AN OVERVIEW ON THE NONLINEAR BEHAVIOR OF INTERMEDIATE SOILS UNDER SEISMIC CONDITIONS

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Abstract: *Risk-sensitive areas are predominantly located in a wide variety of natural depositional environments, such as alluvial plains, where intermediate soils (silty sands, silts, and sandy silts) are commonly found. Additionally, intermediate soils are common in man-made geotechnical structures such as riverbanks, hydraulic fills, dredging sediments and mine tailings. Due to the complexity of their behavior and consequent variability of their main physical and mechanical properties, the geotechnical modelling of intermediate soils is still relatively poorly understood. Existing characterization and interpretation approaches, typically developed for markedly cohesive-behavior soils (clays) or cohesionless-behavior soils (sands), show severe limitations for geotechnical engineering applications that involve intermediate soils under dynamic conditions. Most of the recent studies, both experimental and theoretical, have been carried out on clean and reconstituted sands, while considerable effort is still devoted to understanding the behavior of natural soils with different grading and plasticity. An emblematic case history that revealed this need is, for instance, the city of Adapazari (Turkey), where liquefaction was observed after the 1999 Kocaeli earthquake of magnitude* $M_W = 7.4$*. Hundreds of buildings settled as much as 1.5 m, or tilted, in part, due to liquefaction-induced softening and shear strength loss of the foundation soils. Punching of buildings into cyclically softened foundation soils caused occasional bulging of sidewalks, while lateral translations of buildings were observed in the same cases. The soils investigated in Adapazari consist of silt and clay of variable proportions, with cyclic triaxial tests correlating the results with the observed ground failure-induced damage. Moreover, the classification of liquefiable soils based on current criteria does not successfully identify liquefaction triggering, highlighting the peculiarities of these challenging soils and the need for further studies. This paper proposes an overview of the nonlinear behavior of intermediate soils under seismic conditions, which is based on the most recent literature on this demanding topic. The outcomes of this collection are used to discuss the adequacy of currently available methods and testing techniques for the characterization and modelling of intermediate soils and define deficiencies where the research should concentrate in the next near future.*

1. Introduction

The intermediate soils are silts and many other types of soil mixtures (silty, sands, sandy silts, clayey sands) which are typically characterized by values of permeability falling in the so-called intermediate range.

Consequently, the idealized assumption of a sharp distinction between drained (sands) and undrained (clays) conditions is no longer applicable during in-situ experimental testing and data interpretation. A further, still unsolved issue with intermediate soils concerns their response during earthquakes. Recent events, among them the 1999 Kocaeli earthquake in Turkey and the 2012 earthquake in Emilia-Romagna, demonstrated that significant strains and strength losses can occur also in saturated silty sands and silts (Sancio et al. 2004, Tonni et al. 2015, Maurer et al. 2015).

Several experimental studies, both in the laboratory and in the field, have investigated the fines-content effects on liquefaction hazard evaluation (e.g., Porcino and Diano 2016, Amoroso et al. 2020, 2022). Remarkable differences in the stress-strength-strain and pore pressure responses have been observed, mainly due to the combined effect of soil grading characteristics and type of fabric, together with fine mineralogy, plasticity, activity, and particle shape (Polito and Martin 2001, Mitchell and Soga 2005, Boulanger and Idriss 2006). All these factors, affecting the transition from clay-like to sand-like response, lead to a complex framework that is still far from being exhaustively studied and understood. Additionally, approaches based on the cone/piezocone penetration test (CPT/CPTU) and the flat dilatometer test (DMT) typically used to assess liquefaction susceptibility have shown some limitations when applied to these soils (Porcino et al. 2019).

This paper aims to provide an overview of the most recent findings on the behavior of intermediate soils reported in literature, with special attention to the liquefaction problems. The following Section 2 briefly summarizes the observed damage pattern in Adapazari city (Turkey) after the 1999 Kocaeli earthquake, which represents an exemplificative case history to understand the importance of studying the intermediate soils under seismic conditions. Then, Section 3 collects some of most recent studies on the soil behavior of intermediate deposits from the small to the high strain levels until failure conditions. Section 4 reports one of the cutting-edge interpretations of the mechanisms behind the observed soil response as obtained from laboratory investigations, while Section 5 concentrates on the most recent findings coming from in-situ tests gained after the 2012 Emilia earthquake (Italy). The conclusions show the limitations of the traditional approaches and open the discussion about the necessary steps to carry out in future research.

