (Table III.1.1). Finn and Bathia (1982) defined a relationship applying a non-linear least squares fitting method of the experimental data, where $F, G, H, I$ are constant of the soil. A similar expression was adopted by Ivšić (2006) in combination with an exponential function. Booker et al. (1976) developed an
empirical expression for \( r_u \) as a function of cycle ratio. This expression was subsequently proposed by Park et al. (2014) and is given as Equation (III.1.15) below:

\[
ru = \frac{2}{\pi} \arcsin \left[ \left( \frac{N}{N_L} \right)^{1/2} \beta \right]
\]

Equation (III.1.15) has two calibration parameters, \( N_L \) and \( \beta \), both of which can be determined from stress controlled cyclic triaxial tests. For a given soil, \( N_L \) is proportional to relative density and inversely proportional to the magnitude of loading; \( \beta \) is an empirical constant that depends on the soil type and test conditions. It has been the object of an extensive studies in the past decades. Booker et al. (1976) recommended \( \beta = 0.7 \) for clean sands, while Polito et al. (2008) correlate \( \beta \) to the fine content, relative density and shear stress ratio.

A wide review of the excess pore pressure relationships is reported in Biondi (2002) that distinguished empirical relations based on laboratory test measurements of excess pore pressure from that based on an energetic approach.

More recently, Khshila et al. (2015) modeled the relationship between excess pore pressure for uniform strain controlled tests and normalized number of cycles adopting a power function:

\[
ru = 0.9 \left( \frac{N}{N_L} \right)^r
\]

where \( r \) is a material parameter function of the strain levels, determined from laboratory tests.

In this study, a non linear least square curve fitting method has been used to determine the relationship between \( ru \) and \( N/N_L \). The following relationship has been proposed

\[
ru = a \left( \frac{N}{N_L} \right)^b + c \left( \frac{N}{N_L} \right)^d
\]

where \( a, b \) and \( c \) are curve-fitting parameters. A validation of the proposed curve will be described in the next paragraph.

### III.2. Calibration of PWP model parameters

The calibration of the PWP model parameters is aimed at:

- identifying soil properties that mainly affect the model parameters,
- evaluating the sensitivity of the model parameters to soil mechanical properties and
- defining some simple relationships between soil mechanical properties and model parameters.
More in details the parameters describing the damage ratio function as well as those characterizing the pore water ratio relationship should be calibrated. As regard to the parameters of the damage ratio function, their calibration requires the availability of data in terms of cyclic resistance curves from laboratory tests. Since it is very difficult to find a meaningful amount of data in literature, it was preliminary set up a general procedure to generate cyclic resistance curves (N,SR) from empirical relationship based on the corrected number of blow count from SPT test, (N1)_{60}.

The empirical relationship proposed by Idriss and Boulanger (2014) has been used since it includes data obtained from the most recent events (2010-2011 Canterbury earthquake sequence in New Zealand and the 2011 Tohoku earthquake in Japan). The calibration of the parameters of the damage ratio function has been then carried out on the set of generated cyclic resistance curves.

As regard to the proposed pore pressure ratio relationship the calibration of the parameters has been carried out based on a careful analysis of more than 200 experimental curves collected in literature.

### III.2.1. Generation of cyclic resistance curves from empirical relationships

Starting from Idriss and Boulanger (2014) triggering relationship a set of cyclic resistance curves has been generated, each corresponding to a given number of corrected blow counts, N1_{60}. To this aim it has been necessary to define the dependency of CRS on magnitude, through the magnitude scaling factor (MSF), as well as the relationship between magnitude and number of equivalent stress cycles. The Idriss and Boulanger (2014) relationship here reported allows to define the cyclic resistance ratio of the soil characterized by a given value of corrected blow counts at a reference earthquake of 7.5 magnitude. It is well known that the soil's CRR is dependent on the duration of shaking (which is expressed through an earthquake magnitude scaling factor, MSF) and the effective overburden stress (expressed through a K_{p} factor). The effect of the initial effective confining pressure is accounted by introducing a correction factor k_{o}, as in the following equation

\[ \text{CRR}_{M=7.5,0^c} = \exp \left( \frac{(N_{1})_{60}}{14.1} + \frac{(N_{2})_{60}}{126} - \frac{(N_{3})_{60}}{23.6} + \frac{(N_{4})_{60}}{25.4} - 2.8 \right) \]  

(III.2.1)

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\[ C_s = \frac{1}{18.9 - 2.55 \sqrt{\text{N}_{\text{diss}}} \leq 0.3 \]  

(III.2.3)

On the other end, to take into account for the dependency on shaking duration Boulanger and Idriss (2014) suggest an upgraded relationship for the magnitude scaling factor that take into account for the soil properties.

Fundamentally, the MSF is known to be physically affected by numerous factors, including the earthquake source characteristics, distance from the site to the source, soil profile characteristics, and depth in the soil profile (e.g., Liu et al. 2001, Green and Terri 2005), but the inclusion of all dependencies may not be warranted in practice. The MSF relationship proposed by B&I (2014) includes functional dependence on an index of the soil properties in addition to the earthquake magnitude M.

The magnitude scaling factor is based on the original weighting scheme introduced by Seed et al. (1975) for converting an irregular cyclic loading history to some equivalent number of uniform loading cycles. A key parameter in this procedure is the slope of the relationship between the CRR and number of uniform loading cycles to failure (N) obtained from laboratory tests. The CRR versus N relationship, over the range of N values important to earthquake loading, can often be reasonably approximated using a power law as,

\[ CRR = a N^{-b} \]  

(III.2.4)

where the fitting parameter b describe the slope of the relationship. For two individual stress cycles having magnitudes CSR\textsubscript{A} and CSR\textsubscript{B}, the relative number of cycles to cause failure at these two stress ratios can be obtained from the above relationship as,

\[ \frac{CSR_A}{CSR_B} = \left( \frac{N_B}{N_A} \right)^b \]  

(III.2.5)

The damage from one cycle of stress at CSR\textsubscript{B} would then be equivalent to the damage from X\textsubscript{A} cycles at CSR\textsubscript{A} if their numbers of cycles are an equal fraction of the number of cycles to failure at their respective CSRs. This leads to the expression,

\[ X_A = \left( \frac{CSR_B}{CSR_A} \right)^{\frac{1}{b}} \text{ (1 cycle at CRS}_B) \]  

(III.2.6)

This expression can be used to convert individual stress cycles into an equivalent number of uniform cycles at some reference stress level. The total number of equivalent uniform cycles at a reference stress level can then be determined for a wide range of earthquake ground motions, from which a representative number of equivalent uniform cycles can be obtained for a given earthquake magnitude (N\textsubscript{M}). The MSF can then be determined as,
Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

\[
MSF = \frac{CSR_M}{CSR_{M=7.5}} = \left(\frac{N_{M=7.5}}{N_M}\right)^b
\]

(III.2.7)

where \(N_M=7.5\) is the number of uniform cycles for \(M = 7.5\).

The effect of \(b\) on the MSF relationships is illustrated in Figure III.2.1.

![Figure III.2.1 - MSF relationships for clay and sand (Boulanger and Idriss 2007)](image)

The MSF relationship derived by Idriss (1999) for sands used \(b = 0.34\) based on cyclic test data by Yoshimi et al. (1984) for samples of Niigata sand obtained using frozen sampling techniques. The MSF relationship by Boulanger and Idriss (2007) for clays and plastic silts used \(b = 0.135\) for clays and plastic silts based on a compilation of cyclic testing data. The smaller \(b\) value for clays and plastic silts results in a much flatter MSF relationship than that obtained for sands, as shown in Figure III.2.1.

It is clear that the coefficient \(b\) influences the number of equivalent loading cycles and then MSF.

The effect of \(b\) on the number of equivalent uniform loading cycles is illustrated in Figure III.2.2 for earthquakes with \(M \approx 7.5\). This figure shows \(N_{M=7.5}\) computed for \(b\) values of 0.06 to 0.40 for a set of 42 motions at category D sites with peak ground accelerations (PGA) of 0.11 to 0.51 g during \(M = 7.3-7.6\) earthquakes. At any given value of \(b\), the \(N_{M=7.5}\) values varied by factors of 2 to 3. The geometric mean values for \(N_{M=7.5}\) were relatively constant at about 15 for \(b\) values of 0.2 to 0.5, and then increased rapidly for \(b\) values progressively smaller than about 0.2. Also shown on this figure is the trend in mean values obtained by Kishida and Tsai (2014) based on their analyses of more than 3500 ground motion recordings at category D sites from the PEER strong ground motion database.
The relationship between $N_{M=7.5}$ and the $b$ parameter has been described analytically in this study as

\[
N_{M=7.5} = \frac{18.61 \cdot b^2 - 3.71 \cdot b + 0.38}{b^2 - 0.091 \cdot b + 0.0021}
\]  

(III.2.8)

Which is the yellow curve represented in Figure III.2.2.

The maximum value of the magnitude scaling factor, $\text{MSF}_{\text{max}}$, is related to small magnitude earthquake and corresponds to the case where the motion is dominated by a single strong acceleration pulse. If this single pulse scenario is represented by $\frac{3}{4}$ of a cycle at its peak stress, then the equivalent number of uniform cycles at 65% of the peak stress would be (Idriss and Boulanger 2008),

\[
N_{\text{min}} = \left( \frac{1}{0.65} \right)^{1/b} \cdot \left( \frac{3}{4} \right) \text{cycles}
\]  

(III.2.9)

This minimum number of equivalent cycles can be used to compute the upper limit on MSF as

\[
\text{MSF}_{\text{max}} = \left( \frac{N_{M=7.5}}{N_{\text{min}}} \right)^b
\]  

(III.2.10)

The resulting relationship between $\text{MSF}_{\text{max}}$ and $b$ is shown in Figure III.2.3.
The MSF relationships used by Idriss and Boulanger (2008), as shown in Figure III.2.4, can be rewritten in a more general form as:

\[
MSF = 1 + (MSF_{\text{max}} - 1) \left( \exp \left( \frac{M}{4} \right) - \exp \left( \frac{-7.5}{4} \right) \right) \left( \exp \left( \frac{-5.25}{4} \right) - \exp \left( \frac{-7.5}{4} \right) \right)
\]  

(III.2.11)

where \( MSF_{\text{max}} \) is related to a minimum magnitude equal to 5.25. This form of the MSF relationship, coupled with a relationship between \( MSF_{\text{max}} \) and \( b \), allows generation of MSF curves for different values of \( b \), as shown in Figure III.2.4.

The last step in deriving a revised MSF for SPT-based liquefaction triggering procedures requires relating the parameter \( MSF_{\text{max}} \) (via the parameter \( b \)) to some combination of soil characteristics (e.g., FC, PI), SPT parameters (e.g., \( N_{160} \), \( N_{160cs} \)).

Boulanger and Idriss (2014) reviewed a meaningful amount of laboratory test data to compute \( MSF_{\text{max}} \) for different \( b \) values and plot them versus equivalent values \( N_{160cs} \). The equivalent
(N1)_{60cs} values in the figure were determined as the values for which the applicable triggering correlation produces the same CRR_{M=7.5,\sigma'\rightarrow1} value as obtained from the laboratory test data.

\[
\text{MSF}_{\text{max}} = 1.09 + \left( \frac{(N1)_{60cs}}{31.5} \right)^2 \leq 2.2
\]  
(III.2.15)

Substituting equations (III.2.8) and (III.2.9) in the (III.2.10), the MSF_{\text{max}} can be directly expressed as function of the soil parameter b

\[
\text{MSF}_{\text{max}} = 0.65 \left[ \frac{4}{3} \left( \frac{18.61 \cdot b^2 - 3.71 \cdot b + 0.38}{b^2 - 0.091 \cdot b + 0.0021} \right) \right]^b
\]  
(III.2.13)

Since the univocal relation between MSF_{\text{max}} and b, a defined b value can be associated to any MSF_{\text{max}} known from equation (III.2.12). Known b, N_{M=7.5} can be computed through equation (III.2.8), and the magnitude scaling factor can be computed for any number of cycles as, for definition

\[
\text{MSF} = \left( \frac{N_{M=7.5}}{N} \right)^b \leq \text{MSF}_{\text{max}}
\]  
(III.2.14)

Definitely, to each number of cycles, N, corresponds a MSF value, and then CSR_{M} value as

\[
\text{CSR}_{M} = \text{MSF} \cdot k_{\sigma} \cdot CRR_{M=7.5,\sigma'\rightarrow1}
\]  
(III.2.15)

As a result, different CSR-N curves have been obtained for different values of the effective confining pressure and corrected SPT blow count.
A range of (N1)\textsubscript{60} from 6 to 25 has been considered, because it is the range of SPT values that characterize the liquefiable soils. A wide range, from 50 to 800 kPa, has been assumed for the effective confining pressure, while, for the number of cycles, a range from 7 to 300 has been used for defining the cyclic resistance curve in order to have a good prediction of SR\textsubscript{R}.

**III.2.1.1. Calibration**

The calibration procedure has been divided in two steps: the first step is related to the calibration of SR\textsubscript{R} and the second part is related to the calibration of SR\textsubscript{T} and \( \alpha \).

SR\textsubscript{R} data points, which are related to 15 cycles, have been plotted as function of (N1)\textsubscript{60} and \( \sigma'_{vo} \) (Figure III.2.6). It is worth highlighting that SR\textsubscript{R} increases with (N1)\textsubscript{60} and decreases with \( \sigma'_{vo} \), so it could be interpreted with a polynomial function of (N1)\textsubscript{60}, where the coefficients \( x_1, x_2, x_3, x_4 \) and \( x_5 \) are ruled by \( \sigma'_{vo} \) with a logarithmic function. Finally, the following relationship has been obtained

\[
SR\textsubscript{R} = x_1 \cdot (N1)\textsubscript{60}^4 + x_2 \cdot (N1)\textsubscript{60}^3 + x_3 \cdot (N1)\textsubscript{60}^2 + x_4 \cdot (N1)\textsubscript{60} + x_5
\]

where

\[
x_i = x_{i.1} \cdot \ln \left( \frac{\sigma'_{vo}}{p'_0} \right) + x_{i.2} \quad \text{for } i=1, 2, 3, 4, 5
\]

with \( p'_0 \) equal to the atmospheric pressure. The parameters of the expression (III.2.17) have been clearly defined through a non linear regression procedure and are reported in Table III.2.1. The predicted curves are reported also in Figure III.2.6.

![Figure III.2.6 - Reference shear stress ratio, SR\textsubscript{R} and predicted values vs the corrected number of blow count, (N1)\textsubscript{60} and for different effective confining pressure](image-url)
As regard to the calibration of the parameter \( \alpha \), governing the steepness of the cyclic resistance curve, Idriss and Boulanger (2014) relates the slope of the curve to the corrected number of blow count, so the parameter \( \alpha \) is ruled by \( (N1)_{60} \). Essentially a linear dependence can be observed in Figure III.2.7a, but a cubic function is associated with a R-squared equal to unity

\[
\alpha = x_6 \cdot (N1)_{60}^3 + x_7 \cdot (N1)_{60}^2 + x_8 \cdot (N1)_{60} + x_9
\]

(III.2.18)

Finally the threshold shear stress ratio, \( SR_t \), was defined as the shear stress ratio corresponding to one million of cycles and plotted in Figure III.2.7b as a function of \( (N1)_{60} \) for different values of effective stress, \( \sigma'_{v0} \). Due to the small values of shear stress ratio, the effect of \( \sigma'_{v0} \) has been neglected and the following expression has been adopted for modelling \( SR_t \)

\[
SR_t = x_{10} \cdot (N1)_{60}^3 + x_{11} \cdot (N1)_{60}^2 + x_{12} \cdot (N1)_{60} + x_{13} \cdot (N1)_{60}^3 + x_{14}
\]

(III.2.19)

The coefficients have been computed using an optimization procedure based on the non-linear regression method and are reported in Table III.2.2.

**Figure III.2.7 - Plot \( \alpha \) and \( SR_t \) vs \( (N1)_{60} \) for different \( \sigma'_{v0} \) values**

**Table III.2.1 - Coefficients of the relationships (III.2.12)**

<table>
<thead>
<tr>
<th>( X_{6} )</th>
<th>( X_{7} )</th>
<th>( X_{8} )</th>
<th>( X_{9} )</th>
<th>( X_{10} )</th>
<th>( X_{11} )</th>
<th>( X_{12} )</th>
<th>( X_{13} )</th>
<th>( X_{14} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-3.07E-07</td>
<td>1.23E-06</td>
<td>1.35E-05</td>
<td>-5.46E-04</td>
<td>1.01E-03</td>
<td>1.27E-03</td>
<td>-1.94E-03</td>
<td>-8.45E-03</td>
<td>8.12E-02</td>
</tr>
</tbody>
</table>

**Table III.2.2 - Coefficients of the relationships (III.2.18) and (III.2.19)**

<table>
<thead>
<tr>
<th>( x_6 )</th>
<th>( x_7 )</th>
<th>( x_8 )</th>
<th>( x_9 )</th>
<th>( x_{10} )</th>
<th>( x_{11} )</th>
<th>( x_{12} )</th>
<th>( x_{13} )</th>
<th>( x_{14} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.50E-05</td>
<td>-2.25E-03</td>
<td>-7.92E-02</td>
<td>5.31E+00</td>
<td>-3.73E-08</td>
<td>3.67E-06</td>
<td>-1.16E-04</td>
<td>1.03E-03</td>
<td>1.08E-02</td>
</tr>
</tbody>
</table>
Finally, Figure III.2.8 reports the comparison between the generated cyclic resistance curves and the curves that come from the previous equations. It could be noticed a good match in the whole range of $(N1)_{60}$ and $\sigma'_{v0}$.

![Figure III.2.8 - Comparison between generated and predicted CRC](image)

III.2.2. Calibration of the pwp buildup parameters

Differently from the cyclic resistance curves, pore pressure ratio curves, $ru = N/N_L$, cannot be generated elaborating the in situ triggering relationships. Consequently, the only feasible way is that of considering experimental data reported in the literature. The collection of experimental data allows verifying the capability of the proposed model (Equation III.1.17) to reproduce the cyclic behaviour of soils. In addition, the database of experimental data can be used in the calibration of the model parameters in case of lack of experimental laboratory tests.

For this purpose, 213 experimental curves have been collected from scientific papers, related to soils coming from 19 different countries all over the world (Figure III.2.5). The experimental data refer to undisturbed or reconstituted specimens, subjected to cyclic stress-controlled laboratory tests, i.e. cyclic simple shear or cyclic triaxial tests. All the soil properties and any other useful information on the tests have been collected inside a database (e.g. grain size distribution, relative density, effective confining pressure and cyclic stress ratio...
(CSR) applied during the tests, etc). Table III.2.3 reports an esemplificative record of the database. Every curve is identified by an identification number (ID) in the database.

![World map with investigated soils](image)

**Figure III.2.9 – Origin of soil samples whose experimental data were collected (Spiedo, 2016)**

The experimental excess pore pressure curves have been digitalized and normalized in order to obtain \( ru = \frac{N}{N_L} \) curves. Where necessary, a normalization procedure has been applied considering that the liquefaction condition is reached at \( ru = 0.95 \). This assumption implies that a residual 5% of the initial effective stress characterizes the liquefied soil. This assumption in turn implies a soil residual strength that results particularly advantageous when the liquefaction condition should be introduced inside a computer code, since it guarantees numerical stability. This stress approach has been preferred to the strain approach, since it is compatible with the PWP model developed.

All the collected curves have been then processed carrying out a non-linear regression analysis in order to evaluate the parameters of the proposed pore pressure ratio relationship, here recalled

\[
ru = a \left( \frac{N}{N_L} \right)^b + c \left( \frac{N}{N_L} \right)^4
\]

(III.2.15)

Figure III.2.6 reports an example of the results of regression analysis on the experimental curve of the Yatesville sand (Polito, 1999). All the data collected in the database and the parameters of the regression analysis are reported in Appendix B.
Table III.2.3 – Example of record of the database

<table>
<thead>
<tr>
<th>ID</th>
<th>Soil</th>
<th>Specimen</th>
<th>Test</th>
<th>Grain size</th>
<th>( \sigma'_0 ) [kPa]</th>
<th>CSR</th>
<th>Dr [%]</th>
<th>PI</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>187</td>
<td>Yatesville sand</td>
<td>Reconstituted</td>
<td>CTX</td>
<td>100% sand</td>
<td>200</td>
<td>0.22</td>
<td>40</td>
<td></td>
<td>Polito (1999)</td>
</tr>
</tbody>
</table>

Figure III.2.10 – Experimental (Polito, 1999) and modelled pore pressure ratio curve

The quality of the fitting has been computed using a statistic index: the adjusted coefficient of determination. As known, the coefficient of determination, \( R^2 \), is a number that indicates the goodness of fitting; a value of 1 indicates that the regression model perfectly fits the data, while an \( R^2 \) of 0 indicates that the model does not fit the data at all. The adjusted \( R^2 \), \( R_{\text{adj}}^2 \), takes into account also the phenomenon that the \( R^2 \) automatically and spuriously increases when extra explanatory variables are added to the model. Analytically, \( R_{\text{adj}}^2 \) is defined as

\[
R_{\text{adj}}^2 = R^2 - \left(1 - R^2\right) \frac{p}{n - p - 1}
\]

(III.2.15)

where \( p \) is the total number of explanatory variables in the model, and \( n \) is the number of experimental points. In this way, it is possible compare the fitting goodness obtained using different models, independently from the number of parameters characterizing the model.

As an example, the regression analysis carried out on the experimental data of Figure III.2.6 is characterized by \( R_{\text{adj}}^2 \) equal to 0.998. Figure III.2.7 summarizes the results of the regression analyses in terms of goodness of fitting. The result highlights that the proposed relationship is applicable on the 95% of the experimental curves, and 77% of the fitting curves are characterized by \( R_{\text{adj}}^2 \) is greater than 0.98.
Chapter III – Development of a simplified model

Figure III.2.11 – Applicability of the pore pressure relationship (a) and quality of fitting (b)

The experimental curves that could not be fitted by the proposed relationship are characterized by high values of CSR, relative density or effective confining pressure. Moreover, the analysis of the experimental data allowed to identify the soil properties that mostly influence the evolution of the pore pressure buildup during an undrained cyclic test on liquefiable soil. An accurate analyses of the collected experimental curves is reported in Spiedo (2016), while a summary is here described.

The grain size distribution has an important role in defining the shape of the pore pressure ratio curve. In particular, it was observed that: (1) soils with high fine content exhibit a curve with a horizontal asymptote at the liquefaction; (2) sandy and gravelly soils show a sudden change in the pore pressure growth and an almost vertical asymptote at the liquefaction (run-away behavior); (3) it seems that a fine fraction (d<0.06mm) higher than 30% can be considered as a threshold value since a further increase of fine fraction does not change the shape of the curve.

