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Application of colloidal silica grout to stabilize against liquefaction the foundation soil of an existing school building

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ABSTRACT: Chemical grout using a colloidal solution of sodium silicate has been selected to stabilize against liquefaction the foundation soil of a school building located in the town of Cesenatico, Italy. The building is located only few hundred meters from the shore line. The subsoil consists of about 8.5 m of sand and silty sand followed by a 15 m thick layer of silty clay. The ground water table is located at a depth of 1.5 m from the ground surface. In this paper the results of laboratory tests carried out to verify the efficacy of the colloidal solution in increasing the soil resistance against liquefaction are presented.

1 INTRODUCTION

Colloidal solutions are aqueous dispersions of sodium silicate and a reagent solution which change viscosity over time to produce a gel. These solutions have an initial viscosity very close to the water viscosity, so they can be used to permeate the soil. The gel time can be controlled adjusting the concentration of the reagent. Once hardened, the gel stabilizes liquefiable soils by lightly cementing individual grains and reducing the hydraulic conductivity of the deposit. The use of colloidal solutions to mitigate the liquefaction susceptibility of sandy soils has been investigated over the past 20 years by many researchers (Gallagher and Mitchell 2002, Tsukamoto et al. 2006, Porcino et al. 2012). At open sites a wide range of methods can be employed to reduce the liquefaction risk, including densification and drainage techniques. At developed sites, where it may be difficult or impossible to apply conventional remediation technique, the use of colloidal solution can be an advantageous method.

Chemical grout using a sodium silicate solution has been selected as the more suitable technique to stabilize against liquefaction the foundation soil of a school building located in the town of Cesenatico (FC), Italy. The building (which houses about 600 students and will be operating during the remediation works) rises only a few hundred meters from the shore line; its subsoil consists of 8.5 m of sand and silty sand, followed by a 15 m thick layer of silty clay. The ground water table GWT is located at a depth of 1.5 m from the ground surface. The design earthquake magnitude and peak horizontal acceleration are M = 6.14 and a_{max} = 0.289g, respectively. The liquefaction risk of the sandy deposit was estimated using both a CPT based method and the results of undrained cyclic triaxial tests and resulted high, according to the so called Iwasaki criterion (Iwasaki et al. 1978). Permeation grouting using a colloidal solution has been considered the most advantageous mitigation technique for the school subsoil since it will allow to stabilize the sandy deposits against liquefaction avoiding soil disturbance and associated movement of the existing foundations. These paper presents the results of undrained cyclic triaxial tests (CTX) carried out on sandy samples, reconstituted at the site void ratio and permeated using a sodium silicate solution. The tests indicated a very satisfactory increase of the soil liquefaction resistance and allowed to design a trial test which will be performed on site to verify the laboratory results and to calibrate the grout injection procedure (e.g. volume, pressure, rates) and the grout mixing process (e.g. concentration, setting time).

2 DESCRIPTION OF SCHOOL BUILDING AND RETROFITTING SYSTEM

The Enzo Ferrari school complex in Cesenatico consists of a total of six buildings which planimetrically form a large L shape. The building indicated as A in Figure 1 will be object of seismic retrofitting. It is a four story structure on shallow foundations, with a T-shaped plant of about 911 square meters. The retrofitting project consists in the insertion of three external steel towers, equipped with dissipative devices (Roia et al. 2013, Gioiella et al. 2013) at the head of each of the three wings that make up the T planimetric configuration. The towers (TA, TB and TC in Figure 1) present the same elevation of the wings; at the base they are supported by a central spherical hinge and are connected at the perimeter to viscous dampers. The towers foundations consist in12 bored piles, 18 m long, 0.8 m in diameter. The dissipative towers are arranged in-plan so as to regularize the modal response of the building, i.e. limiting possible torsional couplings. Furthermore, they contribute to regularize the inter story drift of the building, which is fundamental for the damage limit state. The main advantages of this innovative retrofitting system are:

- the towers are external to the building and do not interfere with the operability of the structure, thus eliminating the indirect costs related to the arrangement of internal spaces, interruption or relocation of the activities;
- it ensures the Immediate Occupancy Limit state also for high intensity earthquake.
  In parallel with the building retrofitting intervention, also the school and towers foundation soil will be object of a consolidation work in order to reduce the susceptibility to liquefaction of the whole area and to ensure the efficacy of the structural retrofitting project.