2. Earthquake-induced damage in intermediate soils: the case study of Adapazari after the 1999 Kocaeli earthquake

The phenomena observed in Adapazari are of particular interest because of the fine-grained nature of the soil deposits that underwent ground failure. Indeed, Adapazari city (Sakarya Province, Turkey) lies in a fertile plain formed by the fluvial activity of the Sakarya and Cark rivers, which frequently flooded the area until flood control dams were built recently. Sands accumulated along bends of the meandering rivers, and the rivers flooded periodically leaving behind predominantly non-plastic silts, silty sands, and clays throughout the city (Sancio et al. 2002).

The city was densely developed in most areas, primarily with 3–5 storey reinforced concrete frame buildings and older 1–2 storey timber/brick buildings. Reinforced concrete construction is primarily non-ductile, with shallow, reinforced concrete stiff mat foundations located at depths of typically 1.5 m due to shallow groundwater.

After the 1999 Kocaeli (Turkey) earthquake with moment magnitude M_W = 7.4, severe damage to hundreds of structures and lifelines were observed in the City of Adapazari. Sancio et al. (2002) reported that the 27% of the building stock were severely damaged or destroyed. Many reinforced concrete buildings, generally of 3–5 storeys, penetrated the surrounding ground or tilted due in part to liquefaction and ground softening; some of them also had significant structural damage. As detailed in Figure 1, some buildings experienced non-uniform vertical deformation, causing the building to be condemned albeit devoid of structural damage (Figure 1a), while toppling of buildings (Figure 2b) was typically observed in laterally unconstrained slender buildings, i.e., large ratio of building height (H) to its width (B). Uniform vertical displacements (e.g., 30 cm) and purely lateral translation (e.g., about 100 cm), i.e., buildings translated laterally over liquefied soil directly beneath their foundation, were also observed (Sancio et al. 2004).

(a) (b)

Figure 1. Examples of damages in Adapazari after the 1999 earthquake: (a) vertical displacements with significant tilt (H/B=1.7) and (b) bearing capacity failure (H/B=3.2) (modified after Sancio et al. 2004).

3. Mechanical behavior of intermediate soils

This section is divided into three subsections where the main key points in the definition of the mechanical behavior of intermediate soils under cyclic and dynamic loadings are discussed.

3.1. Susceptibility to earthquake-induced soil liquefaction

According to Seed et al. (2003), soils with limit liquid LL < 37 and plasticity index PI < 12 are potentially liquefiable, and those with 37 < LL < 47 and 12 < PI < 20 require laboratory testing. Based on the experience gained from cyclic triaxial tests on Adapazari soils (Turkey), Bray and Sancio (2006) suggested to consider both the soil plasticity index and the ratio of the water content (wc) to the liquid limit (LL) as reliable criteria to assess liquefaction susceptibility. The key role of the wc/LL parameter has been emphasized, by observing that fine-grained soils with high wc/LL ratios (e.g., > 1) are prime candidates for liquefaction, especially in low plasticity sediments. The previous described criteria do not actually provide explicit recommendations for selecting the most appropriate procedures to evaluate the loss of strength and development of strains during cyclic loading. In this context, recommendations by Boulanger and Idriss (2006) suggested that PI can be used to discriminate between two categories: 1) "sand-like" fine-grained soils (PI < 7), which are liquefaction susceptible and thus worthy to be evaluated using liquefaction correlations based on detailed laboratory and in situ testing; 2) "clay-like" fine-grained soils (PI ≥ 7) that can undergo cyclic failure and should be evaluated using cyclic softening procedures. A transition zone between these two categories (4 < PI < 7) ("intermediate behavior") can be identified (Figure 2a).

(a) (b)