III.3. Performance of the pwp buildup model on a soil element

The main advantage of damage parameter is that it can be easily implemented in a code working in time domain to perform coupled effective stress dynamic analyses. In fact, a recorded acceleration time history is a sequence of acceleration values sampled with a preset time interval and the general formulation of the damage parameter (equation III.1.10) can be specified for each time step.

III.3.1.1. Numerical formulation

The numerical formulation of the model can be easily described with reference to a monotonic load, such as in Figure III.3.1.
Applying the general formulation of equation (III.1.10), the damage parameter could be computed as

$$\kappa = \frac{SR}{SR} \left( SR - SR_t \right)^\alpha = \left( SR - SR_t \right)^\alpha$$  

(III.3.1)

Which substantially means that the damage parameter is a power function of the portion of shear stress ratio that exceeds the threshold value. It is also clear that different stress histories with the same final value of stress ratio cause the same final damage (Figure III.3.1).

For a regular cyclic history of given amplitude, SR, remembering equation (III.1.3), the damage parameter is proportional to the number of cycles, as shown it the following equation:

$$\kappa = \frac{4 \cdot N \cdot SR}{SR} \left( SR - SR_t \right)^\alpha = 4 \cdot N \cdot \left( SR - SR_t \right)^\alpha$$  

(III.3.2)

that can be rewritten as

$$\kappa = \sum_{i=1}^{N} \left( SR - SR_t \right)^\alpha$$  

(III.3.3)

Where $i$ is the number of quarter of cycles in the stress history. For a periodic history with variable amplitude as that reported in Figure III.3.2a, the damage parameter can be expressed as

$$\kappa = \sum_{i=1}^{N} \left( SR_{max,i} - SR_t \right)^\alpha$$  

(III.3.4)

Where $SR_{max,i}$ is the amplitude of the $i$-th half cycle of the time history. The latter can be applied to any loading pattern (Figure III.3.2b) and expressed as function of the time, $t$,

$$\kappa(t) = \kappa_0 + \Delta \kappa(t) = \sum_{i=1}^{n} \left( SR_{max,i} - SR_t \right)^\alpha + \Delta \kappa(t)$$  

(III.3.5)

Where $n$ is the number of monotonic stretches at the time $t$ with maximum stress ratio $SR_{max,i}$ and $\Delta \kappa$ is the increment of the damage parameter in the time step.
\[ \Delta \kappa(t) = \begin{cases} |SR_0 - SR| \text{ per } SR \geq SR_t \\ 0 \text{ per } SR < SR_t \end{cases} \] (III.3.6)

Where \( SR_0 \) is the initial shear stress ratio of the current stretch at the time \( t \). It is equal to \( SR_{\text{max},i} \) for decreasing stretches and \( SR_t \) for increasing stretches. The portion of the damage parameter \( k_0 \) takes into account the past stress history and it is updated every time a maximum or minimum value is attained in the time history.

Figure III.3.2 - Periodic history with variable amplitude (a) and general loading pattern with damage parameter at time instant \( t \) (b)

The model has been applied with reference to the periodic history of shear stresses shown in Figure III.3.3a. The damage parameter has been computed based on the time history of the shear stress for given values of \( \alpha \) and \( SR_t \). As it is expected the parameter \( k \) cumulates when the cyclic stress ratio exceeds the threshold value otherwise it remains constant.

The time history of excess pore water pressure, can be then calculated once the pore pressure ratio is expressed as a function of the damage parameter.
III.22 Ph.D. in Geotechnical Engineering – XXVIII curriculum

Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

III.3.22

III.3.1.2. Performance of the model on an ideal soil profile

An ideal profile of 25 m of saturated Crystal Silica sand has been considered for the application of the model. It is characterized by parabolic velocity profile as shown in figure III.3.4 while the non-linear and dissipative behavior of the sand has been characterized adopting the curves proposed by Seed and Idriss (1970).

A total stress dynamic analysis has been carried out using the one-dimensional code SCOSSA (Tropeano et al., 2016) The accelerometric time history recorded at Assisi station on 26th of September 1997 has been adopted as input signal (PGA=0.275 g) (Figure III.3.4e). The results of the analysis are reported in terms of profile of maximum shear stresses (Figure III.3.4c) and maximum shear stresses normalized with respect to the lithostatic effective vertical stresses, \( \sigma_0 \) (Figure III.3.4d). By knowing the induced time histories of shear stress at different depths (Figure III.3.5a-c).

Figure III.3.3 – Generation of excess pore pressure induced by a regular cyclic shear stress history
Chapter III – Development of a simplified model

Figure III.3.4 – Ideal column of a sandy soil (a) with relative velocity profile (b) and results of seismic response analysis expressed as maximum shear stress (c) and shear stress ratio (d) for the recorded input motion (e)

It has been then possible to apply the pwp model. As an example the pore pressure model has been applied at a depth of 3 and 5 m from the ground level, where the cyclic stress ratio is high enough and a phenomenon of liquefaction can be expected. The results are reported in figure III.3.5. It can be noted that at 5 m depth (Figure III.3.5a-b-c-d) excess pore pressure cumulates in the first 10 s of the stress history reaching a final value of pore pressure ratio slightly less than 0.8. On the other hand, at 3 m depth (Figure III.3.5e-f-g-h) liquefaction triggers after 6.69 s. The low value of the initial effective stress at 3 m depth determines a cyclic stress ratio greater than that calculated at 5 m depth. It should be pointed out that, once the condition of liquefaction (\( \kappa = \kappa_L \)) is reached, successive peaks of shear stress ratio (circled in blue in Figure III.3.5f) are ineffective in inducing further increase of excess pore pressure. In both simulations the results show that pore pressure generation is concentrated in the critical phase of seismic motion.
Figure III.3.5 – Time histories of the shear stresses (a-e), damage parameter (b-f) and excess pore water pressure (d-h); pore pressure ratio vs the normalized damage parameter (c-g) at 5 m (a-b-c-d) and at 3 m (e-f-g-h) from the ground surface.
III.4. Formulation of pore pressure dissipation

A MATLAB subroutine has been written to simulate the 1D consolidation process of a soil column. The subroutine is designed to be implemented together with the PWP model in the non-linear code SCOSA (carefully described in chapter IV) in order to simulate the dissipation of excess pore water pressure caused by earthquake shaking. The subroutine should have been written so as to be compatible with the code as well as with the physics of the phenomenon. A first step consisted in reproducing the consolidation of a homogenous soil profile, then the consolidation of a two layered soil profile and as well as that of a multi-layered system with different initial condition of excess pore water pressure were simulated. Finally, the results of the MATLAB routine were compared with theoretical solutions of the one-dimensional problem and with other published solutions in order to test the rightness and reliability of the routine.

III.4.1. Numerical formulation of consolidation equation

The Terzaghi 1D consolidation equation, also known as diffusion equation, is written as follows

\[ \frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \]  

(III.4.1)

where the consolidation coefficient is indicated with \( c_v \), \( u \) is the pore water pressure, function of both the depth, \( z \), and the time, \( t \).

A homogeneous compressible soil layer can be divided into \( n \) sublayers, so that the depth at the bottom of the generic sublayer \( i \) is equals to \( i \cdot \Delta z \). The depth-time space can be divided as in Figure III.4.1, so that \( u_{i,t} \) represents the excess pore pressure at depth, \( z_i \), and time, \( t \).

---

![Figure III.4.1 – Identification of the variables in the depth – time space](image-url)
By applying the definition of derivative to the partial derivative, it can be written

$$\left( \frac{\partial u}{\partial t} \right)_i \approx \lim_{\Delta t \to 0} \frac{u_{i+\Delta t} - u_i}{\Delta t}$$  (III.4.2)

For small, finite value of $\Delta t$, the expression (III.4.2) can be approximate to

$$\left( \frac{\partial u}{\partial t} \right)_i \approx \frac{u_{i+\Delta t} - u_i}{\Delta t}$$  (III.4.3)

Were the smaller the time interval, $\Delta t$, the more accurate will be the solution. The time interval is here defined as forward difference, since it goes forward in time.

Performing same operations on the $u$ variable with respect to depth, $Z$, it can be written

$$\left( \frac{\partial u}{\partial z} \right)_i^F \approx \frac{u_{i+1,j} - u_{i,j}}{\Delta z}$$  (III.4.4)

$$\left( \frac{\partial u}{\partial z} \right)_i^B \approx \frac{u_{i,j} - u_{i-1,j}}{\Delta z}$$  (III.4.5)

Which are the forward and the backward difference in space, respectively.

Finally, the second derivative of the excess pore water pressure respect to the depth can be approximated as

$$\left( \frac{\partial^2 u}{\partial z^2} \right)_i \approx \left( \frac{\partial u}{\partial z} \right)_i^F - \left( \frac{\partial u}{\partial z} \right)_i^B = \frac{u_{i+1,j} - u_{i,j} - u_{i,j} - u_{i-1,j}}{\Delta z} \frac{\Delta z}{\Delta z} = \frac{u_{i-1} - 2u_i + u_{i+1}}{\Delta z^2}$$  (III.4.6)

The consolidation equation (III.4.1) can now be written in differential form, taking into account equation (III.4.3) and (III.4.6), as

$$\frac{u_{i+\Delta t} - u_i}{\Delta t} \approx c_v \cdot \left( u_{i-1} - 2u_i + u_{i+1} \right)$$  (III.4.7)

Or, rearranging as

$$u_{i+\Delta t} \approx u_i + \beta \cdot \left( u_{i-1} - 2u_i + u_{i+1} \right)$$  (III.4.8)

Which is the explicit finite difference form of the consolidation equation, where

$$\beta = c_v \frac{\Delta t}{\Delta z^2}$$  (III.4.9)

According to equation (III.4.8) the pore pressure at any time and depth can be calculated knowing the initial conditions at time $t$ ($u_{i-1}$, $u_i$ and $u_{i+1}$ in Figure III.4.1) and moving through the time by small time increments, $\Delta t$. 
The more $\Delta t$ and $\Delta z$ are small, the more the solution is accurate, but also more calculations will be necessary. The expression (III.4.8) is numerically stable for $\beta<0.5$ and in machine calculation $\beta$ is generally assumed equal or less than 0.1.

In case of a layered system, a boundary separates the above material, characterized by a definite value of permeability, $k_i$, and consolidation coefficient, $c_{vi}$, from that below, characterized by different values of permeability, $k_{i+1}$ and consolidation coefficient, $c_{vi+1}$ (Figure III.4.2). Since $\beta$ and $\Delta t$ must be the same for the whole profile, the discretization for each layer, $\Delta z_i$ and $\Delta z_{i+1}$, must be different.

For continuity, at the boundary between two soil layers the volume of water coming from one soil layer should be equal to that passing in the second layer, and for Darcy’s law it should be that

$$\frac{k_i}{\gamma_w} \left( \frac{\partial u}{\partial z} \right)_i = \frac{k_{i+1}}{\gamma_w} \left( \frac{\partial u}{\partial z} \right)_{i+1} \quad (III.4.10)$$

This introduces the problem of evaluating $\partial u/\partial z$ at the boundary between the two layers, which is a non-differentiable point.

If $u$ is named the excess pore pressure at the point (i-1) belonging to the first layer and at the point(i+1) belonging to the second layer and $v$ is named the pore pressure that it would be obtained at the same points if the material would be homogenous and characterized by the properties of layer 1 or two, $\partial u/\partial z$ at point $i$ can be approximated by the slope of one of the two lines in Figure III.4.2, as
\[
\frac{v_{i+1} - u_{i+1}}{2\Delta z_i} \quad \text{or} \quad \frac{u_{i+1} - v_{i+1}}{2\Delta z_{i+1}} \quad (\text{III.4.11})
\]

If the \( \frac{\partial u}{\partial z} \) is assumed to be equal to the average of these values, the equation (III.4.10) can be written as

\[
k_i \frac{v_{i+1} - u_{i+1}}{2\Delta z_i} = k_i \frac{u_{i+1} - v_{i+1}}{2\Delta z_{i+1}} \quad (\text{III.4.12})
\]

Remembering the expression of \( \beta \) (III.4.9) and defining

\[
\alpha = \frac{k}{\Delta z} \quad (\text{III.4.13})
\]

the equation (III.4.12) can be written as

\[
\alpha_i \cdot v_{i+1} + \alpha_j \cdot v_{i+1} = \alpha_i \cdot u_i - \alpha_i \cdot u_{i+1} \quad (\text{III.4.14})
\]

In the layer immediately above the boundary, for equation (III.4.7),

\[
\frac{\partial u}{\partial t} \approx \frac{c_{vi}}{\Delta z_i^2} \left( u_{i-1} - 2u_i + v_{i+1} \right) \quad (\text{III.4.15})
\]

and in the layer immediately below the boundary

\[
\frac{\partial u_{i+1}}{\partial t} \approx \frac{c_{vi+1}}{\Delta z_{i+1}} \left( v_{i-1} - 2u_i + u_{i+1} \right) \quad (\text{III.4.16})
\]

Solving equation (III.4.15) for \( v_{i+1} \) and equation (III.4.16) for \( v_{i-1} \), substituting into equation (III.4.14) and rearranging, it results

\[
(\beta_i \cdot \alpha_i + \beta_i \cdot \alpha_{i+1}) \cdot u_i \cdot \Delta t = 2\beta_i \cdot \beta_i \cdot (\alpha_i \cdot u_{i-1} - (\alpha_i + \alpha_{i+1}) \cdot u_i + \alpha_i \cdot u_{i+1}) \quad (\text{III.4.17})
\]

Finally, equation (III.4.17) can be rewritten as

\[
\left( \frac{\partial u}{\partial t} \right) \Delta t = A \cdot u_i - 2 \cdot B \cdot u_i + C \cdot u_{i+1} \quad (\text{III.4.18})
\]

where

\[
A = \frac{2\beta_i \cdot \beta_i}{\beta_i \cdot \alpha_i + \beta_i \cdot \alpha_{i+1}} \alpha_i \quad (\text{III.4.19})
\]

\[
B = \frac{2\beta_i \cdot \beta_i}{\beta_i \cdot \alpha_i + \beta_i \cdot \alpha_{i+1}} \left( \frac{\alpha_i + \alpha_{i+1}}{2} \right) \quad (\text{III.4.20})
\]

\[
C = \frac{2\beta_i \cdot \beta_i}{\beta_i \cdot \alpha_i + \beta_i \cdot \alpha_{i+1}} \alpha_{i+1} \quad (\text{III.4.21})
\]

It can be noticed that if \( c_{vi} = c_{vi+1} \) and \( k_i = k_{i+1} \) the coefficients become coincident and the original equation for a homogeneous soil layer is obtained.
Chapter III – Development of a simplified model

In the explicit finite difference form, the time derivative of excess pore water pressure is represented by a forward difference, so the equation (III.4.18) can be rewritten as

\[ u_{i,j,t+\Delta t} \approx u_{i,j} + \left( A u_{i-1} - 2 B u_i + C u_{i+1} \right) \]  

(III.4.22)

Which is the analog of equation (III.4.8) for a layered system.

### III.4.1.1. Description of two simple examples

Equation (III.4.8) and (III.4.22) have been used to solve the consolidation problem for two simple examples. The first is related to a homogeneous soil profile and has been useful to understand the explicit finite difference form of consolidation equation. The second is related to a system of two layers with different permeability and consolidation coefficients. In detail, the lower layer has a permeability four times higher than the upper layer.

In both cases, a constant initial distribution of excess pore water pressure has been assumed, while the boundary drainage conditions at the bottom are assumed locked in both cases.

The consolidation coefficient, \( c_v \), of each layer is computed as

\[ c_v = \frac{k \cdot E_{oed}}{\gamma_w} \]  

(III.4.23)

in function of the unit weight of the water, \( \gamma_w \), the permeability, \( k \), and the oedometric modulus, \( E_{oed} \), of the soil. For the elasticity theory, the oedometric modulus can be directly correlated to the stiffness of the soil, \( G_0 \), and the coefficient of Poisson, \( \nu \), as

\[ E_{oed} = \frac{2 \cdot G_0 \cdot (1 - \nu)}{(1 - 2 \nu)} \]  

(III.4.24)

The results of the two simulations, expressed as isochrones of excess pore water pressure, are reported in Figure III.4.3.
III.4.2. Validation of the dissipation subroutine

For the verification of the MATLAB subroutine, the problem of one-dimensional consolidation has been simulated with different initial excess pore pressure distributions and boundary conditions. All the considered closed form solutions are reported in Figure III.4.4 where the comparisons with the numerical solutions is also shown.

The comparison is shown in terms of excess pore water pressure normalized respect to the initial value, \( U/U_0 \), and in terms of time factor, \( T_v \), which is defined as

\[
T_v = \frac{c_v \cdot t}{H^2} \tag{III.4.25}
\]

where \( H \) is the drainage path.

In the first scheme, the isochrones are perfectly coincident with the theoretical solution, while in the other two schemes the isochrones are generally well-reproduced and no delay is present.
respect to the theoretical solution; sometimes differences are related only to the shape of the isochrones.

Figure III.4.4 - Comparison between the simulated isochrones (in brown) and the solutions of the 1D consolidation equation for several excess pore pressure initial distributions: rectangular (a), triangular (b) and inverse triangular (c) (mod. after Lang et al., 2011)

The most simple case, related to a rectangular initial distribution, has been performed also with the finite element code PLAXIS (version 8.2) for understanding the degree of approximation performed by one of the most popular commercial codes and, consequently, assessing if the error of subroutine solution is tolerable. Simulation has been performed according to "validation and verification" plaxis manual, version 8.2 (Brinkgreve, 2002). In the Plaxis model, the upper side is draining, while the closed consolidation boundary condition is applied on the other sides.
The finite element mesh is shown in the Figure III.4.5a. From the comparison between the simulation and the solution reported by Lang et al., 2011, it can be noticed that the overlapping decreases as the time factor increases (Figure III.4.5b) and Plaxis solution seems to be less reliable if compared to MATLAB results (Figure III.4.4a).

![Figure III.4.5 - Geometry and mesh adopted in Plaxis analysis (a) and comparison between the result (green isochrones) and the 1D solution (mod. after Lang et al., 2011)](image)

The consolidation of two layered system has been simulated for different values of soil permeability and compared with solutions available in literature (Figure III.4.6). In particular, an approximated solution of a two layered system is proposed by Luscher (1965), based on an electric analog model. As stated by the author, Luscher’s model drains slower than Terzaghi’s theoretical model, so the comparison with the MATLAB simulation is consistent with the expected results.

![Figure III.4.6 - Scheme of a two layered system (a) and MATLAB simulation compared to the Luscher solution (Luscher, 1965 mod. Das, 2007) (b)](image)
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III.5. References


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Idriss I.M., Boulanger R.W. (2014). CPT and SPT liquefaction triggering procedures. Report No UCD/GCM-14/01, University of California at Davis, California, USA.

Idriss I.M., Boulanger R.W. (2014). CPT and SPT liquefaction triggering procedures. Report No UCD/GCM-14/01, University of California at Davis, California, USA.


IV. IMPLEMENTATION ON SCOSSA CODE

In this chapter, after a brief description of SCOSSA code, the implementation of the integrated pwp and dissipation model inside the numerical computer program is detailed. The performance of the code in total and effective stress conditions are finally compared with the procedure implemented in others one-dimensional computer codes.

IV.1. The SCOSSA computer code

SCOSSA, acronym of ‘Seismic Code for Stick-Slip Analysis’, is a computer code for one-dimensional seismic response analysis including shear strength of soils (Tropeano et al., 2016). In fact, the code is able to model both the ‘transient’ seismic response (‘stick’ mode) and the permanent deformation mechanisms accounting for the coupled effects of deformability and strength (‘slip’ mode).

For the purposes of this study, only the stick mode will be described because useful to the implementation of the integrated pwp model.

The code models the soil profile as a system of consistent lumped masses, connected by viscous dampers and springs with hysteretic non-linear behaviour (Figure IV.11).

The discretization of the subsoil profile into the lumped parameter system is performed on Lysmer and Kuhlemeyer (1969) criterion, where the maximum sub-layer thickness is a fraction (1/6 - 1/8) of the minimum wavelength to be transmitted.
In the stick phase, the seismic response in terms of absolute displacements, \( u_a \), to a base ground motion, \( \dot{u}_g \), can be computed by integrating the following system:

\[
\mathbf{M} \ddot{u}_a + \mathbf{C} \dot{u}_a + \mathbf{K} u_a = \mathbf{f}
\]  

(IV.1.1)

where \( \mathbf{M}, \mathbf{C}, \mathbf{K} \) are the mass, damping and stiffness matrices, \( \mathbf{f} \) is the vector of applied forces. This latter depends on the input accelerogram, \( \ddot{u}_g \), which can be applied as ‘within’ or ‘outcrop’ motion and on the bedrock impedance. The \( \mathbf{f} \) vector can be set as equal to

\[
\mathbf{f} = \mathbf{i} \cdot \begin{cases} 
 c_n \ddot{u}_g + k_n u_g & \text{ (within motion - rigid bedrock)} \\
 2c_n \ddot{u}_g & \text{ (within motion - deformable bedrock)} \\
 c_{n+1} u_g & \text{ (outcrop motion - deformable bedrock)}
\end{cases}
\]

(IV.1.2)

where

- \( \mathbf{i} \) is a vector with each element equal to zero, except for the \( n \)-th (for inside motion), or \( (n+1) \)-th (for outcrop motion), equal to unity;
- \( c_n \) and \( k_n \) are the viscous damping coefficient and the spring elastic stiffness of the \( n \)-th element, respectively;
- \( c_{n+1} = \rho V_{S_b} \) is the seismic impedance of the bedrock.