3 SITE GEOTECHNICAL CONDITIONS

The town of Cesenatico is locatedin the Central Adriatic coastal plain (the south-eastern edge of the Po Valley). The outcropping soil deposits consist of beach-ridge sands about 10 m thick of late Pleistocene-Holocene age, followed, between 10 e 220 m of depth, by a sequence of

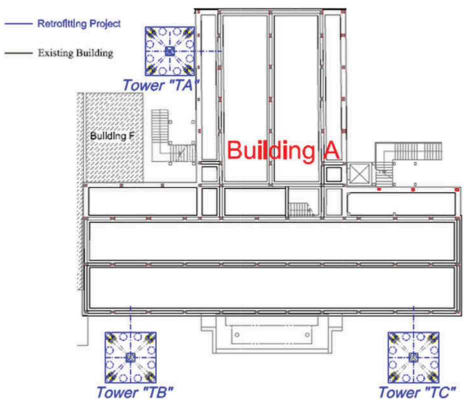

Figure 1. Plan arrangement of dissipative towers.

mainly fine grained soil layers, originated during the Pleistocene medium and Holocene by repeated alternations of coastal and alluvial deposits (known as Emilia Romagna Super synthem, RER-EniAgip 1998, Boccaletti et al. 2004). The school building subsoil was investigated through the following in situ tests:2 piezocone penetration tests (CPTU1 and CPTU3) and 2 seismic CPTs (SCPTU2 and SCPTU4), from 23 m to 27 m deep; 14 dissipation tests; two continuous core boreholes(S1 and S2), 30 m and 40 m deep, during which 12 undisturbed samples were retrieved; variable-head field permeability tests. The undisturbed samples were subjected to classification tests, monotonic undrained triaxial tests, resonant column tests; variable head permeability tests. Undrained cyclic traixial tests were also carried out on reconstituted specimens. The site is practically flat and on the base of the tests carried out the following horizontally layered model was assumed. From the ground surface to a depth z = 8.5 m, a sandy layer (Unit A) is present, which can be subdivided into two subunits: from 0 to 6 m of depth subunit A1, made of fine sand, with a non-plastic fine content FC = 5%; from 6 to 8.5 m of depth subunit A2, made of silty sand with a non-plastic FC = 15-20%. From 8 to 26 m of depth unit B, made of high plasticity silty clay; unit C from 26 to 30 m of depth, made of medium and fine sand and silty sand; from 30 to 40 m of depth unit D, made of clay and silty clay. The ground water table was located at z_w = 1.5 m from the ground surface. Figure 2 shows the soil profile deduced from the cone resistance measures and boreholes logs. Figure 3 reports the grain size curves measured on specimens sampled from the units A1, A2 and B.

The in situ and laboratory tests carried out allowed to evaluate the shearing resistance angle at critical state φ'_{cs} and the relative density D_R of units A1 and A2. The A1 sand is characterized by φ'_{cs} = 36° and D_R = 60-70%; the A2 silty sand is characterized by φ'_{cs} = 32° and D_R = 45-50%.

The liquefaction resistance of the sandy layers A1 and A2 was estimated in first approximation using the CPT-based Idriss and Boulanger (2008) approach. The input parameters adopted for the assessment are: earthquake magnitude and maximum horizontal acceleration, M = 6.14 and a_{max} = 0.289g, respectively. The liquefaction potential index (Iwasaki et al. 1978) resulted high