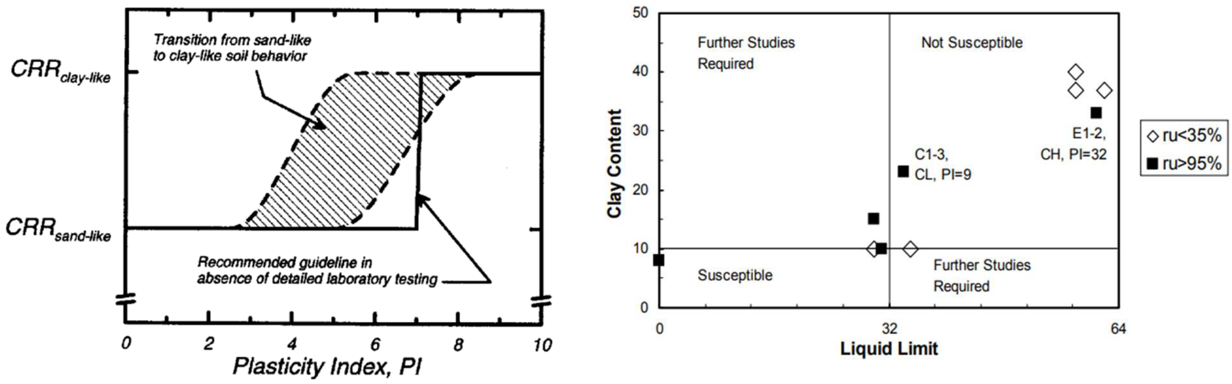

Figure 2. (a) Schematic transition from sand-like to clay-like behavior for fine-grained soils with increasing PI, and the recommended guideline for practice (modified after Boulanger and Idriss 2006); (b) Liquefaction susceptibility chart of silty and clayey sands applied to Adapazari soil (modified after Pekcan et al. 2004).

Pekcan et al. (2004) carried out a series of cyclic triaxial tests on samples retrieved from five selected soil sites in Adapazari, where buildings founded on these soils exhibited significant foundation displacements (above Section 2). Based on the tests results, the same authors concluded that mixtures of low plasticity silts and clays can be cyclically liquefied based on either 100% pore pressure generation or 5% double amplitude axial strain criteria. Moreover, the application of the criteria for liquefaction of silty soils, known as 'Modified Chinese Criteria', as proposed by Andrews and Martin (2000), shows how those criteria are not fully capable of correctly characterizing the liquefaction triggering of the Adapazari soils (Figure 2b).

3.2. Cyclic strength against liquefaction

It is well known that the cyclic resistance of soil is strongly influenced by constitutive factors, such as fines content and plasticity, and state variables typically, density and confining pressure. In the following, the effects of the vertical effective overburden stress, σ'_{v0}, and of the fines content, FC, are specifically considered.

The increase of vertical effective overburden stress, σ'_{v0} in cohesionless soils leads to the suppression of dilation and the corresponding reduction in undrained cyclic resistance (Idriss and Boulanger 2008). The understanding of the effect of σ'_{v0} on the cyclic response of natural silts remains poor in comparison (Dadashiserej et al. 2023a,b). Recently, Dadashiserej et al. (2023a) focused on assessing the effects of σ'_{v0} on the cyclic response of several silts obtained from the Willamette Valley in Oregon (US). These silts vary from non-plastic to slightly plastic. Constant-volume, stress-controlled cyclic direct simple shear tests (CDSS) were performed on both intact and reconstituted specimens to assess the influence of soil fabric on material response. Dadashiserej et al. (2023a) mainly found out that specimens exhibited a reduction in cyclic resistance with increases in σ'_{v0}, despite the accompanying increase in density (i.e., reduction in void ratio). Moreover, the authors found that the overburden correction factor (commonly used for addressing the effects of overburden stress) depends on the plasticity index (PI) with a greater reduction with σ'_{v0} for moderately plastic silts relative to non-plastic to low-plasticity silts, highlighting the role of compressibility on the cyclic resistance of silt.

The role of non-plastic fines on liquefaction susceptibility is still a controversial matter. Recent studies agree on the existence of a threshold value of non-plastic fines content below which the liquefaction susceptibility increases with the FC and above which the cyclic resistance decreases with FC. This experimental evidence can be explained in the framework suggested by Thevanayagam et al. (2002) that identifies a threshold fines content determining the transition from a sand-like to a silt-like behavior; this threshold is related to the fabric of the mixture. As FC increases, fines tend to occupy the space among grains, which remain in contact each other. Up to the FC at which the pores are completely filled with fine particles, these latter do not contribute to the transmission of inter-particle forces. A further increase of FC leads the fines to push apart the grains and to participate to the transfer of contact stresses (Papadopoulou and Tika 2008).

More recently, Gobbi et al. (2022) assessed of the influence of non-plastic fines and mixture-packing conditions on liquefaction triggering through a series of monotonic and cyclic consolidated undrained triaxial and resonant column tests on reconstituted soil specimens by using different fines contents and confining pressures. The authors found that the mixtures with different fines contents show different mechanical behavior even in the case of same relative density. The effect of fines can lead to an increase or decrease of the number of cycles for a given cyclic stress ratio (Figure 3). The liquefaction resistance increases with increasing fines content up to 5% and then decreases with increasing fines content up to 20%. In addition, soils with high relative density show more remarkable variations from the liquefaction resistance of clean sand, and the range of variation of the number of cycles, N_L for a given cyclic stress ratio, CSR is larger (Figure 3b).