The elements of the \( \mathbf{M} \) and \( \mathbf{K} \) matrices are defined from the mass, \( m_i \), and the spring stiffness, \( k_i \), for a generic layer \( j \), as follows:

\[
m_i = \frac{\rho_i h_i}{2}; \quad m_j = \frac{\rho_j h_j + \rho_{j-1} h_{j-1}}{2}; \quad m_{n+1} = \frac{\rho_n h_n}{2}
\]

(IV.1.3)
where $\rho_j$, $h_j$, and $G_j$ are the density, thickness and shear stiffness of the $j$-th layer, respectively. The viscous damping matrix, $C$, is defined according to the full Rayleigh damping formulation (Hashash and Park, 2002)

$$C = \alpha_K M + \beta_K K$$  \hspace{1cm} (IV.1.5)

The constants $\alpha_R$ and $\beta_R$ are set as functions of the minimum soil damping ratio, $\xi_{\min}$, the fundamental frequency of the subsoil profile and the predominant frequency of the input motion. The non-linear hysteretic response of the springs is modelled with an extended version of the modified Kondner-Zelasko (1963) hyperbolic model, MKZ (e.g. Matasovic and Vucetic, 1993), defined by two different relationships for loading and for unloading-reloading conditions. The stress-strain relationship (i.e. the backbone curve) for loading is given by

$$F_{bl}(\gamma) = \frac{\gamma G_0}{1 + \beta (\gamma / \gamma_r)^s}$$  \hspace{1cm} (IV.1.6)

where $\gamma$ is the shear strain level, $G_0$ is the initial shear modulus, $\gamma_r$ is the reference shear strain and $s'$ are two dimensionless factors. The stress-strain relationship for unloading-reloading condition can be either defined by the conventional Masing (1926) criteria or by the more general formulation recently proposed by Phillips and Hashash (2009)

$$F_{ur}(\gamma, \gamma_m) = F^*(\gamma_m) \left[ \frac{G_0(\gamma - \gamma_r)}{1 + \beta \left( \frac{\gamma - \gamma_r}{2 \gamma_r} \right)^s} - F^*(\gamma_m) \right] + G_0(\gamma - \gamma_r) \left( 1 + \beta \left( \frac{\gamma_m}{\gamma_r} \right)^s \right)$$  \hspace{1cm} (IV.1.7)

In equation (IV.1.7), $\gamma_c$ and $\tau_c$ are, respectively, the reversal shear strain and shear stress, $\gamma_m$ is the maximum shear strain attained during the time history, and $F^*(\gamma_m)$ is a damping reduction factor, defined as follows

$$F^*(\gamma_m) = \frac{\xi_{\exp}(\gamma)}{\xi_{\text{Mas}}(\gamma)} = p_1 - p_2 \left( 1 - \frac{G(\gamma_m)}{G_0} \right)^{p_3}$$  \hspace{1cm} (IV.1.8)

In equation (IV.1.8), $p_1$, $p_2$ and $p_3$ are non-dimensional parameters obtained from the best fit of the ratio between the strain-dependent hysteretic damping measured in laboratory tests, $\xi_{\exp}(\gamma)$, and that calculated using the conventional Masing rules, $\xi_{\text{Mas}}(\gamma)$; $G(\gamma_m)$ is the secant modulus corresponding to the maximum shear strain $\gamma_m$ (see Phillips and Hashash, 2009, for details). The updated formulation modifies the Masing unloading-reloading rules, providing a better agreement with the experimental damping-strain curves for large shear strains.
IV.2. Verification of SCOSSA code in total stress analysis

The SCOSSA code (Tropeano et al., 2016) in the first version in total stress analysis has been verified and validated with other codes during an international benchmark: the PRENOLIN Project. The details about the project and the description of the main results about SCOSSA code will be described in the next paragraphs.

IV.2.1. The PRENOLIN Project

PRENOLIN is an international benchmark part of two French larger projects: SINAPS®, funded by the ANR (French National Research Agency) and SIGMA, funded by a consortium of nuclear operators (EDF, CEA, AREVA, ENL).

One of the objectives of the PRENOLIN (PREdiction of soil NONlinear effects) project is the assessment of uncertainties associated with non-linear simulation of 1D site effects. An international benchmark is underway to test several numerical codes, including various non-linear soil constitutive models, to compute the non-linear seismic site response. Table IV.2.1 reports the list of the participating teams and the numerical codes used.

Twentyone participating teams have tested twentythree different numerical codes, which are characterized by more than ten different constitutive models (Regnier et al., 2016). Essentially three different, non-exclusive, code groups were identified by the organization team, according to three main characteristics: 1) numerical implementation scheme; 2) damping formulation, both in the low strain range and in the large strain range; and 3) cyclic hysteretic behaviour formulation (Regnier et al., 2016).

A preliminary verification phase (i.e. comparison between numerical codes) on simple and ideal cases has been performed with 23 different codes. It is followed by a validation phase, with the aim to compare the predictions of the numerical computation with actual strong motion data recorded in a selection of well-characterized sites. The verification was initially conducted by using elastic and visco-elastic soil-behaviour model to ensure a common understanding of the parameters by comparing the results with the available analytical solution. Then, the non-linear results provide a quantification of the epistemic uncertainties linked to wave propagation modeling using different non-linear rheological models (Regnier et al., 2015a).
Table IV.2.1 – Numerical codes and related participating teams to the Prenolin Project

<table>
<thead>
<tr>
<th>Code</th>
<th>Identification team code</th>
<th>Team</th>
<th>Team institution</th>
</tr>
</thead>
<tbody>
<tr>
<td>SeismoSoil</td>
<td>A - 0</td>
<td>Assimaki D. &amp; Shi J.</td>
<td>Caltech, USA</td>
</tr>
<tr>
<td>FLIP</td>
<td>B - 0</td>
<td>Iai S.</td>
<td>DPRI, Japan</td>
</tr>
<tr>
<td>PSNL</td>
<td>C - 0</td>
<td>Kramer S.</td>
<td>University of Washington, USA</td>
</tr>
<tr>
<td>CYBERQUAKE</td>
<td>D - 0</td>
<td>Foerster E.</td>
<td>CEA, France</td>
</tr>
<tr>
<td>NOAH – 2D</td>
<td>E - 0</td>
<td>Gelis C. &amp; Delavaux E.</td>
<td>IRSN, France</td>
</tr>
<tr>
<td>NTUA</td>
<td>G - 0</td>
<td>Gazetas G., Garini E. &amp; Gerolymos N.</td>
<td>NTUA, Greece</td>
</tr>
<tr>
<td>UCSD - OpenSees</td>
<td>H - 0</td>
<td>Gingery J.</td>
<td>UCSD, USA</td>
</tr>
<tr>
<td>DEEPSOIL NL</td>
<td>J - 0</td>
<td>Hashash Y. &amp; Harmon J. Giannakou A.</td>
<td>University of Illinois, USA</td>
</tr>
<tr>
<td></td>
<td>F - 0</td>
<td>Foti S.</td>
<td>Fugro, France</td>
</tr>
<tr>
<td></td>
<td>L - 2</td>
<td>Lanzo G. &amp; Pagliaroli A.</td>
<td>PoliTo, Italy</td>
</tr>
<tr>
<td></td>
<td>M - 2</td>
<td></td>
<td>Università Roma La Sapienza, Italy</td>
</tr>
<tr>
<td>DEEPSOIL EL</td>
<td>J - 1</td>
<td>Hashash Y. &amp; Harmon J.</td>
<td>University of Illinois, USA</td>
</tr>
<tr>
<td>1DFD – NL - IM</td>
<td>K - 0</td>
<td>Moczo P., Kristek J. &amp; Richterova A.</td>
<td>CUB, Slovakia</td>
</tr>
<tr>
<td>ICFEP</td>
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<td>Foti S. &amp; Kontoe S.</td>
<td>PoliTo, Italy &amp; Imperial College, UK</td>
</tr>
<tr>
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<td>Lanzo G. &amp; Pagliaroli A.</td>
<td>University Roma La Sapienza, Italy</td>
</tr>
<tr>
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<td>Lanzo G. &amp; Pagliaroli A.</td>
<td>University Roma La Sapienza, Italy</td>
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<td>GEFDyn</td>
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<td>EGISPEC1D</td>
<td>Q - 0</td>
<td>De-Martin F.</td>
<td>BRGM, France</td>
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<td>real ESSI</td>
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<td>Jeremic B., Pisano F.</td>
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<td>S - 0</td>
<td>Nieto-Ferro A.</td>
<td>EDF, France</td>
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<td>SCOSSA 1.2</td>
<td>T - 0</td>
<td>Tropeano G., Silvestri F. &amp; Chiaradonna A.</td>
<td>UniCa &amp; UniNa, Italy</td>
</tr>
<tr>
<td>STRATA</td>
<td>T - 1</td>
<td>Tropeano G., Silvestri F. &amp; Chiaradonna A.</td>
<td>UniCa &amp; UniNa, Italy</td>
</tr>
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<td>SWAP – 3C – 1C</td>
<td>U - 0</td>
<td>d’Avila Santisi M.P.</td>
<td>UNS Nice, France</td>
</tr>
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<td>SWAP – 3C – 3C</td>
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<td>d’Avila Santisi M.P.</td>
<td>UNS Nice, France</td>
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<tr>
<td>GDNL</td>
<td>Y - 0</td>
<td>Mercaret D.</td>
<td>CEREMA, France</td>
</tr>
<tr>
<td>SANISAND – Opensees</td>
<td>W - 0</td>
<td>Taiebat M. &amp; Arduino P.</td>
<td>UBC, Canada &amp; University of Washington, USA</td>
</tr>
<tr>
<td>EERA</td>
<td>Z - 0</td>
<td>Boldini D. &amp; Amorosi A.</td>
<td>UniBo &amp; PoliBa, Italy</td>
</tr>
<tr>
<td>Plaxis</td>
<td>Z - 1</td>
<td>Boldini D. &amp; Amorosi A.</td>
<td>UniBo &amp; PoliBa, Italy</td>
</tr>
</tbody>
</table>

IV.2.1.1. The verification phase

The verification phase was devised around three 1D canonical cases, chosen to represent simple and ideal soil conditions overlying rigid bedrock substrata; however, varying in complexity (Regnier et al., 2015a)
- Profile 1 (P1): A homogeneous soil layer of 20 m thickness with significant velocity contrast with $V_s$ equal to 300 and 2000 m/s for the soft and sediment layers respectively in order to have significant amplification effects at the intermediate frequency (2-10 Hz);
- Profile 2 (P2): A soil layer of 100 m thickness with a positive vertical velocity gradient (from 150 m/s at the surface to 500 m/s at the soil-bedrock interface) leading to significant amplification at frequencies below 1 Hz;
- Profile 3 (P3): Two homogeneous layers of 20 and 30 m thickness with significant velocity contrasts with $V_s$ equal to 300, 600 and 2000 m/s for the first two soft layers and sediment layers respectively. It was constructed to investigate non-linearity effects within both layers (amplification effects begin around 2.5 Hz), since significant strains can develop at soil interfaces.

P-wave velocity was computed from $V_s$ profiles by setting the Poisson ratio equal to 0.4 for the soil and 0.3 for bedrock. Attenuation parameters were constrained to vary according to $V_s$, using the relationship $\xi = 5/V_s$. No soil shear strength was provided.

The non-linear soil properties provided are typical degradation curves of the normalised shear modulus, $G/G_0(\gamma)$, and those that describe the damping ratio increase with the shear strain, $D(\gamma)$, characterizing the rheological behavior of each layer per profile. It does not represent a complete description of the non-linear soil behaviour because it is restricted to shear deformation and volumetric ones and coupling between both are ignored. These curves were constructed using a simple hyperbolic law that was calibrated adopting the soil properties at the middle of each soil layers (Regnier et al., 2015a).

The soil profile and related shear modulus reduction and damping curves are reported in Figure IV.2.1.
Figure IV.2.1 – $V_s$ profiles, $G/G_0$ and damping curves for the three ideal profiles (Regnier et al., 2016)

The reference input motions used in the project are showed in Figure IV.2.2a, b and they consist of an analytical pulse function (Ricker-beta wavelet with central frequency of 4 Hz – labelled as ‘Pulse’ in Fig. IV.2.2a) and two real recorded accelerograms characterised by high and low frequency contents (with central frequencies about of 11.4 – labelled as ‘Real LF’ - and 4.8 Hz – labelled as ‘Real HF’ - respectively), each scaled to three PGA levels (0.005, 0.01 and 0.05 g), in order to generate a wide range of shear strain levels in the soil column. They are used as input motion at the base of the soil column (at the sediment/bedrock interface) to assess the soil responses. Two boundary conditions at the sediment/bedrock interface were tested, elastic and rigid. The latter condition was considered because in the stage of validation down-hole recordings are used as reference motion (Stewart et al., 2008).
A total of 72 computations were performed for each code during the verification phase. Figure IV.2.3 displays the comparison for the P1 profile of the surface acceleration time history for the pulse-like motion and elastic substratum condition, for the linear elastic computation for a short window (3 s) of signal. All results converged towards the analytical solution calculated with the Haskell-Thompson method (Haskell, 1953; Thomson, 1950) that was achieved among all the participant codes after the second iteration.

The comparison of the results of SCOSSA code with that obtained by the other teams are reported in Figure IV.2.4 for the strongest input motion with PGA equals to 0.5 g. The comparison of the transfer functions shows that the SCOSSA results are always included in the shadow of the others codes for both the bedrock conditions and for the three different ideal soil profiles.
The comparison in terms of maximum shear strain profiles (labelled as $\varepsilon_{\text{max}}$ in Figure IV.2.4b) and stress-strain curves shows that the maximum shear strains exceed slightly the range at the base of the homogeneus profile (P1 and P2) and at the interface between the layers in the P3 profile.

Figure IV.2.4 – Results of the non linear analyses performed by SCOSSA code on three ideal soil profiles for the input motion real-3-HF in case of elastic and rigid bedrock compared with the results of the other codes in terms of transfer function (a) maximum shear strain profiles (b) and stress-strain cycles (c)
IV.2.1.2. The validation phase

To validate the numerical code predictions, the results obtained from the models have been compared with actual observations for a selection of sites with direct measurement of the seismic response. The sites are as close as possible to a 1D soil geometry (horizontal stratification) and associated with available sets of downhole and surface recordings for weak and strong motions. Such informations were completed with careful in-situ and laboratory measurements sufficient to calibrate the parameters of constitutive models. The sites were selected within the Japanese KiK-net and PARI (Port and Airport Research Institute) networks. The site selection was performed by the organization team on the following requirements: availability of both strong and weak events recordings, plausibility of a 1D geometrical soil configuration, i.e., satisfactory agreement between numerical and empirical site responses in the linear/weak motion range, and the downhole sensor must not be less than 250 m (Regnier, 2015b).

Finally, three sites were selected KSRH10 from KiK-net, Onahama and Sendai from PARI, to be fully characterized for the purpose of the validation phase. The KSRH10 site is mainly a clay soil and it not will be described in this study. The other two sites, Sendai and Onahama, are particularly interesting because they consist of sandy soil susceptible to liquefaction. So, they will be widely described and all the analyses results and comparisons will be shown in the chapter V.

IV.3. Compatibility of non-linear stiffness and strength

The MKZ model implemented in SCOSSA code belongs to the hyperbolic model family (Konder and Zalasko, 1963; Hardin and Drnevich, 1972; Darendeli, 2001; Menq, 2003). These shear modulus reduction curves fit their parameters directly to cyclic laboratory test results. However, cyclic laboratory tests often are not run to failure shear stress levels and, consequently, the model G/G0 curves are well constrained by the data at small to moderate shear strains, but do not necessarily provide an accurate representation of soil strength at large shear strains. In some cases, the shear strength can be grossly inaccurate, which may result in significant errors for analyses involving shear stress levels at or near failure (Gingery and Elgamal, 2013).

This inaccuracy is not acceptable when liquefaction phenomena are involved, or more generically, when non-linear effects induced by strong motions want to be simulated. Moreover, the proposed pwp pressure model works on cyclic resistance parameters, one of them, the threshold shear stress ratio, SR0, can be determined directly on the backbone curve. So, it is very important that coherence between stiffness and strength will be garanteed.
Several researchers have proposed \( G/G_0 \) curve adjustment procedure. Stewart et al. (2008) propose to assume the constant value of shear strength when a certain shear strain level (typically 0.1 -0.3 \% ) is reached. This procedure can produce a noticeable singularity in the modulus reduction curve, and additional smoothing is necessary to achieve a satisfactory result. Hashash et al. (2010) proposed manually strength corrections since the backbone curve could underestimated or overestimated shear strength depending on the case. Such corrections are often time-consuming and highly subjective, so that Groholski et al. (2015) proposed directly a new model defines the shear strength of soil at failure, still providing the ability to represent small-strain stiffness nonlinearity.

In this study, an interpolation scheme presented by Gingery and Elgamal (2013) is adopted, which provides a smooth, kinkless hyperbolic-like curve to transition between moderate and failure-level shear strains. \( G/G_0 \) vs \( \gamma \) curves simultaneously match the hyperbolic models at small to moderate strains and the shear strength at large strains.

The Gingery and Elgamal correction can be applied to any model belonging to the hyperbolic family curves; in particular, in this study it has been applied to the MKZ model (IV.1.4), where the normalized shear modulus reduction curve is defined as

\[
\left( \frac{G}{G_0} \right)_{MKZ} = \frac{1}{1 + \beta (\gamma/\gamma_r)^a}
\]  

(IV.3.1)

The Gingery and Elgamal procedure (GH model) scales equation (IV.3.1) using a raised cosine function to force the backbone curve at the correct shear strength by intercepting a \( G/G_0 \) value at a defined shear strain value (i.e., the supposed failure strain) . Analytically, three different range of shear strain are identified by two limit values of shear strain, \( \gamma_1 \) and \( \gamma_2 \). The equation (IV.3.1) is multiplied by a raised cosine weighting function, \( W \), as

\[
\left( \frac{G}{G_0} \right)_{GH} = W \cdot \left( \frac{G}{G_0} \right)_{MKZ}
\]  

(IV.3.2)

where

\[
W = \begin{cases} 
1 & \text{for } \gamma \leq \gamma_1 \\
1 + \left[ \frac{(G/G_0)_{MKZ,\gamma_2}}{(G/G_0)_{MKZ,\gamma_1}} - 1 \right] \left\{ \frac{1}{2} - \frac{1}{2} \cos \left[ \pi \left( \frac{\ln(\gamma) - \ln(\gamma_1)}{\ln(\gamma_2) - \ln(\gamma_1)} \right) \right] \right\}^a & \text{for } \gamma_1 < \gamma \leq \gamma_2 \\
\frac{\tau}{\gamma \cdot G_0 \cdot (G/G_0)_{MKZ,\gamma}} & \text{for } \gamma > \gamma_2
\end{cases}
\]  

(IV.3.3)
The GH model uses equation (IV.3.1) for $\gamma<\gamma_1$, between $\gamma_1$ and $\gamma_2$ the equation (IV.3.1) is multiplied by a raised cosine weighting function, $W$, and for $\gamma>\gamma_2$, $G/G_0$ is provided in order to have the shear stress equals to the strength, $\tau_{ff}$.

The $G/G_0$ value that provides the correct shear strength at $\gamma_2$ is:

$$\left(\frac{G}{G_0}\right)_{\gamma_2} = \frac{\tau_{ff}}{G_0 \cdot \gamma_2}$$  \hspace{1cm} (IV.3.4)

$(G/G_0)_{MKZ,\gamma_2}$ in eq. IV.33 is the shear modulus reduction value predicted by the MKZ model at $\gamma_2$. The exponent $n$ can be used to modify the shape of the raised cosine weighting function, but a value equal to unity is enough to have a good correction. The suggested value of 0.05% and 6% for $\gamma_1$ and $\gamma_2$ respectively, are generally adequate for a wide range of conditions (Gingery and Elgamal, 2013). An example of modified $G/G_0$ curve is shown in Figure IV.3.1.

![Figure IV.3.1 - Normalized shear modulus reduction (a) and shear stress vs shear strain (b) curves computed using the MKZ and the GH model](image)

The value of shear strength, $\tau_{ff}$, depends on the initial stress condition in the soil and on how the shear stress is applied. For initial geostatic stress conditions and shear stress applied to horizontal and vertical planes, $\tau_{ff}$ can be calculated, as reported by Hardin and Drnevich (1972), as

$$\tau_{ff} = \left[\left(\frac{1+K_0}{2} \cdot \sigma_{v} \cdot \sin \varphi' + c' \cdot \cos \varphi'\right)^2 - \left(\frac{1-K_0}{2} \cdot \sigma_{v} \right)^2\right]^{1/2}$$  \hspace{1cm} (IV.3.5)
where $K_0$ is the coefficient of lateral stress at rest; $\sigma_v$ is the vertical effective stress and $c'$ and $\varphi'$ are the static strength parameters in terms of effective stress. Hardin and Drnevich (1972) precised that effective stress conditions are used in equation (IV.3.5) even though undrained conditions occur for dynamic loading.

The modified $G/G_0$ curve, obtained from the application of the GH model, can be interpretated again with the MKZ model. In this way, the final parameters of the MKZ model not only are representative of the best fitting on the available laboratory data, but they are also respectful of the shear strength of the soil.

**IV.4. Modelling the pore pressure buildup and dissipation and the induced stiffness changes**

The excess pore pressure generation and dissipation model, described in chapter II, has been implemented in the SCOSA code. The pore pressure change induced a variation of the effective stress state, which has important consequences on the stiffness and strength of the soil. The effect of confining pressure on dynamic properties should taking into account so that the implementation could be really effective in the code.

The influence of effective confining stress state on the small-strain shear modulus and material damping is a well-established knowledge since 60’s years and many relationships have been published (Hardin and Richart, 1963; Seed and Idriss, 1970; Hardin and Drnevich, 1972; Ishibashi and Zang, 1993; d’Onofrio et al., 1999).

Moreover, EPRI (1993) and Darendeli (2001) show that an increase of confining pressure results in a higher normalised shear modulus and a smaller damping for a given strain value (Figure IV.4.1).
Hashash and Park (2001) included the effect of confining pressure on the shear modulus model (IV.1.4); the reference strain $\gamma_r$ is no longer a constant for a soil type, but a variable that depends on the effective stress, as follows

$$
\gamma_r = a \left( \frac{\sigma_v^*}{\sigma_{ref}} \right)^b
$$

(IV.4.1)

Where $\sigma_v^*$ is the vertical effective stress at the midpoint of the soil layer, $\sigma_{ref}$ is a reference confining pressure of 0.18 MPa and $a, b$ are curve fitting parameters. In the same way, the small strain damping is related to the confining pressure through other two curve fitting parameters. The calibration of these curve fitting required a great amount of experimental data which are rarely available in the engineering practice.

Matasovic and Vucetic (1993) propose a simpler degrading model, in which the shear modulus is directly linked to the excess pore water pressure. This cyclic degradation model have been frequently used in effective stress analyses codes (e.g. DESRA-2, Lee and Finn, 1978; D-MOD2000, Matasovic and Ordóñez, 2011), and it has been implemented also in SCOSSA code in conjunction with the pwp model.

According to this degradation model, the normalized shear modulus reduction curve (equation IV.3.1) is modified as
\[
\left( \frac{G}{G_0} \right) = \frac{\sqrt{1-r_u}}{1 + \beta \left( \frac{\gamma}{\gamma_r} \right) \frac{\sqrt{1-r_u}}{1-r_u}} \quad \text{(IV.4.2)}
\]

where \( \beta, \gamma \) and \( s' \) are the parameters of the backbone curve as defined on the laboratory test data, \( r_u \) is the excess pore pressure ratio and \( v \) is exponential constant which expresses the sensitivity of the backbone curve to porewater pressure changes. Matasovic and Vucetic (1993) defined the costant \( v \) on the cycles obtained from strain-controlled DSS tests on californian sandy soils and it assumes average values between 3.5 and 5.