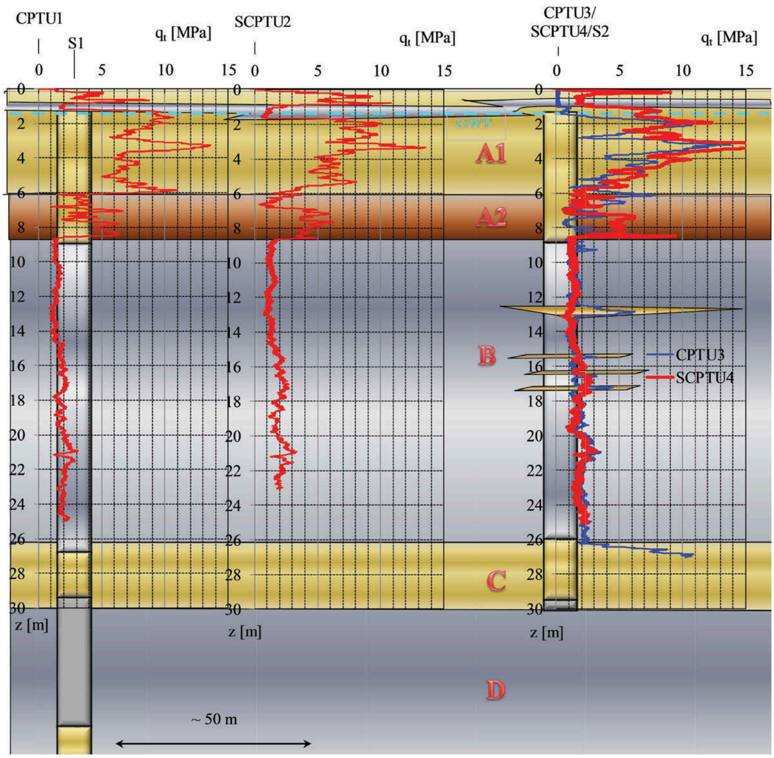

Figure 2. Soil profile deduced from in situ tests.

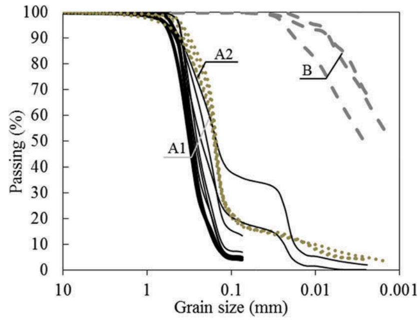

Figure 3. Grain size curves.

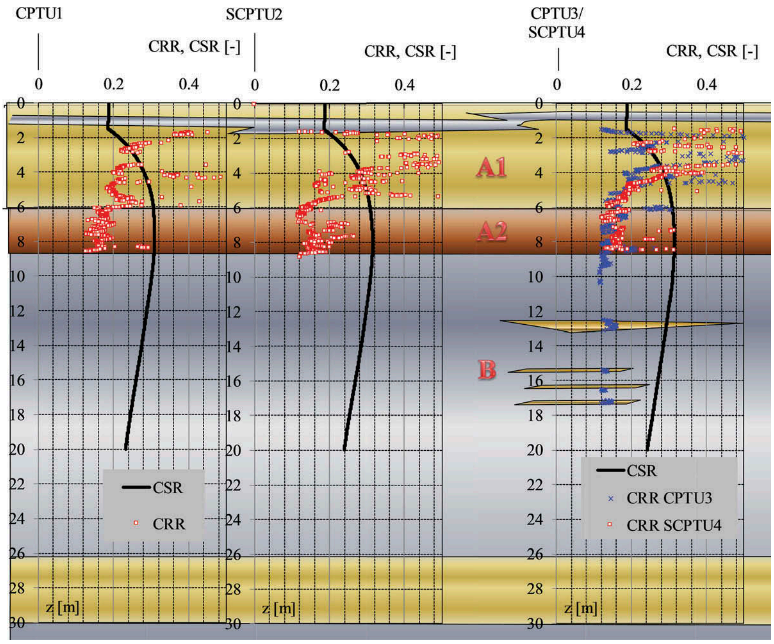

Figure 4. Soil cyclic resistance ratio CRR and earthquake induced stress ratio CSR as a function of depth, according to Idriss and Boulanger (2008).

for all the CPTs, as also shown in Figure 4, where the soil cyclic resistance ratio CRR and the earthquake induced cyclic stress ratio CSR are plotted as a function of depth z.Figure 4 shows that the sandy soils between 3 and 8.5 m of depth below the ground surface are the most susceptible to liquefaction. The liquefaction resistance of the sandy soils of units A1 and A2 was also directly measured by means of undrained cyclic triaxial tests carried out on samples reconstituted at the in situ void ratio. The reconstitution was carried out by pluvial deposition in air of the dry sand at the target dry density. Saturation was achieved with CO2 circulation, flushing of deaerated water and adequate back pressure (large enough to avoid cavitation when the soil develops negative pore water pressure). The value of the Skempton parameter B, measured at the end of the saturation was always higher than 0.98. After saturation, the samples were isotropically normally consolidated at a mean effective stress p'_c = 100 kPa and subjected to cyclic undrained loading. The cyclic tests were interpreted assuming as failure condition the states at which the double amplitude axial strain ε_a^{DA} equals 5%. All the failure conditions of the samples are represented in Figure 5, in terms of applied cyclic stress ratio and number of cycles at ε_a^{DA} = 5%. The pore pressure ratio R_u (defined as the ratio of the excess pore pressure developed during cyclic