The effects of plastic fines have a different impact on liquefaction susceptibility respect to the non-plastic fines, which is clearer to identify. El Mohtar et al. (2014) report the cyclic resistance of various sands treated by adding different percentage of bentonite and this is the cause of an improvement of the cyclic strength, also for low content of additive. Based on the same consideration, the semi-empirical charts for soil liquefaction assessment based on in-situ test results (CPT-based, SPT-based and so on) account for the presence of plastic fines by fictitiously increasing the cyclic resistance ratio of soils (Idriss and Boulanger 2008).

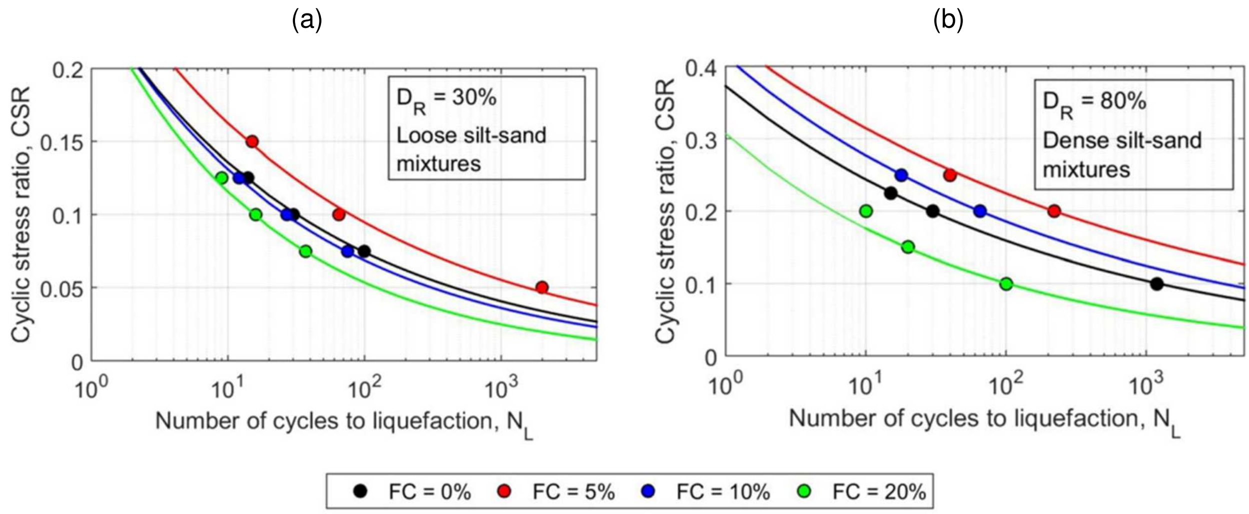

Figure 3. Liquefaction potential curves obtained for different fines contents and for (a) loose and (b) dense silty sand mixtures at 100 kPa confining pressure (modified after Gobbi et al. 2022).

3.3. Nonlinear and dissipative behavior

Dadashiserej et al. (2023b) investigated the response of a low plasticity silt deposit at the Port of Longview (WA, US) over strain amplitudes ranging from linear elastic to nonlinear inelastic from in-situ (T-Rex) and element-scale laboratory tests. The authors compared the estimated threshold shear strain for excess pore pressure generation, γ_{tp}, from in-situ, cyclic direct simple shear (CDSS) and torsional shear (TS) tests to those reported by Mortezaie and Vucetic (2016) for reconstituted kaolinite, kaolinite and bentonite mixtures, and natural plastic soils with PIs ranging from 14 to 55 and by Jana and Stuedlein (2022) from CDSS tests conducted on intact, high plasticity silt (MH; PI = 28) specimens. The results indicate a clear, though somewhat scattered, trend of increasing γ_{tp} with PI (Figure 4a).