Defining a stress degradation index function, \( \delta_\tau \), as

\[
\delta_\tau = 1 - r_u^v \quad \text{(IV.4.3)}
\]

And the corresponding modulus degradation index function, \( \delta_\sigma \), as

\[
\delta_\sigma = \sqrt{1-r_u} \quad \text{(IV.4.4)}
\]

the stress-strain relationship (i.e. the backbone curve) for loading of equation (IV.1.6) can be generalized by

\[
F_{bb}(\gamma) = \frac{\delta_\sigma \cdot G_0 \cdot \gamma}{1 + \beta \left( \frac{\gamma}{\gamma_r} \right) \frac{\delta_\sigma}{\delta_\tau}} \quad \text{(IV.4.5)}
\]

In the same way, also the stress-strain relationship for unloading-reloading formulation (IV.1.7) of Phillips and Hashash (2009) has been generalized by Moreno-Torres et al. (2010) including degradation model

\[
F_{ww}(\gamma) = \sum \left[ \frac{G_0 \cdot \delta_\sigma (\gamma - \gamma_r)}{1 + \beta \left( \frac{\delta_\sigma}{\delta_\tau} \right) \left( \frac{\gamma - \gamma_r}{2\gamma_r} \right)^v} \right] + \sum \left[ \frac{G_0 \cdot \delta_\sigma (\gamma - \gamma_r)}{1 + \beta \left( \frac{\delta_\sigma}{\delta_\tau} \right) \left( \frac{\gamma - \gamma_r}{\gamma_r} \right)^v} \right] + r_e \quad \text{(IV.4.6)}
\]

Figure IV.4.2 shows as the shear modulus reduction curve is updated every time an excess pore pressure change occurs.
IV.4.1. Numerical integration

The numerical integration of the updated version of SCOSSA code with pwp and degradation model implemented is described in this section.

SCOSSA uses the Newmark $\beta$ method (Newmark, 1959) to numerically integrate at each time step the equations of motion

$$\begin{align*}
M \ddot{u}_{r+1} + C \dot{u}_{r+1} + K u_{r+1} &= f_{r+1} \\
\dot{u}_{r+1} &= \ddot{u}_r + \Delta t \cdot (1 - \gamma_N) \cdot \dot{u}_r + \Delta t \cdot \gamma_N \cdot \ddot{u}_{r+1} \\
u_{r+1} &= u_r + \Delta t \cdot \dot{u}_r + \Delta t^2 \left(1/2 - \beta_N\right)\dot{u}_r + \Delta t^2 \beta_N \cdot \ddot{u}_{r+1}
\end{align*}$$

(IV.4.7)

The default coefficients are set equal to $\gamma_N = 0.5$ and $\beta_N = 0.25$, so that the method is unconditionally stable and no numerical damping is introduced.

For a model with $n$ degrees of freedom, the system (IV.4.7) can be easily solved if expressed in the form

$$A \cdot x_{r+1} = B \cdot x_r + q_{r+1}$$

(IV.4.8)

where $x$ is the vector of the unknown variables

$$x = \{\ddot{u}^T, \dot{u}^T, u^T\}^T$$

(IV.4.9)

In equation (IV.4.8), $q$ is the vector of the external forces, where the first $n$ elements are equal to $f$, as defined in equation (IV.1.2), and the remaining $2n$ elements are null; $A$ and $B$ are the matrices of the integration method, defined as
Chapter IV – Implementation on SCOSSA code

IV.17

\[ A = \begin{bmatrix} M & C & K \\ -\Delta t \gamma_N \cdot I & I & 0 \\ -\Delta t^2 \beta_N \cdot I & 0 & I \end{bmatrix} \quad B = \begin{bmatrix} 0 & 0 & 0 \\ \Delta t \cdot (1 - \gamma_N) \cdot I & I & 0 \\ \Delta t \cdot (1/2 - \beta_N) \cdot I & \Delta t \cdot I & 1 \end{bmatrix} \]  

(IV.4.10)

where \( I \) and \( 0 \) are the unit and null matrix, respectively, with \( n \times n \) dimension.

The numerical solution of system (IV.4.8) is reduced to the inversion of the \( 3n \times 3n \) matrix \( A \) for each time step, which can be performed with exact or iterative methods. The code uses an exact inversion method based on the Crout-Doolittle factorization algorithm, modified for bounded matrices.

For non-linear analyses, the variability of the stiffness matrix, \( K \), depending on the solution of the system, has to be considered. At each time step, the value of the stiffness for the \( j \)-th sub-layer, \( k_j \), is computed by estimating the tangent shear modulus, \( G_j \), as follows

\[ k_j = \frac{G_j(\gamma_j)}{h_j} = \frac{1}{h_j} \frac{\Delta F_j}{\Delta \gamma_j} \]  

(IV.4.11)

In equation (IV.4.11), \( \Delta F_j \) is the increment of spring reaction force in the \( j \)-th sub-layer during the time interval \( \Delta t \), to be evaluated from the current shear strain level through either equation (IV.4.5) or equation (IV.4.6) in function of the excess pore pressure ratio, \( ru \), coming from the dissipation and generation pw model.

The shear strain increment, \( \Delta \gamma_j \), is computed through

\[ \Delta \gamma_j = \frac{\Delta u_j - \Delta u_{j+1}}{h_j} \]  

(IV.4.12)

where \( \Delta u_j = u_{j+1} - u_j \) is the displacement variation in the time interval \( \Delta t \) and \( h_j \) is the sub-layer thickness.

Due to the mutual dependency between \( K \) and the system solution, \( x \), an iterative computation needs to be done at each time step.

In detail, as a first attempt, the stiffness matrix is set equal to that calculated at the previous instant, i.e. \( [K]_{k=0}=K \); The system is solved and the strain, \( [\gamma]_{k=0} \), and stress, \( [\tau]_{k=0} \), vectors are calculated, by which the stiffness matrix of the layering is again evaluated as \( [K]_{k=1} \). The procedure is iterated until the maximum value of the relative error, \( \varepsilon_{rel} \), of two subsequent solutions, \( [x]_{k=0} \) and \( [x]_{k} \)

\[ \varepsilon_{rel} = \max \left\{ \frac{\left| x_{j_1} - x_{j_2} \right|}{\| x_{j_1} \|_2} \right\} \]  

with \( j=1, 2, ..., n \) (layer index)  

(IV.4.13)
is less than a fixed tolerance value, $\epsilon_{tol}$. This iterative solution reduces the numerical instability occurring at the reversal points in the hysteresis loop modelled with the Masing criteria.

**IV.5. References**


EPRI (1993). Guidelines for determining design basis ground motions. EPRI Tr-102293, Electric Power Research Institute, Palo Alto, CA, USA.


V. VALIDATION

In this chapter, the performances of SCOSSA code are assessed on well documented case histories in which vertical array records and, sometimes, pore pressure buildup measurements, are available. Both the accuracy and practical usefulness of the code are assessed by comparing calculated predictions and field records.

V.1. Sendai site

Sendai is an array site from the PARI network in the north-east of Japan (Figure V.1.1). The recorded station consists of a surface sensor and a downhole sensor at 10.4 m depth, located near the Sendai port. The array is founded on a Holocene sedimentary soil, called “beach ridge”, consisting of gravel and sand, formed through transportation and sedimentation by waves. This surface deposit is underlain by the Pliocene “Geba formation”, forming the northern and eastern hills and consisting of gravel stone, sandstone, tuff, tuffaceous siltstone and lignite (OYO Corporation, 2014).

As anticipating in chapter IV, the site was chosen in the Prenolin Project for the 1D geometrical soil configuration, also checked with MASW measurements (Regnier et al., 2015).

An extensive investigation campaign was carried out to obtain the in-situ shear wave profile, using downhole PS logging, and the non-linear soil parameters, using laboratory tests on samples obtained by the SPT, the undisturbed sampling and recovered boring cores.
These data and observed linear empirical site response were used by the organizing team to define a soil column on which all participants performed total stress simulations in the first iteration of the validation phase. In the second and third iterations a soil profile based on literature shear modulus and damping curves has been also adopted in the simulations (SC1 profile), which has given back a better result on the surface.

In the following sections, the experimental soil column using in the first iteration will be described and the total and effective stress analyses carried out on it by SCOSSA code will be shown. Finally, the results will be compared with the results obtained using the soil profile based on literature curves.

V.1.1. Recorded input motions

Nine recorded input motions were selected, representing 3 different PGA levels (≥ 0.06, 0.02-0.03 g and ≤ 0.01 g at the downhole sensor) and approximately 3 distinct frequency contents. The numbering of input motion corresponds to decreasing PGA level from #1 to #9 (Regnier et al., 2015).

For simplicity, only the EW components are considered and the characteristics of the downhole and surface records are reported in Table V.1.1.
Table V.1.1 – EW components of the downhole and surface records

<table>
<thead>
<tr>
<th>Record</th>
<th>Frequency content</th>
<th>downhole PHA [g]</th>
<th>surface PHA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Low frequency</td>
<td>0.251</td>
<td>0.405</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>0.025</td>
<td>0.075</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>0.005</td>
<td>0.0085</td>
</tr>
<tr>
<td>2</td>
<td>Intermediate frequency</td>
<td>0.063</td>
<td>0.223</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>0.026</td>
<td>0.052</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>0.004</td>
<td>0.0097</td>
</tr>
<tr>
<td>3</td>
<td>High frequency</td>
<td>0.062</td>
<td>0.183</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>0.035</td>
<td>0.074</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>0.012</td>
<td>0.033</td>
</tr>
</tbody>
</table>

V.1.2. Description of the array site

The stratigraphic sequence revealed by the investigations is shown in Figure V.1.2, together with SPT and Vs profiles and the corrispondent values reported on the array station by the PARI website (PARI, 2015).

The upper part of the soil column is composed of loose gravel and gravelly sand with a thickness of 1.25 m, underlying fine sand moderately compacted. In this latter, a 5 cm thick sand with silt layer is revealed, which contains organic silt and shell fractions. From 7.15 m in depth there is a slate, generally hard, which can be considered the rigid bedrock. The ground water table is located at 1.45 m from the ground surface.

The results of the investigations performed by OYO Corporation show significant discrepancies from the data reported by PARI for both SPT blow count and shear wave velocity profiles, while density are quite similar. NSPT profiles show a reduction around 4.5 – 6 m, which is not present in the shear velocity profile.

Main differences in the velocity profile are related to the first four meters below the ground level, where OYO reported a velocity of 240 m/s, which is almost two times the value of 130 m/s reported by PARI.
Undisturbed samples were retrieved at the mean depth of 3.3 and 5.4 m to conduct laboratory tests. Both samples were subjected to cyclic triaxial test to determine deformation properties of soil, while on the shallower sample a stress-controlled cyclic undrained triaxial test (CTX) and a consolidated drained triaxial test (CID) were performed. Cohesion equals to 1.2 kN/m$^2$ and internal friction angle equals to 43.63 degree were computed from the CID test.

**V.1.3. Simulations**

In the onedimensional analyses carried out by SCOSSA code the rigid bedrock was assumed at the depth of 10.4 m, where the downhole sensor is located and downhole acceleration record is applied as input motion. The geotechnical model is reported in Figure V.1.3 and Table V.1.2. The assumed shear velocity profile has been proposed by Prenolin organization team; it is an adjustment of the OYO shear wave profile in order to match the empirical borehole Fourier transfer function (Regnier et al., 2015).
The experimental equivalent Young modulus vs axial strains, $\varepsilon_a$, curves related to the 5th cycle of cyclic triaxial tests have been used for the defining the normalized shear modulus curves. According to the criteria of equality of maximum values of shear stress and strain (Silvestri, 2001), the axial strains have been converted in shear strains, $\gamma$, as

$$\gamma = 1.5 \cdot \varepsilon_a$$  \hspace{1cm} (V.1.1)

while the ratio $E/E_0$ has been assumed equals to $G/G_0$. The experimental and modelled curves are reported in Figure V.1.4b.

**Table V.1.2 – Subsoil model**

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Soil type</th>
<th>Unit weight, $\gamma$ [kN/m$^3$]</th>
<th>S-wave, Vs [m/s]</th>
<th>G/G0 and D curves</th>
<th>PWP model (only in TE )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.0</td>
<td>Sand with gravel</td>
<td>18.15</td>
<td>120</td>
<td>MKZ1</td>
<td>-</td>
</tr>
<tr>
<td>1.0 - 1.45</td>
<td>Fine sand</td>
<td>18.15</td>
<td>170</td>
<td>MKZ1</td>
<td>-</td>
</tr>
<tr>
<td>1.45 - 2.0</td>
<td></td>
<td>18.15</td>
<td>170</td>
<td>MKZ1</td>
<td>-</td>
</tr>
<tr>
<td>2.0 - 3.0</td>
<td></td>
<td>18.15</td>
<td>200</td>
<td>MKZ1</td>
<td>-</td>
</tr>
<tr>
<td>3.0 - 4.0</td>
<td></td>
<td>18.54</td>
<td>230</td>
<td>MKZ2</td>
<td>-</td>
</tr>
<tr>
<td>4.0 - 5.0</td>
<td></td>
<td>18.54</td>
<td>260</td>
<td>MKZ2</td>
<td>-</td>
</tr>
<tr>
<td>5.0 - 6.0</td>
<td></td>
<td>18.54</td>
<td>280</td>
<td>MKZ2</td>
<td>-</td>
</tr>
<tr>
<td>6.0 - 7.0</td>
<td></td>
<td>18.54</td>
<td>300</td>
<td>MKZ2</td>
<td>-</td>
</tr>
<tr>
<td>7.0 - 8.0</td>
<td>Slate (rock)</td>
<td>24.33</td>
<td>550</td>
<td>elastic</td>
<td>-</td>
</tr>
<tr>
<td>8.0 - 10.4</td>
<td></td>
<td>24.33</td>
<td>550</td>
<td>elastic</td>
<td>-</td>
</tr>
</tbody>
</table>
The rock layer above the downhole sensor has been modelled as an elastic material with constant damping equals to 1 \%.

In effective stress analyses, cyclic resistance parameters have been assigned to the fine sand layer below the ground water table. Figure V.1.5 shows the cyclic resistance and pore pressure ratio curves and the result of the CTX test performed on undisturbed sample at 3.3 m depth.

An univocal relationship has not been highlighted in the experimental results, since the pore pressure ratio curves are strongly dependent from the cyclic stress ratio applied in the tests. Consequently, a pore pressure ratio curve has been defined on the whole set of data (in red, Figure V.1.3b).

V.1.3.1. Total stress analyses results

In the total stress analyses (TT) the pore pressure model is neglected and the non linear behaviour of the soil is modelled only through the shear modulus and damping curves reported in Figure V.1.2.

The results are expressed in terms of comparison between recorded and simulated acceleration time histories and response spectra at the surface. For uniforming the representation scale, the spectral accelerations have been normalized respect to the PGA recorded at the surface for each input motion.

Figures V.1.4, V.1.5, V.1.6 show the results of the simulations in total stress conditions for the low, intermediate and high frequency content input motions, respectively.
For the low frequency input motions, the computed numerical response spectra are in good agreement with the recorded one for input #8 and #4, which have low intensity. The same good match is highlighted by the comparison of the time histories at the surface. Significant overestimation is related to the input motion #1, which strongly mobilizes the nonlinear behavior of the soil. Both simulated PGA and maximum spectral acceleration are overestimated of 1.5 times respect to the recorded values.
For the low frequency input motions, Figure V.1.5 shows that the simulations in total stress conditions overestimate the response at the surface for low values of the applied actions. The maximum amplification in the response spectra of input #5 and #9 is around 0.12 s, which is also the natural period of the assumed soil column. Overestimation of the PGA can be noticed from both the acceleration time histories and response spectra for the input #5 and #9. This result suggests that some uncertainties still remain on the definition of soil properties in linear elastic conditions.

As regard to the strongest input motion, a good prediction of the PGA of the record #2 has been returned by the simulation and the differences in the predicted and recorded time histories are limited respect to the others input.
The results of analyses on the high frequency input motions show a good prediction of the response spectra and time histories of the input #6 and #7, while a great overestimation is related to the input #3 for periods minor than 0.15 s.

V.1.3.2. Effective stress analyses results

Effective stress analyses have been performed on the all set of input motions, but the PWP model was triggered only for the input #1, #2 and #3, which have recorded dowhole PGA more than 0.06 g. So, effective and total stress analyses results coincide for all the others input motions and they need further comment.

Results of effective stress analyses for the strongest input motions are reported in Figure V.1.7, together with the comparison with recorded and total stress simulations. Spectra comparisons show that effective stress analyses induced a significant improvement in the surface response, maximum recorded spectral accelerations are well-reproduced and also the predominant period is matched.
Figure V.1.7 – Comparison for the strongest input motions between recorded and simulated response spectra and time histories at the surface in the total and effective stress analyses

Analytically, the analyses results are reported in Table V.1.3 in terms of variance of spectral acceleration in the range of 0.02 ÷ 10 s. The introduction of the PWP model reduced the variance more than three times respect to the total stress analyses for the input #1 and #3, and two times for the input #2. On average, the simulations have been improved of three times taking into account the pore water pressure effects.

Table V.1.3 – Variances between simulated and recorded acceleration response spectra for total and effective stress analyses

<table>
<thead>
<tr>
<th>Analysis type</th>
<th>Input #1</th>
<th>Input #2</th>
<th>Input #3</th>
<th>mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT</td>
<td>9.49</td>
<td>0.64</td>
<td>1.45</td>
<td>3.86</td>
</tr>
<tr>
<td>TE</td>
<td>3.02</td>
<td>0.31</td>
<td>0.31</td>
<td>1.21</td>
</tr>
</tbody>
</table>

The maximum pore pressure profile induced by the strongest input motions is reported in Figure V.1.8a. The input motion #1 caused the full liquefaction of the saturated sand layer, because is characterized by the strongest intensity, while the input #2 caused the liquefaction only of the upper part of the deposit and the input #3 reached the maximum pore pressure ratio of 0.70. The pore pressure time histories at 2.5 m depth the ground level, where the highest pwp are generated, are also reported in Figure V.1.8 for the three input motions.
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V.1.4. Comparison with other simulations

The effective stress analyses results have been compared with that obtained using the profile based on literature $G/G_0$ and damping curve. In details, during the second iteration of the Prenolin project, Darendeli (2001) curves were defined in function of the lithostatic stress state along the soil column and adopted in the analyses instead of the laboratory curves. The computed normalized shear modulus curves are definitely more non linear compared to the laboratory ones, while the damping is lower at high values of shear strains (Figure V.1.9).

The results of the total stress analyses performed by SCOSSA code on this last profile, called SC1 profile, are reported in terms of response spectra in Figure V.1.10.

Figure V.1.8 – Excess pore pressure profile (a) and time history at 2.5 m depth in the effective stress analysis for the strong ground motion #1 (b), #2 (c) and #3 (d)

Figure V.1.9 – Comparison of the normalized shear modulus and damping curves from CTX tests (red lines) and the Darendeli curves (green lines) considered in the SC1 profile
Moreover, effective stress simulations were on Sendai site by a group of participants to the preolin project, identified as W-team, who used an advanced constitutive model, called SANISAN, to model the behaviour of sand. In details, SANISAN is a simple anisotropic sand constitutive models developed within the framework of critical state soil mechanics and bounding surface plasticity (Talebat and Dafalias, 2008).

Results of the W-team are also reported in terms of response spectra in Figure V.1.10 and compared with the results of SCOSSA analyses, both on SC1 profile in total stress conditions and on profile with laboratory curves in effective stress conditions, previous described.

Figure V.1.10 - Comparison for the strongest input motions between recorded and simulated response spectra and time histories at the surface performed by W-team (violet lines) and by SCOSSA code on SC1 profile in the total conditions (orange lines) and on experimental profile in effective stress conditions (red lines)
The comparison in Figure V.1.10 shows that the SC1 profile returned the best prediction on the input motion #1, while the differences for the input #2 and #3 are limited in a narrow range. It can be noticed that the PWP model implemented on SCOSSA code is able to return predictions which are comparable with more sophisticated and advanced constitutive models.

V.2. Port Island site

Port Island is definitely the most famous case history about liquefaction. After the 1995 earthquake, widespread evidences of liquefaction induced effects on ground surface were massively observed at numerous sites on the man-made island.

Port Island is the first artificial island (the second is Rokko Island) constructed in the south of Kobe city, in the Osaka bay. This island was constructed in two phases. During the first phase, between 1966 and 1981, a 436 hectare area was reclaimed. In the second phase, the island was then extended southward by reclaiming 319 additional hectare (Figure V.2.1). A decomposite granite soil known as ‘Masado’ was used in the first reclamation phase, while sandstone, mudstone and tuff, were employed in the construction of the southern part of the reclaimed island. (Shibata et al., 1996).

A vertical array was installed in the northern area of Port Island by the Development Division, Kobe city, in October 1991 (Iwasaki and Tai, 1996). The vertical array includes triaxial accelerometers at the surface as well as at depths of 16 m, 32 m, 83 m from the ground surface. No piezometers were installed at the array site.

Figure V.2.1 - Location of Hyogoken and Kobe in Japan (Kimura, 1996) (a) and focus on Rokko Island and Port Island with location of the vertical array (b). The dashed line separates the part of the island constructed during the first and the second phase.
V.2.1. The 1995 Hyogoken Nanbu earthquake

On January 17, 1995 at 5:46 AM, the 1995 Hyogoken-Nambu earthquake with a local magnitude of 7.2 struck the southern part of Hyogo Prefecture. The epicenter was 15 km north of Awaji Island, and the hypocenter depth was located 14.3 km below the sea level (Shibata et al., 1996).

Liquefaction spreads throughout the whole island. As shown in Figure V.2.2, earthquake triggered extensive liquefaction induced effects especially in the first phase reclamation area: settlements of 20 cm were observed on average with peak values exceeding 50 cm in the most severely damaged locations. Witnesses reported that the ejection of liquefied materials continued on Port Island for nearly one hour after the earthquake (Shibata et al., 1996). Since liquefied Masado soil contains gravel, the ejection of coarse gravel was also observed around the foundation of a highway near Kobe bridge on Port Island.

Less significant liquefaction occurred in the second phase reclamation area, since during the construction sand or rod compaction pile an sand drains were adopted to accelerate consolidation of the layers underlying the reclaimed soil (Shibata et al., 1996).