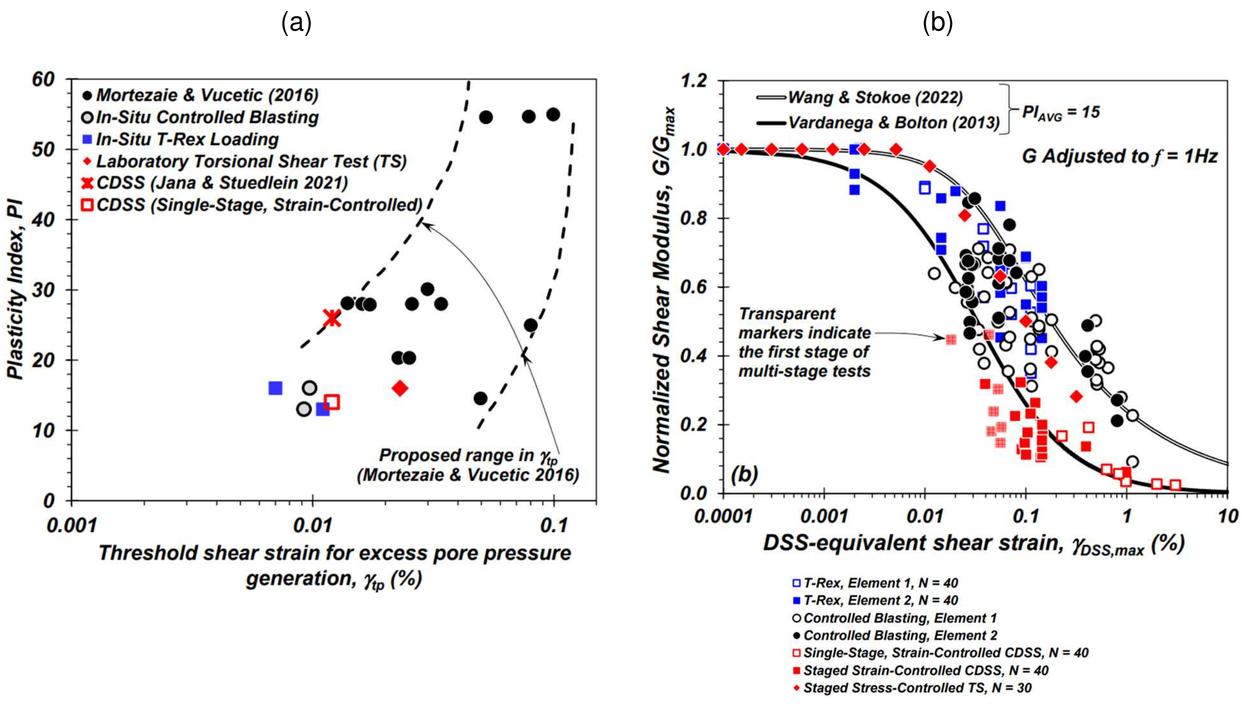

Figure 4. (a) Variation of threshold shear strain for excess pore pressure generation with plasticity index, PI; (b) Normalized shear modulus, G/G_{max}, with maximum DSS-equivalent shear strain, $\gamma_{DSS,max}$ (modified after Dadashiserej et al. 2023b).

The authors also presented the collected data in terms of shear modulus reduction curves, G/G_{max}, for an average PI = 15 and compared them with the curves proposed by Vardanega and Bolton (2013) adjusted to f = 1 Hz and Wang and Stokoe (2022) with a confining pressure equal to 30 kPa (Figure 4b). The difference between the G/G_{max} curves by Vardanega and Bolton (2013) and Wang and Stokoe (2022) is large and would increase if adjusted to f = 1 Hz. The variation of G/G_{max} obtained from the TS tests over the linear and nonlinear elastic regime is very well-captured by the Wang and Stokoe (2022) curve but exhibits a softer and stiffer response for $\gamma_{DSS,max} > \gamma_{tp}$. On the other hand, G/G_{max} determined from the strain-controlled CDSS tests is bounded at the upper range of test results by the Vardanega and Bolton (2013) curve, with good agreement for $\gamma_{DSS,max} > 0.15\%$.

With reference to the effects of the fines content, Gobbi et al. (2022) observed in resonant column tests that silty sands with high fines content FC demonstrate lower stiffness for a given shear strain (Figure 5a). The experimental values are within the lower and upper bounds of the analytical normalized shear modulus reduction curve proposed by Oztoprak and Bolton (2013) up to a shear strain 5 $*10^{-2}$ % and then, they fall into the lower zone. Soils with high fine content FC show higher damping (Figure 5b). The bounds suggested by Rollins et al. (1998) for quartz soils describe reasonably well the response of the investigated HN31 Hostun-RF sands without or with limited quantity of non-plastic fines, i.e., FC < 20%.

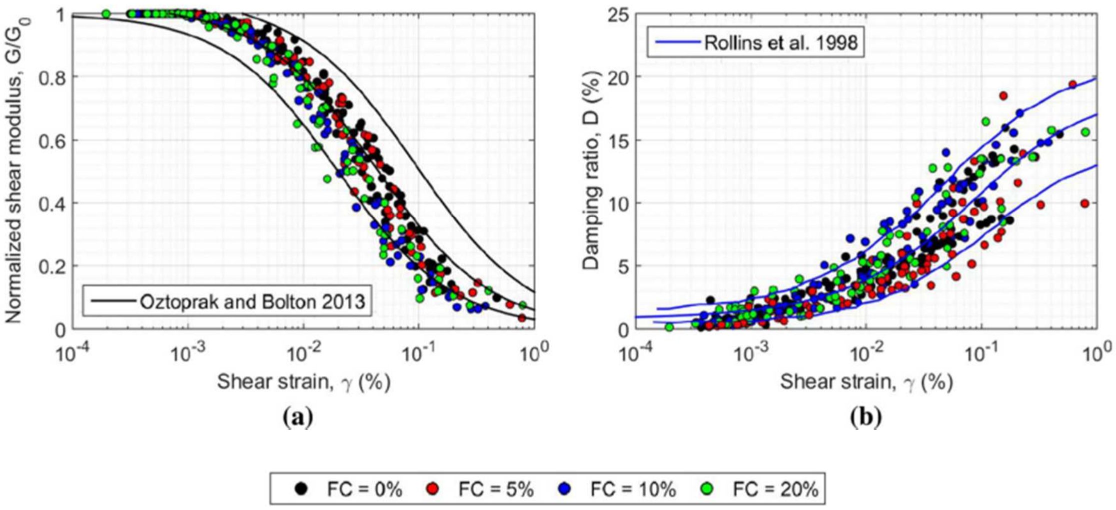

Figure 5. HN31 Hostun-RF sand without or with limited quantity of non-plastic fines: (a) normalized shear modulus and (b) damping ratio (modified after Gobbi et al. 2022).

4. Intergranular mechanism recently assessed by laboratory tests

From a micro-mechanical perspective, three different types of intergranular contacts influence the mechanical behavior of sand with fines: the contact between particles of coarse-grained sand, sand–fine sand contact and sand–fine–fine sand contact. The contact between the coarse particles is the strongest, while the sand–fine–fine sand contact is the weakest. The type of contact in the soil structure is the main factor influencing the behavior of silty sands. Considering two mixtures with the same relative density, the one with the lower fines content presents a stronger structure than the one with the higher fines content, because it is mostly characterized by contacts between coarse particles. When the mixture contains a higher fines content, the behavior becomes mostly characterized by sand-fine-sand contact and sand-fine-fine-sand contact (Gobbi et al. 2022).

To quantify the above concept, the equivalent intergranular void ratio has been defined as (Thevanayagam et al. 2000):

$$e_g^* = \frac{e+(1-b)FC}{1-(1-b)FC} \quad (1)$$

where b varies from 0 to 1 and represents the fine fraction participating to the force transfer. If b = 1, all the fine particles contribute to the force transfer with the sand and the equivalent intergranular void ratio corresponds to the global void ratio e. Similarly, for clean sand FC = 0% and $e_g^* = e$.

Equation (1) is applicable only when the mechanical behavior of the mixture is sand-dominated. Soils are considered sand-dominated when the fines content FC is lower than a threshold FC_{th} (Thevanayagam 1998), which represents the transition from sand-dominant behavior to fines dominant behavior.

Gobbi et al. (2022) found that the equivalent intergranular void ratio is the key parameter to characterize the silty sand mixtures because a single correlation curve, independent of fines content, can be used to represent the mean effective stress at the steady state, liquefaction resistance and shear modulus (Figure 6). It means that data obtained for clean sand are reliable for predicting the behavior of mixtures, independently of the fines content. The same authors proposed an original formula to estimate the active fine fraction b:

$$b = \frac{e_{max} - e_{min}}{1 + e/FC_{th}} \qquad (2)$$

where e is the global void ratio of the mixture after consolidation, FC_{th} is the threshold fines content and e_{max} and e_{min} are the maximum and minimum void ratios, respectively. In Equation (2), FC_{th} is expressed in decimals. The numerator outlines the packing configuration and is dependent on the fines content. The particle size and shape (round or angular) also strongly influence the intergranular mechanism.