![Figure V.2.2 - Distribution of liquefaction on Port Island (first phase) and areas were soil improvement were used (Shibata et al., 1996)](image)

The vertical array recorded the main shock at different depth. The peak accelerations for each components at four depths are listed in Table V.2.1.
Table V.2.1 – Recorded peak ground acceleration (mod. from Iwasaki and Tai, 1996)

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>N-S PHA [g]</th>
<th>E-W PHA [g]</th>
<th>U-D PVA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>GL 00</td>
<td>0.35</td>
<td>0.29</td>
<td>0.57</td>
</tr>
<tr>
<td>GL -16</td>
<td>0.58</td>
<td>0.55</td>
<td>0.80</td>
</tr>
<tr>
<td>GL -32</td>
<td>0.55</td>
<td>0.47</td>
<td>0.20</td>
</tr>
<tr>
<td>GL -83</td>
<td>0.69</td>
<td>0.31</td>
<td>0.19</td>
</tr>
</tbody>
</table>

The horizontal acceleration was not amplified at the ground surface, because the liquefaction of the reclaimed fill material shallower than 16 m caused de-amplification of the higher frequency components.

Pore water pressure was measured several kilometers south of the array site, in an alluvial sand layer with the same geological conditions as the vertical array site at 30 m depth. An increase of pore water pressure of more than 167 kPa was recorded 14 minutes after the earthquake, and it was also observed decrease with time (Iwasaki and Tai, 1996).

V.2.2. Description of the array site

The stratigraphic sequence at the array site is shown in Figure V.2.3. The top layer is sandy fill called ‘masa’ or ‘masado’ soil, a decomposed granite transported from the nearby Rokko Mountains, overlying a Holocene silty clay layer. These layers are followed by alternating layers of Pleistocene gravel and clay. The measured velocities for each soil layer are listed in Table V.2.2. The ground water table is located at 3 m below the surface (Ziotopoulou, 2010).

Table V.2.2 – Velocity of P and S waves of the soil layers (mod. from Iwasaki and Tai, 1996)

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Soil type</th>
<th>Soil origin</th>
<th>P-wave, Vp [m/s]</th>
<th>S-wave, Vs [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 2.0</td>
<td>Gravel</td>
<td>Fill soil</td>
<td>260</td>
<td>170</td>
</tr>
<tr>
<td>2.0 – 5.0</td>
<td>Gravel</td>
<td></td>
<td>350</td>
<td>170</td>
</tr>
<tr>
<td>5.0 – 12.6</td>
<td>Gravel</td>
<td></td>
<td>780</td>
<td>210</td>
</tr>
<tr>
<td>12.6 – 19.0</td>
<td>Gravelly sand</td>
<td></td>
<td>1480</td>
<td>210</td>
</tr>
<tr>
<td>19.0 – 27.0</td>
<td>Alluvional silty clay</td>
<td>Holocene</td>
<td>1180</td>
<td>180</td>
</tr>
<tr>
<td>27.0 – 33.0</td>
<td>Gravelly sand and silt</td>
<td>Upper Pleistocene</td>
<td>1350</td>
<td>245</td>
</tr>
<tr>
<td>33.0 – 50.0</td>
<td>Gravelly sand and silt</td>
<td></td>
<td>1530</td>
<td>305</td>
</tr>
<tr>
<td>50.0 – 61.0</td>
<td>Gravelly sand and silt</td>
<td></td>
<td>1610</td>
<td>350</td>
</tr>
<tr>
<td>61.0 – 79.0</td>
<td>Stiff alluvial silty clay</td>
<td></td>
<td>1610</td>
<td>303</td>
</tr>
<tr>
<td>79.0 – (85.0)</td>
<td>Gravelly sand</td>
<td></td>
<td>2000</td>
<td>320</td>
</tr>
</tbody>
</table>
V.2.3. Simulations

In the onedimensional analyses carried out by SCOSA code the bedrock was assumed at the depth of the three available records (16 m, 32 m and 83m). The best prediction was obtained considering the bedrock at 16 m, since the uncertainties about the mechanical behaviour of the soil were limited to that of the only liquefiable layer of Masado soil. In the following the results of the numerical analyses and the comparison with the records will be just referred to the profile shown in Figure V.2.4.

Figure V.2.3 – Soil profile; SPT and $V_s$ measurements for Port Island array (Ziotopoulou, 2010)
The record at 16 m depth was used as input motion and applied to the rigid bedrock (Figure V.2.4b). The most demanding phase was the geotechnical model of the masado soil to be used in the numerical simulations.

**V.2.4. Geotechnical model of Masado soil**

The soil used for reclamation was transported by conveyor belt from mountain areas far inland. While the soil was in transit, crushing and weathering altered the grain size distribution to one higher in fines content than the natural material (Shibata et al., 1996).

Figure V.2.5 shows the grain size distribution curve of the Masado, including coarse gravel smaller than 150 mm. It should be noted that the masado soil is well-graded, containing a fairly large portion of gravel, about 55%.
A relative density of 47% for the silty sand portion of undisturbed sample of masado was reported by Ishihara et al., 1996.

Prior to earthquake, a series of cyclic triaxial test was performed by Nagase on undisturbed samples recovered from the bottom of a cut (Ishihara et al., 1996).

After the event, cyclic undrained tests were performed by Hatanaka (1997) on undisturbed samples recovered using the in-situ freezing samples method, while other comprehensive laboratory studies on Masado soils are reported by Yoshida (2000) and Cubrinovski et al. (2000).

All these data are plotted in Figure V.2.6, together with the results of Suetomi and Yoshida (1998), that measured the liquefaction strength of undisturbed samples not far from the site of the earthquake observations. In the same Figure V.2.6, the cyclic resistance curve assumed in the analysis is also reported.

As regard to the excess pore pressure ratio curve, a relationship for gravelly soils of similar grain size distribution and relative density reported in Flora and Lirer (2013) has been considered in the analyses since no measurements were available from undisturbed samples of Masado soil (Figure V.2.7).
The non-linear and dissipative behaviour of the soil was analysed by Suetomi and Yoshida (1998) that performed also cyclic torsional shear tests on disturbed and undisturbed samples of masado soil, deposited about 18 m depth from the ground surface (Figure V.2.8). In Figure V.2.8 the experimental results are compared with the literature curves proposed by Rollins et al. (1994) for gravelly soils: they seems to be well fitted by the mean curve proposed by Rollins that was than adopted to model the non-linear and dissipative behaviour of the soil.

\[ \text{Figure V.2.7 – Excess pore pressure ratio relationship considered for masado soil} \]

\[ \text{Figure V.2.8 – Non-linear characteristics of Masado soil: (a) normalized shear modulus and (b) damping ratio vs shear strain} \]
V.2.5. Results of total stress and effective stress analyses

Since the liquefation phenomena lasted more than one hour, perfect undrained condition could be assumed during the earthquake and the dissipation has been neglected in all the analyses.

In the total stress analyses (TT) the pore pressure model is neglected and the non linear behaviour of the soil is modelled only through the shear modulus and damping curves reported in Figure V.2.8.

![Figure V.2.9](image)

**Figure V.2.9** – Comparison for the NS component between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface in the total stress analysis

In Figure V.2.9 the numerical results are compared with the recorded ones in terms of acceleration response spectra and time histories. In details, the computed NS component of the numerical response spectra seems to be in a quite good agreement with the recorded one even if it shows higher spectral accelerations for periods higher than 0.8 s. The acceleration overestimation of the numerical analyses at low frequencies can be clearly detected also in the comparison between the recorded and computed acceleration time histories where a numerical noise at high frequencies is also evident.
The total stress analysis well reproduces the PGA of the EW component, as shown in Figure V.2.10 while significantly overestimates the spectral ordinates in the range between 0.8 and 5 s. High frequencies are predominant in the simulated time history, visible discordance between record and simulation are observed after 17 s (Figure V.2.10).

![Figure V.2.10 – Comparison for the EW component between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface in the total stress analysis](image)

The results of the effective stress analysis are reported in terms of response spectrum and time history of the acceleration of the two horizontal components, in Figures V.2.11 and V.2.12 where they are again compared with the records.
The computed spectral ordinates of the NS component are lower than the recorded in the range of 0.2 ÷ 3 s and the time history is characterized by smaller accelerations if compared to the observed values. The detail of the time histories in the range 10 ÷ 40 s shows that the simulated accelerations are in phase with the records (Figure V.2.11).

Effective stress analysis of the recorded EW component is able to predict the peak spectral acceleration and slightly overpredicts the spectral ordinates beyond 1 s. Excepting the overestimation under 0.08 s and the underestimation between 0.25 ÷ 0.8 s, the shape of the recorded spectrum is well reproduced as such as the surface acceleration time history (Figure V.2.12).
The liquefaction strength attributed to the masado soil caused the excess pore pressure accumulation in the saturated soil after about 13 s and the initial liquefaction condition is reached at around 17 s between 8 and 14 m under the ground level (Figure V.2.13).
The analyses results, summarized in terms of variance of spectral acceleration in the range of 0.02 ± 20 s, are reported in Table V.2.3. The mean variance becomes more than 3 times smaller from a total to effective stress analysis, showing that, notwithstanding the simple model adopted to model the pore pressure buildup, the effective stress analysis provides a substantial improvement of the prediction respect to that given by the total stress simulation.

Table V.2.3 – Variances between simulated and recorded acceleration response spectra for total and effective stress analyses

<table>
<thead>
<tr>
<th>Analysis type</th>
<th>N-S</th>
<th>E-W</th>
<th>mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>TT</td>
<td>13.20</td>
<td>34.58</td>
<td>23.89</td>
</tr>
<tr>
<td>TE</td>
<td>8.07</td>
<td>5.11</td>
<td>6.59</td>
</tr>
</tbody>
</table>

V.2.6. Comparison with literature simulations

Several authors performed simulation on Port Island site using both equivalent linear (Iwasaki and Tai, 1996) and effective stress analysis (Elgamal et al., 1996; Suetomi and Yoshida, 1998; Cubrinovski et al., 2000; Yoshida, 2000; Ziotopoulou, 2010).

An undrained effective-stress finite-element analysis of the site amplification was conducted by Elgamal et al. (1996), and the excess pore pressure ratio time history at 8 m depth was computed.
In order to compare the SCOSSA result with the prediction of Elgamal et al. (1996), the NS and EW recorded components at the surface and at 16 m depth have been projected along the N44W direction. The computed N44W recorded component at the surface has been overlapped to the available figure reported in Elgamal et al. (1996) as to set the zero of the accelerogram as considered by Elgamal et al. (1996) (green line in Figure V.2.14a).

The N44W recorded accelerogram at 16 m depth has been used as input motion of the analysis.

Finally, the excess pore pressure time history generated at 8 m depth has been compared with the results of Elgamal et al. (1996) at the same depth. The ordinate scale of excess pore pressure ratio has been adjusted in consideration that the maximum excess pore pressure ratio attained in the SCOSSA computer code is less than the unity.

SCOSSA predicts excess pore pressure accumulation at the same time instant of Elgamal et al. (1996), while the liquefaction condition is reached slightly in advance (Figure V.2.14b).

\[ \text{Figure V.2.14 – Comparison for the N44W component (a) between recorded and simulated acceleration time history at the surface and (b) excess pore pressure ratio time history computed by Elgamal et al. (1996) and by SCOSSA code (mod. after Elgamal et al., 1996)} \]

Recently, Ziotopoulou (2010) performed simulations on Port Island site using three different advanced constitutive models implemented in a finite difference code. Figure V.2.15 shows the comparison between the surface acceleration response spectra reported by Ziotopoulou and
the SCOSSA results. The comparison shows that the predictions of SCOSSA code are quite reasonable even though the simple modelling of the pore pressure generation.

![Figure V.2.15](image)

**Figure V.2.15** – Comparison between the acceleration response spectra performed by SCOSSA code and the results of other constitutive models for the EW (a) and NS (b) component (mod. after Ziotopoulou, 2010)

### V.3. Wildlife site

Wildlife site is located in one of the most seismic areas of North America, along the popular system of strike-slip faults which crosses the California region along the NW - SE direction (Figure V.3.1).

![Figure V.3.1](image)

**Figure V.3.1** – Map of the faults in the west side of North America, classified for geological age with highlighting the Wildlife site and, in red, the faults historically known (mod. after NEES website)
The Wildlife Liquefaction Array (WLA) is located in the foodplain of the Alamo River, in the Imperial Valley, 160 km east of San Diego, California (Figure V.3.2a). In this area, liquefaction manifestations were observed during the 1930, 1950, 1957, 1981 and 1987 earthquakes. Because of the high seismicity of this region, USGS (United States Geological Survey) decided to install accelerograms and piezometers in 1982. Thanks to the plain topographic condition and the sedimented horizontal deposits (Figure V.3.2b), Wildlife constitutes a natural laboratory for monitored earthquake responses and many researchers used the collected data for developing or verifying models for predicting ground response and ground deformation.

The site was equipped with surface and downhole accelerometers and six electrically transduced piezometers (Figure V.3.3). The downhole sensor was placed at a depth of 7.5 m, immediately below the liquefiable layer, and five of the six piezometers were placed within the liquefiable layer. Figure V.3.3a is a plan view of the instrumentation and Figure V.3.3b is a cross section showing the soil stratigraphy and the positions of the six piezometers and accelerometers (Bennett et al., 1984).

The soil profile can be broken into four general soil layers. The upper 2.5 m consists of sandy silt, silt and clayey silt, which is interpreted as a food-plain deposits. Cone penetration test data indicates that sediments are very loose and soft. Extending 4.3 m below, this is a silty sand layer, which is understood to be the liquefiable layer in the profile. This deposits contains two subunits identified as Wildlife sand A and B (WSA and WSB), respectively; the first 1 meter consists of very loose to loose sandy silt while the second subunit consists of loose to medium silty sand to very fine sand. The contact between the two subunits is gradational. This second

Figure V.3.2 - Location of Wildlife Liquefaction array in the Imperial Valley, California (a) and satellite image of the array site (b)
layer is interpreted to be a point-bar deposit, because located on the concave side of the river meander curve. A lacustrine silty clay deposit is extended from 6.8 to 12 m depth, underlain a cemented silt (Bennett et al., 1984).

Moreover, after the 1981 Westmoreland earthquake, a cooperative was undertaken by the University of Texas, Rensselaer Polytechnic Institute and Woodward-Clyde Consultants for making investigation of some of the sites at which prominent liquefaction occurred. An extensive cyclic strain-controlled laboratory testing program, including both triaxial and simple shear tests on intact and reconstituted sand specimens were performed by Vucetic and Dobry (1986), while resonant column tests were carried out by Haag (1985).

In details, in January 1983, subsurface investigations and field seismic tests were performed at Wildlife and others sites. At this time, approximately 27 tubes of undisturbed soil were recovered, eight of that were later transported to the Woodward-Clyde Laboratory in New Jersey and twelve were transported to the Rensselaer Polytechnic Institute in Troy, New York. The remaining seven tubes remained at Austin for laboratory testing.
Figure V.3.4 reports the exact location of the investigation, with ubication of Cross-hole, MASW and other field tests.

![Figure V.3.4 - Location of the 1983 investigation (Haag, 1985)](image)

**V.3.1. The 1987 earthquakes**

The Wildlife liquefaction array triggering four times during the Superstition Hills earthquake sequence of 23 and 24 November 1987 (Table V.3.1). The Elmore Ranch earthquake, a 6.2 magnitude event occurred on November 23rd, 1987, and the Superstition Hills earthquake, a magnitude 6.6 event that occurred the next day. The epicenters of the two main events respect to the array location are reported in Figure V.3.5.

*Table V.3.1 - Seismic sequence of the 1987 earthquakes (Zeghal and Elgamal, 1994)*

<table>
<thead>
<tr>
<th>Event</th>
<th>Date (1987)</th>
<th>Time (PST)</th>
<th>Magnitude</th>
<th>Epicentral distance [km]</th>
<th>Peak horizontal surface acceleration [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elmore Ranch</td>
<td>November 23</td>
<td>17:54</td>
<td>6.2 Mw</td>
<td>23</td>
<td>0.13</td>
</tr>
<tr>
<td>Aftershock</td>
<td>November 23</td>
<td>22:23</td>
<td>4.0 Ml</td>
<td>-</td>
<td>0.01</td>
</tr>
<tr>
<td>Superstition Hills</td>
<td>November 24</td>
<td>05:15</td>
<td>6.6 Mw</td>
<td>31</td>
<td>0.21</td>
</tr>
<tr>
<td>Aftershock</td>
<td>November 24</td>
<td>05:34</td>
<td>4.8 Ml</td>
<td>-</td>
<td>0.02</td>
</tr>
</tbody>
</table>
During Elmore Ranch earthquake, the level of the shaking was low, no seismic excess pore pressure were recorded, while Superstition Hills event caused full liquefaction (Thilakaratne and Vucetic, 1987). Sand boils erupted water and muddy sediment and turned the arid array site into a quagmire, affected an area in the foodplain of about 33 ha. Liquefaction interested the silty sand layer, especially in the upper part of the stratum (WSA) as reported by Vucetic and Dobry (1986). Extensive ground cracking indicative of lateral spreading accompanied liquefaction at the array and cumulative opening across ground cracks was 126 mm (Holzer et al., 1989).

The recorded acceleration time histories and acceleration spectra of the horizontal components of Elmore Ranch earthquake are reported in Figure V.3.6. The surface PGA recorded at the array station, was 0.127 g and 0.128 g for NS and EW component, respectively. At the downhole sensor depth, the recorded maximum acceleration was 0.078 g for the NS and 0.068 g for the EW component.

Both the components had the maximum spectral acceleration at 0.16 s while the maximum amplification between the downhole and surface records is at 0.26 s.
Chapter V – Validation

Figure V.3.6 - Recorded acceleration time histories of the Elmore Ranch earthquake at surface (a,b) and at the 7.5 m downhole sensor depth (c,d), and acceleration response spectra (e,f)

The acceleration time histories and spectra of the horizontal components of Superstition Hills earthquake are reported in Figure V.3.7. The surface PGA was 0.171 g for the NS and 0.105 g for the EW component. At the downhole sensor depth, the recorded maximum acceleration was 0.183 g for the NS and 0.205 g for the EW component.

For the EW components, the maximum spectral acceleration was reached at 0.13 s, which is the period that corresponds to the maximum amplification too (Figure V.3.7f). It can be observed that the EW components

For the NS components, the maximum spectral acceleration was reached at 0.34 s while the maximum spectral amplification was at 1.3 s (Figure V.3.7f). The mean period is 0.907 s for the downhole acceleration and 1.186 s at the surface.

The change in frequency content can directly be seen in ground surface accelerograms component, where a sudden change in frequency occurs at approximately 15–18 s (Figure V.3.7a,b).
The change in frequency content can directly be seen in ground surface accelerograms component, where a sudden change in frequency occurs at approximately 15–18 s (Figure V.3.7a,b). Liquefaction triggering detection procedure can be performed using the Stockwell transform (Stockwell et al., 1996), which combines elements of both short-time Fourier transforms and wavelet transforms for a balanced resolution of both time and frequency (Kramer et al., 2015). Figure V.3.8 shows the normalized Stockwell spectrum for the NS component of the Superstition Hills record in the first 30 s. At time of 13 s, the normalized amplitudes of the high-frequency components quickly drop and the amplitudes of the very low-frequency components persist for the remainder of the motion. At around 20 and 27 s, strong high-frequency components can be observed, which are related to dilation pulses occurred after the initiation of liquefaction.
Prevalence of low frequency motions in the latter part of the ground motion is the direct consequence of the pore water pressure increasing. Except for the P4, all the piezometers worked during the event, but the accuracy of the recorded pore pressure signals has been the subject of some controversy over the years and only the piezometer P5 record at 2.9 m depth is considered actually reliable (Holzer and Youd, 2007). The P5 piezometer is very close to the surface and the drops in the record are in agreement with the mechanism of lateral spreading at the surface (Zorapapel and Vucetic, 1994). Similar but smaller pore water pressure drops can be also noticed on the record of the other shallow piezometer P2, while such distinctive drops cannot be seen on P1 and P3 records due to the larger depth (Figure V.3.9).
V.3.2. Simulations

After that geotechnical model and the PWP model have been calibrated, effective stress analyses have been performed on Elmore Ranch and Superstition Hills earthquakes for checking the reliability of the simulations on events with different magnitude.

V.3.2.1. Geotechnical model

The modelled soil profile consists of a soil column, divided in three layers, with the rigid bedrock at 7.5 m depth, where the downhole sensor is located (Figure V.3.10). The ground water table has been assumed at 1.2 m depth as reported by Holzer and Youd (2007) and the physical properties of the soils are reported in Table V.3.2.

Table V.3.2 – Physical properties of the soils

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth [m]</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$D_{50}$ [mm]</th>
<th>$\text{Fines} &lt; 75 \mu m$ [%]</th>
<th>$\text{Clay} &lt; 5 \mu m$ [%]</th>
<th>w [%]</th>
<th>LL [%]</th>
<th>LP [%]</th>
<th>PI [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>silt</td>
<td>0.0 - 2.5</td>
<td>18.65</td>
<td>0.025</td>
<td>93</td>
<td>25</td>
<td>32</td>
<td>30</td>
<td>22</td>
<td>8</td>
</tr>
<tr>
<td>Silty sand (WSA)</td>
<td>2.5 - 3.5</td>
<td>18.82</td>
<td>0.055</td>
<td>78</td>
<td>8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty sand (WSB)</td>
<td>3.5 - 6.8</td>
<td>18.82</td>
<td>0.091</td>
<td>36</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Silty clay</td>
<td>6.8 - 7.5</td>
<td>19.18</td>
<td>0.005</td>
<td>98</td>
<td>60</td>
<td>28</td>
<td>59</td>
<td>30</td>
<td>29</td>
</tr>
</tbody>
</table>
The modelled $V_s$ profile (in black) is based directly on the result of Cross-Hole test data (in blue) reported by Vucetic and Dobry (1986) and it is more detailed of the profile proposed by Holzer and Youd (2007).

V.3.2.2. Normalized shear moduli and damping curves

The mechanical properties of silty sand deposit has been built on the resonant column test data performed by Haag (1985) on specimens at a confining pressure of 55 kPa. Three specimens for each sublayer were analyzed by Haag (1985) and the correspondant physical properties are reported in Table V.3.3. It was confirmed that the fine content significantly decreases in the sublayer WSB compared to WSA, where the the $D_{50}$ is one order of magnitude smaller.