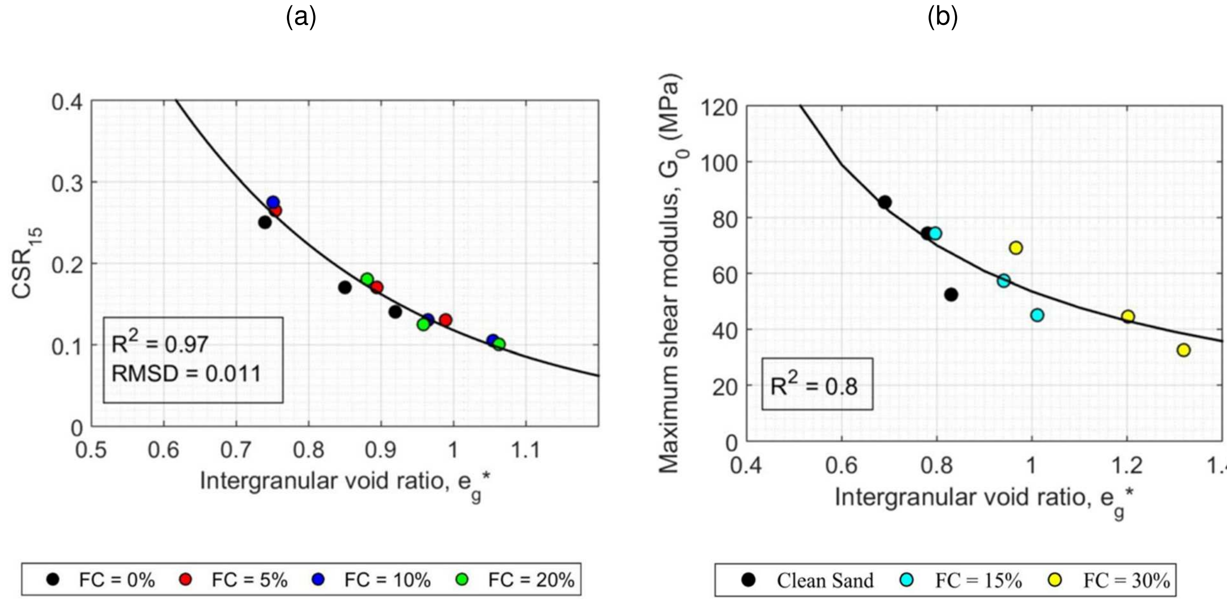

Figure 6. (a) Cyclic strength CSR_{15} and (b) maximum shear modulus with respect to the equivalent intergranular void ratio (modified after Gobbi et al. 2022).

5. Intermediate behavior evaluated through in-situ tests: the case of the Emilia-Romagna silty soils

The following findings involve the intermediate sandy silts with low plasticity fines located in Emilia Romagna, Italy, where a M_W = 6.1 earthquake in 2012 triggered ground deformations and cracks.

Monaco et al. (2021) investigated a site located near Bondeno (Ferrara, Italy). The soil profile basically includes an upper unit, about 9 m thick, composed of clays and silts followed by sandy silts, and a lower unit consisting of medium-grained sands and silty sands, about 20 m thick.

The application to CPTU data of the classification method developed by Schneider et al. (2008) reveals a pronounced intermediate nature of the sediments between 0.8 and 3.6 m in depth. Figure 7 shows indeed that most of the experimental points fall in the domains of silts (1a) and transitional soils (3), the latter including a wide variety of soil mixtures (i.e., clayey sands, silty sands, silty sands with clay, clayey sands with silt). In these soils, partial drainage is very likely to occur during cone penetration.

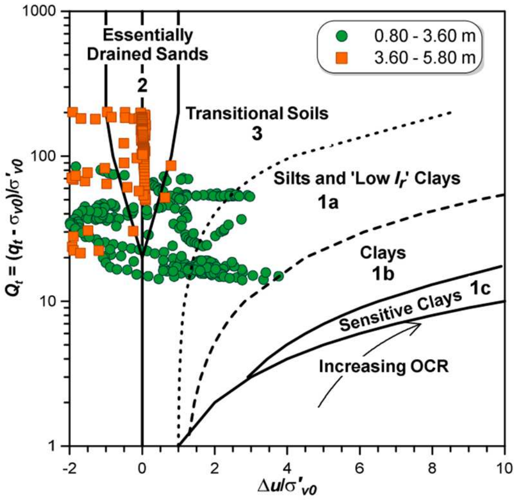

Figure 7. Soil classification using the Schneider et al. (2008) chart (modified after Monaco et al. 2021).

Monaco et al. (2021) performed a series of tests with the Medusa DMT (fully automated dilatometer) both by the standard procedure and by varying penetration rate, membrane inflation rate and time delay before membrane expansion. In particular, the variable-rate Medusa DMT tests were executed adopting variable penetration rates (slower and faster than standard), combined with variable pressurization rates (slower and faster than standard) achieved by regulating different time-for-reading intervals after stopping the blade at the test depth.

Figure 8 summarizes the combined effects of both variable penetration rate and variable pressurization rate. The results obtained from the reference standard DMT 6 (standard penetration rate 2 cm/s, standard pressurization rate with A-reading 15 s after stop, B-reading 15 s after A) are compared with the "slowest" DMT 3 (slow penetration rate 0.2 cm/s, slow pressurization rate with A-reading 30 s after stop, B-reading 30 s after A) and the "fastest" DMT 5 (fast penetration rate 6 cm/s, fast pressurization rate with A-reading 7.5 s after stop, B-reading 7.5 s after A). Figure 8 shows that the slower the penetration and pressurization rate, the more "drained" the test, with lower values of the pore pressure index U_D (moving to the left towards the "fully drained" U_D = 0 vertical line); accordingly, the material index I_D moves to the right towards the "sand" region. On the other hand, the faster the penetration and pressurization rate, the more "undrained" the test, with higher U_D (moving to the right away from the "fully drained" U_D= 0 vertical line); accordingly, I_D moves to the left towards the "clay" region. This finding reinforces the use of the DMT pore pressure index U_D to discern between drained, undrained or partially drained soil behavior. Additional support is provided to viewing the material index I_D, introduced as an indicator of soil type (clay, silt, sand), as a parameter which broadly reflects some "soil behavior type", including "sand-like" or "clay-like" behavior of intermediate soils.

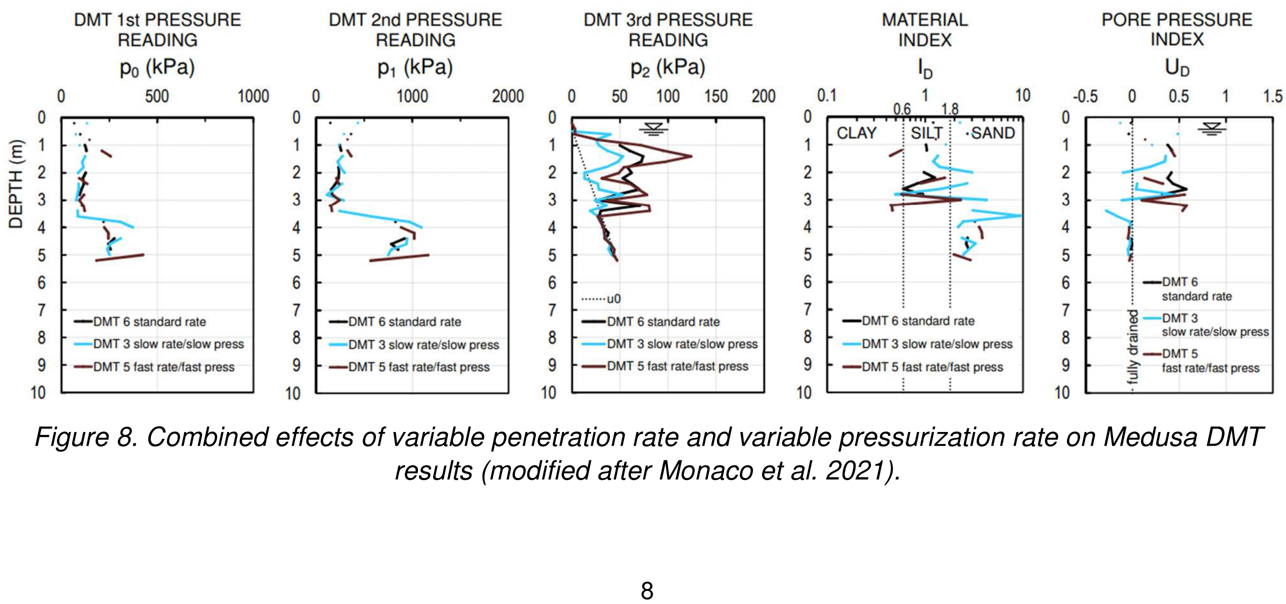

Figure 8. Combined effects of variable penetration rate and variable pressurization rate on Medusa DMT results (modified after Monaco et al. 2021).

6. Discussion and conclusions

This paper provides a tentative summary of the most recent findings on the mechanical behavior of intermediate soils under seismic actions, with special attention to the soil nonlinearity until liquefaction. It can be observed that little guidance is still available on how to tackle intermediate soils. Different findings seem to be reported by several research groups especially with reference to laboratory investigations on the effects of the fines content and liquefaction susceptibility. The interpretation based on the equivalent intergranular void ratio seem a promising path for unifying in the same framework the mechanical behavior of these soils. However, effective guidelines for a speed calculation of the intergranular void ratios are still missing, as well as how to transfer this concept to the interpretation of the in-situ tests. The collection of a large database of data from different representative test sites and the cross-checking of the results obtained with different approaches based on laboratory and in-situ tests are required to move forward the characterization and modelling of the intermediate soils.

7. Acknowledgements

This work has been supported by the Research Project of National Interest ‘TRANSITION – characTeRization And modelling of iNtermediate Soils through Innovative approaches for robust design of straTegic Infrastructures and protection Of risk-seNsitive areas’ funded by the Italian Ministry of University and Research (PRIN 2022 – Project 2022PZW7FL – Grant E53D23003630006).

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