Table V.3.3 – Physical properties of the specimens subjected to resonant column tests (Haag, 1985)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Depth [m]</th>
<th>LL [%]</th>
<th>$D_{50}$ [mm]</th>
<th>Percent fines</th>
<th>$D_{20}$ [mm]</th>
<th>Percent clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>W12A1</td>
<td>2.68</td>
<td>32.6</td>
<td>0.021</td>
<td>95</td>
<td>≤ 0.001</td>
<td>22</td>
</tr>
<tr>
<td>W11A1</td>
<td>2.68</td>
<td>23.3</td>
<td>0.072</td>
<td>50</td>
<td>0.022</td>
<td>11</td>
</tr>
<tr>
<td>W3A4</td>
<td>3.26</td>
<td>23.3</td>
<td>0.054</td>
<td>57</td>
<td>0.007</td>
<td>15</td>
</tr>
<tr>
<td>W3B1</td>
<td>4.45</td>
<td>21.5</td>
<td>0.140</td>
<td>12</td>
<td>0.100</td>
<td>2</td>
</tr>
<tr>
<td>W12B2</td>
<td>4.54</td>
<td>22.7</td>
<td>0.140</td>
<td>23</td>
<td>0.055</td>
<td>10</td>
</tr>
<tr>
<td>W3B3</td>
<td>4.82</td>
<td>24.7</td>
<td>0.097</td>
<td>38</td>
<td>0.004</td>
<td>11</td>
</tr>
</tbody>
</table>
These specimens were subjected to a detailed investigation program of the small-strains properties, summarized in terms of normalized shear modulus in Figure V.3.11. It is possible to individualize the mean behaviour of each sublayer with the highlighted curves in blue and red, respectively for WSA and WSB sublayers.

![Figure V.3.11](image)

*Figure V.3.11 – Variation of normalized shear modulus with shearing strain (mod. after Haag, 1985)*

The parameters of the MKZ model for WSA and WSB sublayers have been calibrated on experimental data representative of the behaviour of each sublayer (Figure V.3.11). In the same way, the damping curves have been defined on the experimental data reported by Haag (1985).

![Figure V.3.12](image)

*Figure V.3.12 – Normalized shear modulus and damping curves adopted in the model for WSA (a) and WSB (b) sublayers, based on RC test data reported by Haag (1985)*

Since the experimental data are related to shear strains smaller than 0.1 %, an adjustment of the shear modulus reduction curves has been performed for obtaining compatibility between strength and stiffness, according to the procedure described in chapter IV.2.
Vucetic (1986) performed monotonic triaxial tests on samples of both WSA and WSB and the friction angle was found to be equal to 37°, practically the same for both sands. Using these resistance parameter in the Hardin and Drnevich (1972) relationship, a maximum shear stress of 23.4 kPa has been computed for the WSA layer at the mean depth of 3 m and a stress of 35.13 kPa has been determined for the WSB layer at the mean depth of 5 m. Finally, the shear modulus reduction curves of WSA and WSB have been modified in order to reach the previous computed strength.

The shear modulus reduction and damping curves adopted in the model for the WSA and WSB layers are compared with the results of cyclic triaxial and simple shear (DSS) tests data (Figure V.3.12). In the same figure are also reported upper and lower bound proposed by Seed and Idriss (1970) and the theoretical curves adopted by Vucetic (1986) for performing effective stress analyses. The shear moduli curve named \( (K_2)_{\text{max}} = 30 \) is related to the application of the Seed and Idriss (1970) model to the experimental data.

As validation, it can be noticed that the curves adopted for modelling the WSA and WSB layers are very close to the experimental data at high shear strains. Further consideration that the curves have a more linear behaviour than the literature curves for sand proposed by Seed and Idriss (1970).

Since no laboratory tests were carried out on the silt and silty clay deposits, reduction curves could be defined on plasticity index of the soil as proposed by Vucetic and Dobry (1991). As reported in Table V.3.3 plasticity index is 8% for the silty layer and 29% for the silty clay.
deposit. Vucetic and Dobry (1991) curves interpolated for PI between 0 and 15% and for PI equals to 30% are reported in Figure V.3.14. The comparison with the curves based on experimental data shows that the normalized shear modulus curve of Vucetic and Dobry (1991) for PI equals to 7.5% is definitely less linear than that for sand soils, which is physically unacceptable. Consequently, the WSA curves have been adopted to model the silty deposit in the upper part of the profile.

Finally, the normalized shear modulus and damping curves of Vucetic and Dobry (1991) for PI equals to 30% have been considered for represent the dynamic properties of silty clay deposit.

![Figure V.3.14 – Shear moduli reduction and damping curves adopted in the analyses compared with literature curve for PI=7.5 (dotted lines)](image)

**V.3.2.3. Cyclic resistance curve and pore pressure relationship**

A cyclic resistance curve has been computed from the available laboratory tests. In the 1983 investigation, some stress-controlled cyclic triaxial tests were performed by Vucetic (1986) and Ladd (1984), but they are in number insufficient for defining a curve because related to different effective confining pressure. On the other hand, a great amount of strain-controlled cyclic triaxial tests was performed by Vucetic (1986) with the aim to apply his strain-based model for generation of excess pore pressure implemented in the computer code DESRA. All these tests were performed at the same effective confining pressure of about 96 kPa. Strain-controlled cyclic triaxial tests can be used for defining a cyclic resistance curve following the method proposed by Silver and Park (1976). This procedure consists in the representation of the stress ratio measured during strain-controlled CTX tests in function of the deformation amplitude applied (Figure V.3.15a). In this plot is possible to identify curves with the same number of cycles. Finally, it is possible to define the maximum shear stress ratio of each iso-cycle curve and to assign it to the correspondent number of cycles.
Alternatively, the same curve can be obtained representing the data in function of the normalized pore water pressure, but this second procedure is less simple of the previous one (Figure V.3.15b).

Figure V.3.15 - Method for defining CRC from strain-controlled cyclic triaxial tests (Silver e Park, 1976)

Figure V.3.16 shows the experimental data of six strain-controlled CTX tests performed on silty sand layer by Vucetic (1986) in the way prescribed by Silver and Park (1976). In red is highlighted the maximum of each iso-cycle curve.

Plotting the maximum points in the SR – N space has been obtained the cyclic resistance curve reported in Figure V.3.17. The experimental data of four stress-controlled cyclic tests carried out by Vucetic (1986) at a mean confining pressure of 24, 48 and 96 kPa are also reported for comparison. These latter are quite close to the cyclic resistance curve assumed in the simulations.
The parameters of the normalized pore pressure relationship have been defined on the simple shear (DSS) tests data performed by Vucetic (1986) (Figure V.3.18). DSS data have been used to calibrate the pwp model since the quality of the data was better than that related to the cyclic triaxial tests.

The best fitting of the data gave a curve which is more rigid in the first part respect to the mean trend and an underestimation in the elbow. Moreover, the maximum value of the modelled curve has been set at 0.98, in order to leave a residual resistance to the liquefied soil.
Chapter V – Validation

Figure V.3.18 - Experimental curves from DSS tests vs analytical curve (red line) used in the analyses

The numerical values of the pwp parameters are reported in Table V.3.4. The pwp parameters, calibrated on data related to the silty sand layer, have been adopted also for modelling the silty deposit under the ground water depth.

For the silty clay deposit at the bottom of the profile has been hypothesized that the volumetric threshold strain is greater enough to not induce significant excess pore water pressure during earthquake.

Table V.3.4 – Pwp parameters considered in the simulations

<table>
<thead>
<tr>
<th>α</th>
<th>SRt</th>
<th>SFr</th>
<th>a</th>
<th>b</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6072</td>
<td>0.2328</td>
<td>0.243</td>
<td>0.9858</td>
<td>0.05</td>
<td>-0.00585</td>
</tr>
</tbody>
</table>

V.3.2.4. Simulations on Elmore Ranch earthquake

The results of the analyses in terms of acceleration time histories and acceleration response spectra at the surface are reported in Figure V.3.19 and V.3.20. As regards to the NS component, the predicted response spectrum is very close to the recorded one for low and high values of period while it underpredicts the record in the range of periods 0.07 ÷ 0.3 s. A good prediction is obtained also in terms of acceleration time history because the predicted accelerogram well reproduced the record, especially in the range of 15 – 20 s.
Figure V.3.19 - Comparison between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface for the NS component of Elmore Ranch.

Figure V.3.20 - Comparison between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface for the EW component of Elmore Ranch.
As regard to the EW component at the surface, the response spectrum well simulates the recorded PGA, while underestimates the spectral ordinates in the range 0.1 ÷ 0.25 s and overestimates the spectral acceleration at around 3 s. In the simulated accelerogram can be observed amplitudes generally greater than the record (Figure V.3.20b).

In both simulations excess pore pressure are not generated, consequently total and effective stress analyses coincide for Elmore Ranch event.

V.3.2.5. Simulations on Superstition Hills earthquake

The results of the total and effective stress analyses in terms of acceleration time histories and acceleration response spectra at the surface are reported in Figure V.3.21 and V.3.22. For the NS component, the comparison of the response spectra at the surface shows that the total stress analysis induces significative overestimation of the spectral ordinates in the range 0.2 ÷ 0.8 s, while underpredicts the recorded ordinates for period greater than 0.8 s. The response spectrum of the effective stress analysis is more close to the recorded spectrum than the total stress analysis, even though a general underestimation of the spectral accelerations. In terms of time histories, it can be noticed a clear over-prediction of the recorded amplitude in the total stress analysis after 13 s, which is the instant when liquefaction triggered.

![Figure V.3.21 - Comparison between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface for the NS component of Superstition Hills](image-url)
As regard to the EW component, the response spectrum of the effective stress analysis well-reproduces the recorded PGA and the peak of acceleration even though it is reached for a period smaller than the recorded one. Both simulations show an unrealistic acceleration peak at around 0.4 s and underpredicts the spectral acceleration in the range 1 ÷ 4 s.

The comparison in terms of time histories clearly shows an overprediction of the recorded amplitude after 15 s in the total stress analysis, while the effective stress simulation is more close to the recorded trend.

Table V.3.5 reports the results of the analyses in terms of variance of spectral acceleration in the range of 0.02 ÷ 20 s. The introduction of the pwp model reduces the variance more than two times on average, and almost three times for the EW component.

Table V.3.5 – Variances between simulated and recorded acceleration response spectra for total and effective stress analyses

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Analysis type</th>
<th>N-S</th>
<th>E-W</th>
<th>mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elmore Ranch</td>
<td>TE</td>
<td>0.481</td>
<td>1.180</td>
<td>1.180</td>
</tr>
<tr>
<td>Superstition Hills</td>
<td>TE</td>
<td>5.364</td>
<td>4.880</td>
<td>5.122</td>
</tr>
<tr>
<td></td>
<td>TT</td>
<td>11.558</td>
<td>13.690</td>
<td>12.624</td>
</tr>
</tbody>
</table>

Figure V.3.22 - Comparison between recorded and simulated (a) acceleration response spectra and (b) time histories at the surface for the EW component of Superstition Hills
Both the recorded components show dilation pulses after the liquefaction triggers which are not reproduced in the SCOSA simulations.

In the EW accelerogram of the effective stress analysis is clear the drop of amplitudes after 15 s as the recorded one (Figure V.3.22 b), while the same change is not clearly detected in the simulated NS component of effective stress analysis (Figure V.3.21b). The change in frequency occurred at initial liquefaction can be observed in the ratio between Stockwell spectrogram of simulated accelerogram at surface and recorded at the downhole sensor in the first 30 s (Figure V.3.23 d,e,f). The drop in frequency content happens at 13 s as in the normalized Stockwell spectrogram of the recorded accelerograms (Figure V.3.23 a,b,c). In the simulated spectrogram the high frequencies completely disappear after that liquefaction triggered and frequencies around 8 Hz appear after the twentieth second. In the recorded spectrogram the drop of frequency content is less drastic and high frequencies are correlated to the occurrence of dilation pulses.

At 13 s starts also the accumulation of excess pore pressure even if the trend of the simulated history is less gradual than the record of P5 piezometer (Figure V.3.24). Indeed, the generation of pwp is concentrated in few seconds between the instant times 13 and 22 s. It should be precised that the recorded time history is still object of discussion since the increase of excess pore pressure seems to continue at the end of the event.
V.3.3. Comparison with literature simulations

The prediction performed in this study has been compared with the simulation performed by other authors and computer codes. Figure V.3.25 shows the comparison between the surface acceleration response spectra at the surface performed with three different advanced constitutive models (Ziotopoulou, 2010) and the SCOSSA results. The comparison for the NS component shows that the predictions of SCOSSA code is comparable with the performance of more complex models and quite close to the best of the three predictions (Figure V.3.25a).

As regard to the EW component, all the simulations seem to return the same response after 1 s, while they are very different for low periods. The SCOSSA response overestimates the accelerations around 0.3 s even if it catches the first peak of the recorded spectrum. The same peak is predicted in advance by the URS model, while it is completely absent in the simulation of the PM4-sand model, which is the best prediction in the range $0.25 \div 1$ s (Figure V.3.25b).
Figure V.3.25 - Comparison among acceleration response spectra performed by SCOSSA and results of other constitutive models for the NS (a) and EW (b) component of Superstition Hills event (mod. after Ziotopoulou, 2010)

Figure V.3.26 shows the comparison of SCOSSA results with the prediction of two codes: D-MOD2000 (Geomotions, 2007) and WAVE (Horne, 1996). The first code ignores phase transformation behavior as SCOSSA does, but the second is able to model it. It is possible to observe that SCOSSA returns the best prediction at low periods, until 0.5 s, while, for greater periods, WAVE code reproduces better the recorded spectra.

Figure V.3.26 – Comparison among acceleration response spectra performed by SCOSSA and results of other 1D codes for the NS component of Superstition Hills event (mod. after Kramer et al., 2011)
V.4. References


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Ziotopoulou K, (2010). Evaluating model uncertainty against strong motion records at liquefiable sites. Master of Science in Civil and Environmental Engineering, University of California, Davis, USA.
VI. INTERPRETING THE DEFORMATION PHENOMENA ON SCORTICHINO DYKE

In this chapter, the case history of a dyke damaged during the seismic sequence occurred in Emilia plain (Italy) in May, 2012 is presented. An extensive in-situ and laboratory investigation have been allowed the characterization of the subsoil model. Potential liquefaction phenomena of the shallow sandy soils were also taken into account in simplified analyses. Slope stability and liquefaction potential evaluations of the embankment have been highlighted some criticisms on the foundation soils of the dyke. Effective stress analyses have been adopted for studying deeply the source of damage induced by the Emilia 2012 earthquake, taking into account also the sequence of three seismic events occurred in less than ten minutes.

VI.1. The May 20, 2012 Emilia events and effects induced on Scortichino dyke

The seismic sequence that in May 2012 struck a large area of the river Po Valley (Emilia-Romagna region, Northern Italy) triggered significant fractures and deformations in a number of riverbanks located close to the earthquake epicenter. Among them, one of the most severely damaged structures turned out to be the banks of an irrigation canal known as Diversion channel of Burana, flowing through the small village of Scortichino (Municipality of Bondeno), near the historic town of Ferrara (Figure VI.1.1).
The diversion channel of Burana was designed in 1884 with a dual aim. During the winter season, the channel collects rainwater from the countryside and several small villages through a dense network of secondary channels and then it delivers the rainwater in the Panaro river. On the contrary, the direction of the water flow is reverse during the summer season when the Burana channel works as an irrigation canal for the surrounding plain.

The channel becomes a hanging canal along the Scortichino village, where an embankment was built on the purpose. Over the centuries, houses and small productive activities settled on the crest of embankment, forming little settlements along the channel where the cross section of the embankment is wider (Figure VI.1.1).

After the 2012 Emilia earthquake, large, longitudinally-oriented ground cracks where observed along a 3 km bank stretch, causing in turn severe structural damages to a large part of the approximately one hundred houses and productive activities built on the bank crown. Many of the buildings were affected by a series of failures, such as detachments and rotations of bordering buildings, cracks in walls of houses and structural damage (Figure VI.1.2).

This surface evidences did not occur with the same intensity along the bank, but they were particularly concentrated along four sections, named a-a’, b-b’, c-c’ and d-d’ (Figure VI.1.1).

Figure VI.1.1 – Scortichino bank stretch and location of the investigated sections (Tonni et al., 2015)

Figure VI.1.2 – Seismic-induced damages to structures on the crest embankment (Tonni et al., 2015)
As described in chapter I, the Emilia earthquake was characterized by widespread ground effects, and in particular liquefaction occurrence. The observed cracks pattern seems compatible with liquefaction phenomena, but surface evidences were not observed along the crest embankment. Nevertheless the lack of liquefaction evidences on the surface do not exclude the possible liquefaction of deep sandy layers, as shown by Kramer et al. (2011). Indeed, if the liquefiable soil is overlain by a thick non-liquefiable crust, sand boils will not form even if liquefaction does occur.

Moreover, Sinatra and Foti (2015) recently performed studies on the role of aftershocks in the liquefaction phenomena. They found that the excess pore pressure caused by the mainshock was likely fully retained when two subsequent aftershocks occurred about two and three minutes after the mainshock. This situation was caused by typical stratigraphic conditions where liquefiable layers are confined on top and at the bottom by low permeability layers, such as revealed in San Carlo and Mirabello (see chapter I.1). Also in Scortichino, the saturated soil of the dyke could be suffered the effects of the increasing of excess pore water pressure.

Also according to Facciorusso et al. (2014) the sequence of seismic events may have determined an accumulation of excess pore water pressure corresponding to a single event with an "equivalent" magnitude $M_{eq} = 6.68$ greater than that of the only main event.

### VI.2. Field and laboratory investigation

In order to interpret the response of such soil structure during the 2012 earthquake by identifying possible damage causes as well as to suggest relevant remedial measures and seismic risk mitigation actions towards possible future earthquakes, the Emilia-Romagna regional authority launched an in-depth study carried out by a number of research groups from various Italian universities in cooperation with technical experts of the Geological, Seismic and Soil Survey Regional Department.

To this purpose, a number of geotechnical investigations were performed, in situ and laboratory tests, for defining an accurate geotechnical model. The soil investigation was concentrated around the four aforementioned cross sections of the dyke (Figure VI.2.1). A greater number of investigations were carried out around the cross section c – c' respect to the others, since the most serious damage were observed there.

The field investigation consists of 5 boreholes, during with were retrieved 29 undisturbed samples, 12 static penetration tests with piezocone (CPTU) and 4 seismic dilatometer tests (SDMT). As shown in Figure VI.2.2, penetration tests were performed both at the crest of the riverbank and at the toe, until a depth variable between 25 – 35 m. Three of the four SDMTs
were performed from the crest of the embankment, while a SDMT was performed at the base of the slope in the area of study D, where cracks was observed after the earthquake.

\[\text{Figure VI.2.1 – Location of investigations for the four areas of study (mod. after Tonni et al., 2015)}\]

The boreholes reached 20 – 30 m under the ground level except for the borehole S3 that was performed until to 50 m depth.

The boreholes S1 and S3 were equipped with two inclinometers until 20 m depth, while the S2, S4 and S5 with Norton piezometers in order to measure the water pressure between 12 and 20 m, 16 and 20 m and 12 and 20 m under the ground level, respectively.

Finally, in-situ permeability tests were performed using the Lefranc method in the borehole S3 at the depth of 8 and 15 m.

The undisturbed samples were subjected to several laboratory tests at University of Florence, Naples Federico II, Rome La Sapienza and Reggio Calabria. The program of laboratory investigation consisted of 12 triaxial isotropic undrained tests (TX-CIU), 4 direct shear tests (ST), 4 cyclic torsional tests (CTS) and 4 resonant column tests (RC), 5 double specimen direct simple shear tests (DSDSS) and 4 cyclic simple shear tests (CSS).

Finally, Figure VI.2.2 reports the topographic profiles of the four sections detected after the seismic events (Figure VI.2.2). The height of the dyke is variable from a minimum of 5 m in the section b-b', to a maximum of 8 m in the section d-d', where is also the widest crest of the dyke (about 55 m). Moreover, Figure VI.2.2 highlighted that the slope is more steeper along the countryside respect to the riverside.
A detailed description and interpretation of both field and laboratory tests is reported in Tonni et al., (2015).

In the following sections, the c – c’ cross section will be described in detail and considered in the analysis, since it has been subjected to the most severe damage. Also the widest section d –d’, will be considered for some final consideration.

![Figure VI.2.2 – Topographic profiles of the investigated cross sections (mod. after Tonni et al., 2015)](image)

**VI.3. Characterization of subsoll model**

The geotechnical model of the dyke has been based on the soil investigations. The stratigraphic model is composed by four deposits, indicated for simplicity as AR, B, C, A units in the selected cross sections of the dyke (Figure VI.3.1). In the same figures are also reported the profiles of the penetrometric resistance, $q_t$ and pore water pressure, $u$, from the CPTU tests and the Vs profiles resulted from the SDMT tests.

![Figure VI.3.1 – Stratigraphic model of the cross section c – c’ (mod. after Tonni et al., 2015)](image)
In the stratigraphic model, the embankment is composed by subsequent layers of sand, sandy silt and silty sand, generally 10 cm thick, identified as AR unit (Figure VI.3.2a). Unit B consists of a silty sand deposit that is attributable to natural levee or river deposit from a geological point of view. Since the dyke was built with material coming from the surrounding areas, the units AR and B are basically the same material, as revealed by the grain size curves (Figure VI.3.3).

A clay or silty clay deposit is denominated unit C and, sometimes, it contains thin peat layers. This clayey soil is characterized by a plasticity index equals to 40%. As shown in Figure VI.3.1, unit C is not constantly present along the cross section of the dyke and it has a variable thick along the longitudinal axis of the dyke. Moreover, the investigation detected that the clay deposit is present only on the landside of the d – d’ section (Figure VI.3.2).

Finally, the sand layer, unit A, is a deposit of the Po River which is typical in the whole Po plain, about 40 m thick, and it consists of coarse sand with a high degree of uniformity (Figure VI.3.4). Moreover, the deepest borehole has showed that there are thin layers of clay inside the sand deposit between 30 and 34 m from the crest of the dyke.

![Stratigraphic model of the cross section d – d’ (mod. after Tonni et al., 2015)](image)

*Figure VI.3.2 - Stratigraphic model of the cross section d – d’ (mod. after Tonni et al., 2015)*

![Samples of silty sand - AR and B units (a), clay - unit B (b) and sand - unit A (c)](image)

*Figure VI.3.3 - Samples of silty sand - AR and B units (a), clay - unit B (b) and sand - unit A (c)*
The resistance parameters are mainly defined on the estimates obtained from the tests piezocone, in order to have a continuous description with depth of the shear strength of the sediment. The resulting estimates were subsequently validated on the basis of the comparison with the results of the triaxial tests. From TX-Clu tests, a peak friction angle of about $34^\circ$ has been computed for the AR and B units and $37^\circ$ for unit A (Figure VI.3.5). The clayey unit C exhibited a lower resistance of about $22^\circ$, resulted from a direct shear test (Tonni et al., 2015).

To perform dynamic analyses, it was necessary to identify the depth of the seismic bedrock and the shear wave velocity profile, $V_s$, up to that depth. Geological studies show that the area of the Po plain is characterized by a thick layer of deformable soil, with a depth of the rigid bedrock greater than 100 m (Tonni et al., 2015). The shear wave velocity profile has been
Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

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defined by integrating the results of the SDMTs with measurements of two Cross-Hole tests carried out in the towns of Mirandola and Medolla (about 20 km distant from the area of study) which reach 130 m depth (Laurenzano and Priolo, 2013). The borehole related to the Cross-Hole tests made possible to reconstruct the stratigraphic sequence at Scortichino over 50 m deep. The deep stratigraphy essentially consists of a powerful sand layer (Unit A) resting on an alternation of centimetric layers of sands and silts (AL unit).

Figure VI.3.6 shows the shear wave velocity profiles for the two vertical related to the \( d - d' \) and \( c - c' \) sections. The adopted model provides the bedrock at 120 m depth from the crest of the embankment. This model was validated by comparing amplification functions calculated by linear analysis and the spectral ratio \( H/V \) from seismic noise recorded at Scortichino. The experimental ratio \( H/V \) shows a peak at 0.9 Hz, while the subsequent peaks are located at frequencies of 1.8 and 2.5 Hz (Figure VI.3.7). For the validation of the model have been carried out various analyzes where the position of the bedrock was varied between 90, 120 and 150 m depth below the surface level. Figure VI.3.7 shows the sensitivity of the numerical amplification function to the bedrock depth. The best agreement between the experimental and numerical amplification is found for the bedrock depth at 120 m from ground level.

![Figure VI.3.6 - Soil and shear wave velocity profile for section c-c'](image)

The borehole related to the Cross-Hole tests made possible to reconstruct the stratigraphic sequence at Scortichino over 50 m deep. The deep stratigraphy essentially consists of a powerful sand layer (Unit A) resting on an alternation of centimetric layers of sands and silts (AL unit).

Figure VI.3.6 shows the shear wave velocity profiles for the two vertical related to the \( d - d' \) and \( c - c' \) sections. The adopted model provides the bedrock at 120 m depth from the crest of the embankment. This model was validated by comparing amplification functions calculated by linear analysis and the spectral ratio \( H/V \) from seismic noise recorded at Scortichino. The experimental ratio \( H/V \) shows a peak at 0.9 Hz, while the subsequent peaks are located at frequencies of 1.8 and 2.5 Hz (Figure VI.3.7). For the validation of the model have been carried out various analyzes where the position of the bedrock was varied between 90, 120 and 150 m depth below the surface level. Figure VI.3.7 shows the sensitivity of the numerical amplification function to the bedrock depth. The best agreement between the experimental and numerical amplification is found for the bedrock depth at 120 m from ground level.
The nonlinear behaviour of the soils has been defined on the results of RC and DSDSS tests (Figure VI.3.8). It can be observed that the DSDSS tests are able to reach shear strains until 10%, showing a decay of the damping ratio over that value.

The characterization of the first 30 m of sand layer has been referred to tests performed at confining pressure between 180 and 200 kPa, while the tests related to a confining pressure of 400 kPa has been used to characterize the deepest sands. The respective curve have been named A200 and A400, and this nomenclature will be adopted also in the following.

For the alternation (AL) unit, average curves between those of clay and deepest sands have been adopted. The normalized shear modulus curves have been interpreted with the Ramberg-Osgood model (1943), while the damping ratio curves have been modelled with the modified Masing criteria, as suggested by Santucci de Magistris et al. (2004).

Figure VI.3.7 - Comparison of the experimental spectral ratio H/V and the amplification functions provided by the model varying the bedrock depth
VI.4. Summary of simplified analyses

Analyses on slope stability and susceptibility to liquefaction of the embankment soils were performed using the results of seismic response analyses. In the next session, it will be described the input motion adopted for the seismic response analyses and the related results. Then, it will be described the seismic slope stability analyses of the embankment and the susceptibility analyses to liquefaction.

VI.4.1. Input motion

Since no acceleration records were available at the site of Scortichino, a selection of recorded accelerograms was adopted in the simplified analyses in order to simulate the 20th May event. The input motion was defined through a selection of records within the Italian database ITACA (Pacor et al., 2011), based on the magnitude (5.5 – 6.5) and distance (5 – 10 km) bins approach.
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Table VI.4.1 reports the characteristics of temporal traces resulting from the research. To them was added the component parallel to the fault (FP) of the L'Aquila earthquake of April 6, 2009, recorded at the AQG station and deconvoluted at the bedrock (Landolfi, 2013), in order to consider also a near-fault event in the set of accelerograms. The selected time histories were then scaled to the PGA estimated at the site through the attenuation law proposed by Bindi et al. (2011). Figure VI.4.1 reports the average spectrum (black line) of the selected accelerograms, showing a good agreement with that provided by the law of attenuation (grey line) over the whole range of periods.

**Table VI.4.1 – Input motions**

<table>
<thead>
<tr>
<th>Event</th>
<th>date</th>
<th>M&lt;sub&gt;W&lt;/sub&gt;</th>
<th>station</th>
<th>Soil class</th>
<th>R&lt;sub&gt;epi&lt;/sub&gt; [km]</th>
<th>Component</th>
<th>PGA [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friuli 4° after</td>
<td>15/09/1976</td>
<td>6.0</td>
<td>Tarcento (TRC)</td>
<td>A</td>
<td>8.5</td>
<td>NS</td>
<td>0.129</td>
</tr>
<tr>
<td>Val Nerina</td>
<td>19/09/1979</td>
<td>5.8</td>
<td>Cascia (CSC)</td>
<td>A*</td>
<td>9.3</td>
<td>EW</td>
<td>0.202</td>
</tr>
<tr>
<td>Val Comino</td>
<td>05/11/1984</td>
<td>5.5</td>
<td>Villetta Barrea (VLB)</td>
<td>A*</td>
<td>8.6</td>
<td>E-W</td>
<td>0.201</td>
</tr>
<tr>
<td>Appennino Lucano</td>
<td>09/09/1998</td>
<td>5.6</td>
<td>Lauria (LRS)</td>
<td>A*</td>
<td>9.8</td>
<td>N-S</td>
<td>0.165</td>
</tr>
<tr>
<td>L'Aquila</td>
<td>06/04/2009</td>
<td>6.3</td>
<td>Colle Grilli (AQG)</td>
<td>A</td>
<td>5.3</td>
<td>FP</td>
<td>0.308</td>
</tr>
</tbody>
</table>

**Figure VI.4.1 – Acceleration spectra of the selected input motions**

**VI.4.2. Seismic response analyses**

Seismic response analysis were conducted assuming one-dimensional propagation of shear waves through a horizontal stratigraphy. EERA (Bardet et al., 2000) code was adopted to perform equivalent linear analysis in the frequency domain.
The bedrock was modeled as a linear visco-elastic material, characterized by $V_s = 800$ m/s, and damping of 0.5%. The unit weights employed for the various layers came from laboratory determinations on undisturbed samples and SDMT tests.

The results of the analyses are shown in Figure VI.4.2 in terms of profiles of the maximum values of acceleration, $a_{\text{max}}$, shear stress, $\tau_{\text{max}}$, and shear strain, $\gamma_{\text{max}}$, reached during the shaking. Each diagram shows the results for the different inputs and the average profile. All the acceleration profiles show a more or less accentuated amplification on the surface, with $a_{\text{max}}$ ranging from 0.20 to 0.35 g. The trends of the profiles in the embankment shows some irregularities, reflecting the asynchronous motion of the silt-clay formations, B and C. Moreover, in the same layers are attained maximum values of shear strains greater than the corresponding volumetric thresholds strain, $\gamma_v$, measured in the resonant column tests, variable in the range 0.025 and 0.030%.

![Profiles of maximum acceleration, shear stress and shear strain (section c – c')]()
CSR values with depth were obtained by the maximum shear stress profile of the seismic response analysis. As described in chapter II, conventionally, the irregular shear stress history is converted in a cyclic uniform load of amplitude equals to 65% of the maximum stress. With reference to the CRR values, they were carried out with parallel independent estimates based on the profiles of the shear waves velocity, Vs, horizontal coefficient, KD, both obtained from SDMT tests, and on the penetrometric measures. A further calculation of CRR was based on the results of the CSS tests. In this case, the CRR is computed on the experimental cyclic resistance curve for a number of equivalent cycles correspondent to the earthquake magnitude of 6.1 which is characteristic of the 20th May mainshock.

The results of the analyses are reported in terms of safety factor, FSliq, defined as ratio between cyclic resistance ratio and stress ratio. Figure VI.4.3a reportes the results of the analyses carried out with the Andrus and Stokoe (2000) method, related to the shear wave velocity profile of the SDMT C test. Figure VI.4.3b shows the results of the analyses on four CPTU tests located around the section c – c’. Results of the analyses on CPTU show the presence of several FSliq <1 along the vertical profile, especially in the silty sand layer of the unit B. Conversely, contradictory results come from the laboratory CSS test and the Vs profile regarding to the sand layer of the unit A.

Figure VI.4.3 – Liquefaction analyses based on Vs profile from SDMT (a) and CPTU test (b) (mod. after Tonni et al., 2015)
VI.4.4. Seismic slope stability analyses

The seismic stability analyses were performed with both pseudo-static approach, using the SLOPE/W software (GEOSLOPE International Ltd, 2007), and displacement method (Newmark, 1965). In both cases, the seismic actions were determined from the local seismic response analyses already described.

For such of simplicity, the water table was assumed coincided with the water level in the channel, equals to 11.2 meters above sea level, for the analysis of riverside slope, while it was assumed equals to 8 m for that of landside slope as indicated by piezometer measures. A uniformly distributed load was placed on the crest of the embankment for taking into account the buildings founded there, but the overload road was neglected due to the low service level of the local road (Figure VI.4.4).

Static analyses confirmed the stability of the structure before the earthquake, with a factor of safety greater than unity.

In the pseudo-static analyses, the equivalent seismic coefficient, \( k_{h,eq} \), was determined with reference to the average profile of the maximum acceleration obtained from the seismic response analyses. In details, the equivalent seismic coefficient, \( k_{h,eq} \), was computed as

\[
k_{h,eq} = \beta \frac{a_{\text{max,eq}}}{g}
\]  

(VI.4.1)

where \( a_{\text{max,eq}} \) was the average value along the height of the landslide mass and \( \beta \) is a coefficient linked to tolerable displacements. Since the high degree of urbanization of the embankment, \( \beta \) was assumed equals to 0.5 corresponded to displacements of 2 – 3 cm.

Morgenstern and Price (1965) method provides a factor of safety of 1.003 and 1.159 for the land and riverside slope analysis, respectively (Figure VI.4.4). Due to the greater steepness, the landside slope is characterized by a safety factor lower than that on the riverside.
Moreover, displacement analyses were performed using the Newmark method. According to a pseudo-static approach, the critical acceleration, $a_c$, and the associated critical failure surface were preliminarily determined. Assimilating the slope at an overposition of horizontal layers subject to vertical propagation of S-waves, an equivalent accelerogram, $a_{eq}(t)$, was obtained as the ratio of the time history of shear stress, coming from seismic response analysis, and the total vertical stress at the base of the landslide mass.

Figure VI.4.5 shows the values of accumulated displacements along the riverside and landslide of the dyke, calculated for the five selected seismic signals. The calculated displacements are compared with the estimates obtained by the application of some empirical correlations (Rampello et al., 2010, for deformable soil; Ausilio et al., 2007, and Madiai, 2009, for rock and soil) for predicting the displacements induced by the earthquake. As expected, the displacements of the landslide slope are systematically higher than those calculated on the opposite side.

![Figure VI.4.5 - Displacement computed on both riverside and landside of the dyke and relative prediction by different empirical correlations](image)

### VI.5. Advanced analyses

Simplified analyses have excluded severe damage due to the stability of the slopes of the dyke, while local liquefaction phenomena are not excluded. Consequently, a deep study is necessary to reconstruct the condition of the dyke at the time of the earthquake. First of all, it is necessary to define the effective stress state inside the embankment in order to define clearly the saturated soil which are susceptible of liquefaction.
VI.5.1. Effective stress state inside the embankment

Filtration analysis with the code SEEP/W (GEOSLOPE International Ltd, 2007) has been possible to reconstruct the system of pore pressure within the embankment section (Figure VI.5.1), consistent with the piezometric measurements performed within the sands deep. The high degree of saturation of the undisturbed samples (> 95%), taken at small depth from the ground level, allowed to develop the analysis only with reference to the saturated permeability of soils. Hydraulic conductivity of the soils has been deducted from field investigations, both directly from Lefranc tests and indirectly using empirical correlations with CPT tests.

The boundary conditions are constituted by the free surface in the channel and the no-flow condition along the perimeter of the section, while the piezometer level equals to 8 m above sea level has been applied along the opposite side of the cross section, as indicated by the piezometric measures (Figure VI.3.1).

The presence of the channel triggers a steady-state filtration inside the embankment, but the thick layer of clay (unit C) above the sands of the Po (unit A) determines a storage basin of water in the sand layer silty (unit B).

![Figure VI.5.1 - Simulation of the pore pressure inside the embankment section (isobars in kPa)](image)

This configuration allows to identified the soil potentially liquefiable and make some first consideration about the observed damage. Indeed, the saturated soils are overlain by a non-liquefiable crust of significant thickness, at least 7 m thick in correspondence of the crest.

According to the existing relationship between the thickness of the liquefiable soils and the crust and the liquefaction manifestations at the surface, it appears that the non-liquefiable crust is too thick, thus sand boils could not be occurred, even if liquefaction happened in the depth layers (Figure VI.5.2).
Figure VI.5.2 – Relationship between thickness of liquefiable layer and thickness of overlying layer at sites for which surface manifestation of liquefaction were observed (Kramer et al., 2015)

VI.5.2. Preliminary one-dimensional effective stress analysis

Dynamic analyses have been carried out by total and effective stress analyses through the non-linear code SCOSSA. The stratigrafic profile is the same adopted in the analyses of the previous sections, only the thick of the first deposits has been adapted to the vertical section at the edge of the riverside (Figure VI.5.3 a).

With reference to the input motion, the five selected accelerograms have been further processed following the criteria suggested by Athanasopoulos-Zekkos and Saadi (2012) for selecting reference ground motions for liquefaction analysis of earth levees. The NS component of the mainshock of Irpinia earthquake (11/23/1980), recorded at the Lauria station was finally selected and adopted as input motion in the analyses (Figure VI.5.3).
Figure VI.5.3 - Soil profile (a), stiffness and damping vs strain (b), and reference input motion (c)

Figure VI.5.3b shows the normalized shear modulus and damping ratio curves, obtained from resonant column and double specimen direct simple shear tests and adopted to simulate the non-linear soil behaviour. The shear modulus reduction curves were analytically fitted by the MKZ model, modified according to the procedure for strength compatibility proposed by Gingery and Elgamal (2013), widely described in chapter IV. As an example, Figure VI.5.4 reports the analytical curves obtained for the silty sand deposit (B).

Figure VI.5.4 - Shear modulus reduction curve (a) and backbone curve (b) reproduced by MKZ and Gingery and Elgamal (2013) models in the silty sand deposit B
Data from cyclic simple shear tests (CSS) were used to calibrate the parameters of the PWP model for silty sand (B) and sand (A) deposits. Figure VI.5.5 reports the experimental results and the modelled curves used in the analyses. The relationship between the pore pressure ratio and the damage parameter was defined as the best fitting function through the available laboratory results. The number of cycles at liquefaction, \( N_L \), was established assuming that liquefaction occurs at a pore pressure ratio \( r_u = 0.90 \). Since the threshold shear stress ratio was not clearly defined by the experimental data, \( \text{SR}_t \) was estimated from the backbone curve as that corresponding to the volumetric threshold strain measured in RC tests (Figure VI.5.6). The parameter \( \alpha \) was finally determined from the slope of the cyclic resistance curve in the logarithmic plot (Figure VI.5.5a).

For the clay unit (C), the cyclic resistance curve was not measured in the laboratory, so it has been estimated by averaging from literature data on clays (Boulanger and Idriss, 2006). The threshold value, \( \text{SR}_t \), was defined from the volumetric threshold strain measured by resonant column tests, as described for unit A and B. The pore pressure ratio relationship has been defined on the result of simple cyclic shear test carried out on the sample of the unit B with maximum clay fraction. It can be observed that the accumulation of excess pore pressure is characterized by a different shape respect to that typical of the sandy materials (Figure VI.5.5b). The calibrated parameters for the three soil deposits are reported in Table VI.6.1.

\[
\begin{array}{|c|c|c|c|c|c|c|}
\hline
\text{Soil deposit} & \alpha & \text{SR}_t & \text{SR}_c & a & b & c \\
\hline
\text{Silty sand - B} & 1.71 & 0.087 & 0.176 & 0.902 & 0.534 & 0.098 \\
\text{Clay - C} & 8.548 & 0.075 & 0.940 & 1.076 & 0.513 & -0.076 \\
\text{Sand - A} & 5.20 & 0.032 & 0.159 & 0.702 & 0.613 & 0.298 \\
\hline
\end{array}
\]

Figure VI.5.5 - Cyclic resistance curves (a) related pore water pressure relationship (b)

Table VI.5.1 - Parameters of the pore water pressure model
The results of the coupled analysis are plotted in Figure VI.5.7 in terms of maximum acceleration, shear strain and shear stress profiles (green lines). The distribution of the maximum strain shows the highest value at a depth of 15.5 m, which corresponds also to a peak acceleration. In the coupled analysis the profiles of $a_{\text{max}}$ and $\gamma_{\text{max}}$ show two singular spikes at 13.5 m and 25 m in the sand (A) layers, which are not observed in the decoupled analysis. Excess pore water pressure is significant between 7 and 30 m depth, but liquefaction condition is reached only in the sand layer (A). According to the PWP model, liquefaction triggers in sand deposit (A) after 3.2 s from the beginning of the seismic event, while in the silty sand deposit (B) a maximum pore pressure ratio of 0.47 is reached (Figure VI.5.8). The excess pore pressure ratio in Figure VI.5.7 show that only the first seconds of the input motion are meaningful to the accumulation of excess pore water pressure.

Similarly, an effective stress analyses have been performed also for the d’ – d’ section, considering a vertical soil column in correspondence of the borehole S4 (Figure VI.3.2).

The results are shown in Figure VI.5.9. Excess pore water pressure is significant between 7 and 30 m depth, but no liquefaction conditions are reached conversely from the c – c’ section. Compared to total stress analysis, also in this case, the profiles of $a_{\text{max}}$ and $\gamma_{\text{max}}$ show two singular spikes at 10.9 m and 15.1 m, respectively in the silty sand (B) and sand (A) layers.
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Figure VI.5.7 - Vertical profiles resulting from coupled and decoupled dynamic analyses on c – c’ section

Figure VI.5.8 - Shear stress and excess pore pressure ratio histories at 10.9 m (a) and at 15.1 m (b) from effective stress analysis

Figure VI.5.9 - Vertical profiles resulting from coupled and decoupled analyses on d – d’ section
To sum up, preliminary effective stress analysis seems to indicate a significant increase of the pore water pressure inside the sand deposit (A). Such hypothesis about the damage source can be verified only through a reconstruction of the actual earthquake that struck the dyke. To this purpose, a definition of the most notable shocks of the 20th May 2012 is performed in the next paragraph. In detail, an input motion comprehensive of the three main shocks of the May, 20 will be considered in order to assess also the role of aftershocks in the observed damage.

**VI.5.3. Ground motions of May 20th, 2012**

The mainshock of the Emilia 2012 seismic events occurred on May 20th, 2012 at 02:03:53 UTC time (04:03:53 local time). The mainshock was followed by two strong aftershocks of local magnitude, $M_L$, 4.8 and 5.0, respectively (Table VI.5.2). The first aftershock occurred two minutes and half after the main event, while the second aftershock occurred one minute after the first.

**Table VI.5.2 – Main seismic events of May 20th, 2012**

<table>
<thead>
<tr>
<th>Event</th>
<th>Time (UTC)</th>
<th>$M_L$</th>
<th>Epicenter coordinates</th>
<th>Depth [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mainshock</td>
<td>02:03:53</td>
<td>5.9</td>
<td>44.90 Long 11.26</td>
<td>9.5</td>
</tr>
<tr>
<td>I aftershock</td>
<td>02:06:26</td>
<td>4.8</td>
<td>44.91 Long 11.16</td>
<td>4.3</td>
</tr>
<tr>
<td>II aftershock</td>
<td>02:06:28</td>
<td>5.0</td>
<td>44.81 Long 11.27</td>
<td>6.1</td>
</tr>
</tbody>
</table>

The station of the Italian Strong Motion Network (DPC) located in Mirandola recorded the strong motion and the subsequent aftershocks. This station is named MIRANDOLA (NAPOLI) in the Italian accelerometric archive (ITACA, 2016), because it is located in Napoli Street of the Mirandola Municipality, and it is identified by the station code MRN. Such station recorded the highest PGA during the mainshock of May 20th, 2012 as shown in Figure VI.5.10, since it is very close to the epicenter (Lat = 44.8782°, Long = 11.0617°). The same station recorded also the subsequent shocks which can be downloaded from the ITACA website. Unfortunately, the aftershocks recorded at the MRN station are still unprocessed on the ITACA website. Moreover, Figure VI.5.11 shows that the same data are provided for the first and the second aftershock. This acceleration time history lasts more than four minutes and this indicates that it is comprehensive of both the aftershocks and, probably, of other smaller events.

Remembering that the aim of the study is to define an input motion that includes the main three shocks of May, 20th, it has been necessary to work on the unprocessed data for uniformity, even if only the processed data of the mainshock are available on the ITACA
website. Thus, the unprocessed accelerograms of the two horizontal components of the mainshock and of the aftershocks have been downloaded from the ITACA website.

Figure VI.5.10 – Distribution of the PGA recorded by the accelerometric stations after the mainshock of May, 20th in Northern Italy

Figure VI.5.11 – EW acceleration time histories provided for the first (a) and the second (b) aftershock by the ITACA website (ITACA, 2016)
The original records of the station have been reconstructed by plotting one after one the unprocessed records of the mainshock and the aftershocks. This procedure has been possible since station was continuously recording, and the initial time of each shock is known (Table VI.5.2). From this elaboration it emerged that the time histories of the mainshock and the aftershock have an overlapping time interval (Figure VI.5.12). In fact, a perfect overlap of the time histories between 150 and 175 s was observed by reversing the sign of the aftershock record for both components.

Figure VI.5.12 – Unprocessed NS (a) and EW (b) components of the main shocks of May, 20th and focus on the overlapping time intervals (c-d)

These records cannot be used as input motions in the analyses, because the station is located on a class C site, with a $V_{S,30} = 208$ m/s according to the EC 8 code. Indeed, as widely described in sec. II.1, the 2012 seismic sequence interested an alluvial plain with a significant
depth of the seismic bedrock, so that the closest class A stations are located too far from the epicenter to provide useful input motions for the analyses.

To overcome this problem, the record in Figure VI.2.12 has been split into three events; each one of that has been de-convoluted to the bedrock and, finally, the de-convoluted outcrop motion has been scaled to account for the epicentral distance of each shock.

The original record has been split giving the three records separately. Each of them has been corrected with the same procedure applied to the processed records of the mainshock, i.e. each record has been baseline-corrected with a linear function and corrected with a 2nd order Butterworth bandpass filter between 0.04 and 40 Hz.

The EW and NS components are considered in the following, since the plot of the acceleration motion in the NS–EW plane does not highlight the same preferential direction for the three seismic events (Figure VI.5.13).

![Figure VI.5.13 – Acceleration paths for the (a) mainshock, (b) first aftershock and (c) second aftershock](image)

**VI.1.1.1. De-convolution of surface motion**

A de-convolution procedure requires the definition of the subsoil model under the recording station. The MRN station is located along Napoli street in Mirandola municipality, very close (∼100 m) to the site where a downhole seismic array has been deployed after the earthquakes (Figure VI.5.14). During the deployment of the array, a cross-hole test was carried out until 125 m depth. Figure VI.5.15 reports the stratigraphic sequence and the shear wave velocity profile obtained from the cross-hole test. This configuration has been adopted for defining the subsoil model under the recording station. Since no site-specific laboratory data are available, the shear modulus reduction and damping curves were assumed to be the same of the soil deposit at Scortichino.
De-convolution consists of assigning a recorded ground motion at the surface of a 1D soil column and using a linear equivalent analysis to calculate the acceleration time history at a point beneath the ground surface.
For soft sites, Silva (1988) measured the coherence in order to provide an estimate of goodness between the simulated propagated motion and that recorded at surface. The results of Silva (1988) research shows that frequency content beyond 15 Hz should be filtered out prior to de-convolution, since the coherence falls off at 15 Hz between the recorded surface motion and that analytically propagated to surface. For this reason, a low pass (LP) filter at 15 Hz has been applied to the recorded surface motion to be used for the de-convolution analysis. Moreover, from the coherence analysis it appears that the maximum power that is propagated as normally incident shear waves is about 75%. The remaining energy may be due to scattered waves and perhaps to P-waves. This has significant implications on the non-linear behavior of the soils. In particular, attempting to de-convolute the total surface motion as vertically propagating shear waves may result in too much energy predicted at depth, leading to excessive estimates of modulus-reduction and mobilized damping. The overall effect is to overestimate the motion at depth that is required to produce the total observed surface motion (Silva, 1988).

The guidelines for equivalent-linear de-convolution proposed by Silva (1988) were summarized by Markham et al. (2015) in the following steps:

1. a low pass (LP) filter was applied to the recorded surface motion to be used for the de-convolution analysis at 15 Hz and scaled by 0.87; SeismoSignal was used to perform a 4th order, LP Butterworth filter;
2. the filtered and scaled motion from step 1 was assigned as input motion at the surface of a 1D soil column;
3. the motion from a layer of interest at some depth below the surface is obtained via an equivalent linear solution;
4. the final iteration values of shear modulus reduction and material damping for each layer during the de-convolution process is obtained;
5. the de-convolution process was performed again by using a linear analysis with the final values of G/G₀ and D from step 4 for each layer of the 1D soil column and introducing the LP filtered (15 Hz) full surface motion (i.e., not scaled by 0.87) at the top of the column to obtain the “final”, outcropping, de-convoluted motion.

In this study, the EERA code was utilized to perform all de-convolution analyses. The de-convoluted outcrop input motions are reported in Figure VI.5.16. The time histories of the II aftershock has been cut at 125 s, since the Arias intensity reaches the 95% around the 90th second.
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Figure VI.5.16 – De-convoluted outcrop motions of the NS (a-c) and EW (d-f) components of the three shocks
VI.5.3.1. Attenuation law

As shown in Figure VI.5.17, two of three seismic events occurred much closer to Scortichino dyke than Mirandola station. Due to the great difference in distance from the epicenter of the MRN station and of the dyke site, a study of ground motion attenuation is necessary. This consists essentially of predicting ground motion at a particular site by means of attenuation laws, formulated on the basis of large strong motion databases and depending on soil type and faulting mechanism.

Similarly to Sinatra and Foiti (2015), a ground motion prediction equation (GMPE) based on the Italian strong motion database has been adopted (Bindi et al., 2011). For peak ground acceleration (PGA, cm/s²), GMPE is characterized by the following functional form

\[
\log(\text{PGA}) = e_1 + F_D(R,M) + F_M(M) + F_S + F_{\text{sof}}
\]

(VI.5.1)

where \(e_1\) is a constant term equals to 3.672, \(F_D(R,M)\), \(F_M(M)\), \(F_S\) and \(F_{\text{sof}}\) represent the distance function, the magnitude scaling, the site amplification and the style of faulting correction, respectively. \(M\) is the moment magnitude, \(M_w\), and \(R\) is the Joyner-Boore distance, \(R_{\text{jb}}\), or the epicentral distance, \(R_{\text{epi}}\), when the fault geometry is unknown (generally when \(M<5.5\)).

The proposed equation for the distance function is

\[
F_D(R,M) = \left[ c_1 + c_2 (M - M_{\text{ref}}) \right] \log \left( \frac{\sqrt{R^2 + h^2}}{R_{\text{ref}}} \right) - c_3 \left( \sqrt{R^2 + h^2} - R_{\text{ref}} \right)
\]

(VI.5.2)

while the proposed magnitude function is:
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\[
F_s(M) = \begin{cases} 
    b_1 (M-M_h) + b_2 (M-M_h)^2 & \text{for } M \leq M_h \\
    b_3 (M-M_h) & \text{otherwise}
\end{cases}
\]  

(VI.5.3)

The numerical values of the constants for peak ground acceleration are reported in Table VI.5.3.

Table VI.5.3 – Constants and Coefficients for PGA considering the geometrical mean of the horizontal components (Bindi et al., 2011)

<table>
<thead>
<tr>
<th>(c_1)</th>
<th>(c_2)</th>
<th>(h)</th>
<th>(c_3)</th>
<th>(b_1)</th>
<th>(b_2)</th>
<th>(M_{ref})</th>
<th>(M_h)</th>
<th>(R_{ref} [\text{km}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1.940</td>
<td>0.413</td>
<td>10.322</td>
<td>1.34 \times 10^{-4}</td>
<td>-0.262</td>
<td>-0.0707</td>
<td>5</td>
<td>6.75</td>
<td>1</td>
</tr>
</tbody>
</table>

The functional form \(F_s\) in Equation VI.5.1 represents the site amplification and it is given by

\[
F_s = s_j \cdot C_j \text{ for } j=1,\ldots,5
\]  

(VI.5.4)

where \(C_j\) are dummy variables used to denote the five different EC8 site classes from A to E.

The functional form \(F_{sof}\) in Equation VI.5.1 represents the type of faulting correction and it is given by

\[
F_{sof} = f_j \cdot E_j \text{ for } j=1,\ldots,4
\]  

(VI.5.5)

where \(E_j\) are dummy variables used to denote fault classes. Bindi et al. (2011) considered four types of style faulting: normal (N), reverse (R), strike-slip (SS) and unknown (U).

Table VI.5.4 shows the numerical values of the \(s_j\) and \(f_j\) coefficients. The columns \(s_A\) through \(s_E\) show the site coefficients for the EC8 classes. The columns \(f_1\) through \(f_4\) show the style of faulting coefficients for normal (\(f_1\)), reverse (\(f_2\)), strike slip (\(f_3\)) and unknown (\(f_4\)) mechanisms. The total (\(\sigma\)), the between-event (\(\sigma_B\)) and within-event (\(\sigma_W\)) standard deviations are shown as well.

Table VI.5.4 – Site and style of faulting coefficients for horizontal PGA (Bindi et al., 2011)

<table>
<thead>
<tr>
<th>(s_A)</th>
<th>(s_B)</th>
<th>(s_C)</th>
<th>(s_D)</th>
<th>(s_E)</th>
<th>(f_1)</th>
<th>(f_2)</th>
<th>(f_3)</th>
<th>(f_4)</th>
<th>(\sigma_B)</th>
<th>(\sigma_W)</th>
<th>(\sigma)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.162</td>
<td>0.240</td>
<td>0.105</td>
<td>0.570</td>
<td>-5.03 \times 10^{-2}</td>
<td>1.05 \times 10^{-4}</td>
<td>-5.44 \times 10^{-2}</td>
<td>0</td>
<td>0.172</td>
<td>0.290</td>
<td>0.337</td>
</tr>
</tbody>
</table>

For the mainshock (\(M_w = 6.1\)), the fault mechanism is thrust (reverse) type (ITACA, 2016). The fault projection on the ground surface allows for computing the Joyner-Boore distance of the MRN station, \(R_{jb} = 4.34\) km (Figure VI.5.18). It can be observed that Scortichino dyke is located inside the surface projection of the fault (\(R_{jb} = 0\) km), and an amplification of the recorded motion at MRN station is expected.
Chapter VI – Interpreting the deformation phenomena on Scortichino dyke

![Map of Scortichino dyke with fault projection](image)

*Figure VI.5.18 – Fault projection of the mainshock*

### Table VI.5.5 – Input coefficients for defining the GMPE of the three mainshocks and R distances for MRN station and Scortichino dyke.

<table>
<thead>
<tr>
<th>Event</th>
<th>Time (UTC)</th>
<th>M</th>
<th>Site class A (Fs)</th>
<th>Style of faulting</th>
<th>$F_{sof}$</th>
<th>R [km] MRN station</th>
<th>R [km] Scortichino c-c’ section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mainshock</td>
<td>02:03:53</td>
<td>6.1</td>
<td>0</td>
<td>R</td>
<td>$1.05 \cdot 10^{-1}$</td>
<td>4.34</td>
<td>0.00</td>
</tr>
<tr>
<td>I aftershock</td>
<td>02:06:26</td>
<td>4.8</td>
<td>0</td>
<td>U</td>
<td>0</td>
<td>8.53</td>
<td>15.20</td>
</tr>
<tr>
<td>II aftershock</td>
<td>02:06:28</td>
<td>5.0</td>
<td>0</td>
<td>U</td>
<td>0</td>
<td>16.50</td>
<td>5.96</td>
</tr>
</tbody>
</table>

Figure VI.5.19 reports with black lines the GMPEs of the three mainshocks, defined using the input parameters of Table VI.5.4. The same plots report also PGAs of the de-convoluted outcrop motions of the three shocks (red points).
The GMPEs have been scaled to the PGAs of the de-convoluted components. This latter have been used to define the values of PGA expected at the site of Scortichino.

Every de-convoluted motion (mainshock and aftershock separately) has been scaled to so-computed peak acceleration value; then, the three motions have been re-assembled in an unique record. In order to avoid aliasing during these merging operation, the complete time history has been once again baseline-corrected (Figure VI.5.20).
Chapter VI – Interpreting the deformation phenomena on Scortichino dyke

VI.3.3

Figure VI.5.20 - Input motion time history at Scortichino, May 20, 2012 mainshock and aftershocks

VI.5.4. Simulation of the 20th May events

Effective stress analyses have been performed using the geotechnical model of the c-c’ section of Scortichino dyke, already described in the previous sec. VI.5.3.

The results of the analyses are plotted in Figure VI.5.21 in terms of maximum acceleration, shear strain and shear stress profiles. The acceleration profile is characterized by a significant reduction of acceleration between the surface and the water table depth.

According to the time histories at 14.5 m depth, the mainshock triggers liquefaction, while the aftershocks are ineffective to generate excess pore water pressure, since the shear stress ratio is always under the threshold values (Figure VI.5.22).

Figure VI.5.21 - Coupled analysis on c – c’ section for the recorded EW seismic sequence
Effective stress analyses have also been performed along the d – d’ section of the dyke. Due to the lack of the shallow clay layer in the soil column (Figure VI.5.9), the consolidation properties of the soils are taken into account in the analysis. Permeability coefficients are the same already adopted for the different units (Figure VI.5.1) and a Poisson coefficient equals to 0.3 for all soil layers has been considered.

Figure VI.5.23 reports the results of the analyses in terms of maximum profiles of acceleration, shear strain and shear stress. In this case, liquefaction conditions are not reached and the maximum values of $r_u$ is equal to 0.68 at 19.7 m in the shallow sandy layer (A).

The acceleration profile is characterized by a significant reduction of acceleration between the surface and the base of the dyke, as observed also in the previous case. In both cases, this effect can be attributed to the liquefiable/degraded soils A200 and B, which work as dampers against the motion on the surface.

Figure VI.5.23 shows the time history of excess pore pressure at 15 m depth. It can be noted that the consolidation process starts at 42 s and continues for the rest of the time. Even though the dissipation process induces an increase of the soil stiffness, the shear stress due to the aftershock does not overcome the shear threshold value (dotted line in Figure VI.5.24) and, consequently, additional excess pore pressure is no longer generated.
Finally, a Newmark displacement analysis has been performed considering the simulated accelerogram at the depth of the critic surface failure, detected similarly to as described the previous paragraph VI.4.4. The critic acceleration has been computed equals to 0.175 g, adopting the Morgenstern – Price (1965) method. Figure VI.5.25 shows the result of the displacements analysis. As expected, accumulation of displacements occurs during the main seismic event, while the acceleration values are always lower than the critic acceleration in the aftershock.
The final accumulated displacement results slightly greater to 1.5 cm, which is compatible with the amplitude of the cracks observed along the crest of the dyke.

Figure VI.5.25 - Acceleration and displacement time histories from coupled analysis on d – d’ section

VI.6. References


VII. CONCLUSIONS AND PERSPECTIVES

VII.1. Conclusions

The main objective of this work was to develop and verify a simplified model for a reliable prediction of pore pressure build-up and dissipation induced by seismic loading in granular soils. This new model was developed and validated in several steps. First, it was searched for a simplified model easy to be applied in the engineering practice, yet able to catch the most significant aspects of the cyclic stress-strain behaviour and strength of potentially liquefiable soils. The model proposed by Park and Ahn (2013) was selected, since based on experimental data from cyclic laboratory tests and suitable to be implemented in a time domain seismic response analysis, avoiding the need to convert the irregular load history in an equivalent uniform cyclic loading. The original formulation was further developed, proposing updated functions for the cyclic resistance curve and the excess pore pressure relationship, as well as a straightforward procedure for the calibration of the model parameters.

Thereafter, the model so developed was implemented into an innovative computer code for the one-dimensional analysis of seismic soil response, originally written by Tropeano (2015). The code uses a nonlinear lumped mass approach, capable of simulating the seismic response of a horizontally layered soil profile, including stiffness degradation, shear strength, pore pressure build-up and dissipation.
The capabilities and limitations of the code were tested by applying it to several well-documented case histories. The study of the case histories provided considerable insight into the mechanics of the seismic response and associated pore pressure build-up in full-scale problems where liquefiable potentially soils are present, and/or there is the need of analysing post-seismic deformations and/or safety. The application of the code to four case histories suggests that the upgraded code encompasses the main aspects of the complex phenomenon of liquefaction. In particular, the code was verified to be capable to well-predict the initial triggering and occurrence of liquefaction, as well as the surface ground motion in terms of accelerograms and response spectra. Conversely, the inability of the constitutive model to simulate the phase transformation behaviour can be viewed as a limitation for reproducing the post-liquefaction soil response.

Finally, the developed method was applied to a very complex case history of a dyke damaged by the recent 2012 Emilia earthquake. The proposed approach allowed for deeply investigating on the nature of damage, disproving the contradictory results obtained by the traditional simplified approaches.

Once again, the present study confirms that the seismic response of potentially liquefiable soils is a complex task; it cannot realistically be approached from a single perspective, e.g. focusing only on peak ground acceleration or earthquake magnitude like in semi-empirical methods, but considering the soil deposit as a system. Since thirty years to date, the problem is that semi-empirical approaches cannot be supplanted, until a feasible alternative is adequately supported by field observations. In this context, the constitutive model and the computer code developed may be viewed as a good-working integrated balance between the limitations of semi-empirical methods and the challenges, sometimes prohibitive for the practicing engineer, implied by the use of advanced constitutive models.

Future perspectives of the research are primarily related to further refinements of the pore water pressure model, such as introducing a dependency of the pore water pressure parameters on the stress ratio.

Moreover, further validations on better instrumented full or model scale case studies could be useful to deeply investigate on the role of dissipation and redistribution of excess pore pressure in the surface and in-depth ground motion, as well as on the post-liquefaction deformation.

Finally, the implementation of the phase transformation behaviour is envisaged.
A. APPENDIX

A.1. Cyclic resistance curves based on field test data

It is reported the script file for generation of cyclic resistance curves from field data.

clear all

SPT=[6 8 10 12 14 15 16 18 20 22 24 25]; %corrected SPT N-value
% we suppose clean sand, so (N1)60cs=(N1)60
N=[3 4 5 10 15 20 40 60 100 200 300]; %number of cycles
sig=[50 100 150 200 400 800]; %effective confining stress (in kPa)

A(3:length(N)+2,1)=N'; %set the first column of the matrix results equal
to the number of cycles

for m=1:length(sig)
    A(1,(m-1)*length(SPT)+2:m*length(SPT)+1)=sig(m); %set the first row of
    the matrix results equal to the effective confining pressure
    sig(m) = sig(m)/101.3; %Convert from kPa to atm

    for p=1:length(SPT)
        A(2,(p+1)+(m-1)*length(SPT))=SPT(p); %set the second row of the
        matrix results equal to the number of corrected blow count
        CRR(p)= exp((SPT(p)/14.1)+((SPT(p)/126)^2)-
                     ((SPT(p)/23.6)^3)+((SPT(p)/25.4)^4)- 2.8) % equ. (2.25) by I&B 2014 with
        PL=0.5
        %effect of initial effective confining pressure through overburden
        correction factor
        Csig(p)= min(0.3,1/(18.9-2.55*sqrt(SPT(p)))) %equ. (2.16c) by I&B
        2014
        %for (N1)60<=37!!!
\[ K_{\text{sig}}(p) = \min(1.1, 1 - C_{\text{sig}}(p) \log(sigm(m))) \] %equ. (2.16a) by I&B 2014

%Magnitude scaling factor
\[ MSF_{\text{max}}(p) = \min(1.09 + (\text{SPT}(p)/31.5)^2, 2, 2) \] %equ. (2.21) by I&B 2014

\begin{verbatim}
for i=1:length(N)
    Nmin =((1/0.65)^((1/0.337))*(3/4)); %equ. (3-12) by I&B 2004
    b(i)=log(MSF_{\text{max}}(p))/log(15/Nmin) %N(Mw=7.5)=15 from I&B 2004
    MSF(i) = (15/N(i))^b(i);
    MSF(i) = min(MSF(i), MSF_{\text{max}}(p));
    M = -4*log(((MSF(i) - 1)/(MSF_{\text{max}}(p) - 1) + 1.325)/8.64);
    CSR(i) = CRR(p)*MSF(i)*K_{\text{sig}}(p) %eq. (2.7) by I&B 2014
    \tau_{\text{sig}}(i) = CSR(i)/0.65 %eq.(2.1) by I&B 2014
    \tau_{\text{sig}}(i) = CSR(i)
end
\end{verbatim}

\[ \tau_{\text{sig}} = \tau_{\text{sig}}' ; \]
\[ A(3:length(N)+2,(p+1)+(m-1)*length(SPT)) = \tau_{\text{sig}} ; \] %put the result in the column of the matrix results

\end{verbatim}

A
xlswrite('CRC', A)
B. APPENDIX

B.1. Database of experimental excess pore pressure ratio curves

All the experimental data have been collected in the following database, together with the results of the regression analysis. Each record is defined by an identification number (ID), grain size distribution, i.e. percentage of gravel (G), sand (S), silt (L), clay (A), plasticity index (PI), relative density (Dr), effective confining pressure applied ($\sigma'_0$), cyclic resistance ratio (CSR), type of test, i.e. cyclic triaxial test (CTX) or cyclic shear test (DSS), type of specimen, i.e. reconstituted (Rec) or undisturbed (Und) material. Reference of the data is also reported. Experimental curve that reach $ru=0.95$ have been considered for the regression analysis (Liq= Y), otherwise the curves have been neglected (Liq = N). Finally, the pore pressure coefficients, $a$, $b$, $c$, are reported together with the related value of corrected determination coefficient.
<table>
<thead>
<tr>
<th>ID</th>
<th>Name</th>
<th>%G</th>
<th>%S</th>
<th>%L</th>
<th>%A</th>
<th>Dr (%)</th>
<th>(\sigma'_0) [kPa]</th>
<th>CSR</th>
<th>Test type</th>
<th>Spec</th>
<th>Reference</th>
<th>Reference</th>
<th>Liq</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>R(^2) adj</th>
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</thead>
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<td>1</td>
<td>Adapazaru Silt_00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>CTX</td>
<td>Rec</td>
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<td>0.1944</td>
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<td>-</td>
<td>-</td>
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<td>CTX</td>
<td>Rec</td>
<td>Pekcan et al. (2004)</td>
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<td>Rec</td>
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<td>-</td>
<td>-</td>
<td>-</td>
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<td>Rec</td>
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<td>8</td>
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<td>-</td>
<td>-</td>
<td>CTX</td>
<td>Rec</td>
<td>Pekcan et al. (2004)</td>
<td>Y</td>
<td>-</td>
<td>-</td>
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<td>(3/2)</td>
<td>-</td>
<td>-</td>
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<td>Rec</td>
<td>Pekcan et al. (2004)</td>
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<td>CTX</td>
<td>Rec</td>
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<td>-</td>
<td>-</td>
<td>CTX</td>
<td>Rec</td>
<td>Pekcan et al. (2004)</td>
<td>Y</td>
<td>-</td>
<td>-</td>
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<td>0.3</td>
<td>CTX</td>
<td>Rec</td>
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<td>0.48</td>
<td>CIU DSS</td>
<td>Rec</td>
<td></td>
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<td>Rec</td>
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<td>Rec</td>
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<td>Rec</td>
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<td>CTX</td>
<td>Rec</td>
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<td>CTX</td>
<td>Rec</td>
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<td>0.4191</td>
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<td>0.9998 7</td>
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<td>0</td>
<td>0</td>
<td>130</td>
<td>0.2</td>
<td>CIU</td>
<td>DSS Und Porcino (2013)</td>
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<td>1.013</td>
<td>0.5254</td>
<td>-0.0611</td>
<td>0.9983</td>
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<tr>
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<td>33</td>
<td>7</td>
<td>0</td>
<td>0</td>
<td>130</td>
<td>0.2</td>
<td>CIU</td>
<td>DSS Und Porcino (2013)</td>
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Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

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Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

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Anna Chiaradonna

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## Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

Anna Chiaradonna

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### Development and assessment of a numerical model for non-linear coupled analysis on seismic response of liquefiable soils

Anna Chiaradonna

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