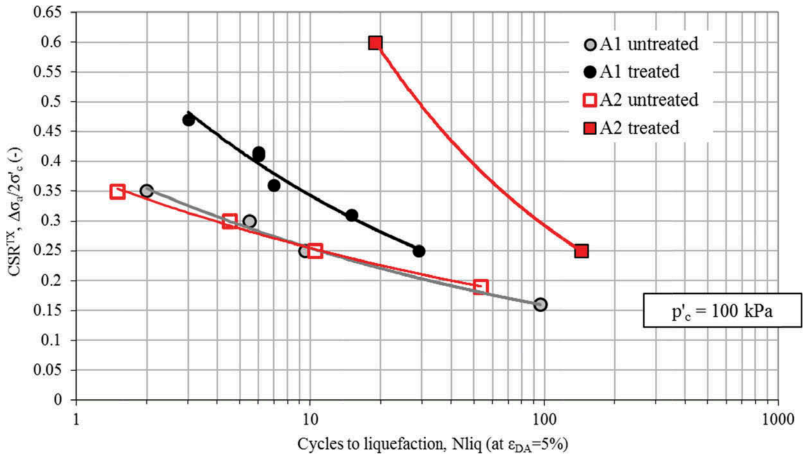

Figure 5. Cyclic strength of treated and untreated A1 and A2 sands.

loading to the mean effective stress at the end of consolidation, $R_u = \Delta u/p'_c$) values at failure were generally larger than 0.9. It's worth noting that A1 and A2 sands have a very similar cyclic resistance, despite of the larger fine content of A2. This can be explained by the non-plastic behavior of the silty fraction of A2. The results of Figure 5 allowed to carry out a direct assessment of the liquefaction resistance of A1 and A2 sandy soils.

The cyclic stress ratio from triaxial condition (CSR^{TX}) was corrected into cyclic stress ratio for simple shear conditions (CSR^{SS}) according to Ishihara et al. (1977) and (1985):

$$CSR^{SS} = CSR^{TX}(1 + 2k_0)/3 \tag{1}$$

where $k_0 = \sigma'_r/\sigma'_a$ = stress ratio at rest, computed as a function of φ'_{cs} for normally consolidated samples using the equation of Jaky (1944) (k_0 = 0.4 and 0.47 for A1 and A2, respectively).

The cyclic resistance ratio CRR in simple shear condition estimated for A1 and A2 from Figure 5 and for a number of equivalent cycles N = 7 (associable to an earthquake of magnitude M = 6.14, according to Idriss 1999) are shown in Figure 6a and compared to the earthquake induced cyclic stress ratio CSR. The laboratory results confirmed the high susceptibility of the A1 and A2 soils to liquefaction.

4 LIQUEFACTION RESISTANCE OF CHEMICALLY GROUTED SOILS

Among the several in-situ ground improvement existing technique to mitigate liquefaction risk, chemical grouting is especially useful when the soil to be treated is difficult to reach, as in the case of soils under existing foundations. This technique has been selected as the more suitable to increase the liquefaction resistance of the sandy layers of Units A1 and A2 present below the school shallow foundations. The adopted chemical grout is a sodium silicate solution with a sodium-aluminate reactant, which at high dilution has low viscosity and density similar to the water (at T = 20°C the solution density is 1.2 g/cm^3 and the dynamic viscosity is 2 cP). The viscosity increases over time and the gel time can be controlled by adjusting the concentration of the reagent in solution (Persoff et al 1999, Gallagher 2000). Once gelled, the silica grout forms a light binder film between and around the soil particles, which, attaining a slight cohesion, become more resistant to seismic stresses induced by an earthquake. The solution is inorganic and nontoxic.

The effective capability of the colloidal silica grout to permeate the soil was first tested in laboratory, flushing the solution through samples of both A1 and A2 soils, reconstituted by pluvial deposition at the site void ratio. The solution circulation was carried out from the bottom to the top maintaining a small differential pressure. The solution permeated easily in both A1 and A2 samples. The tested solution has an evolutive behavior and once in contact

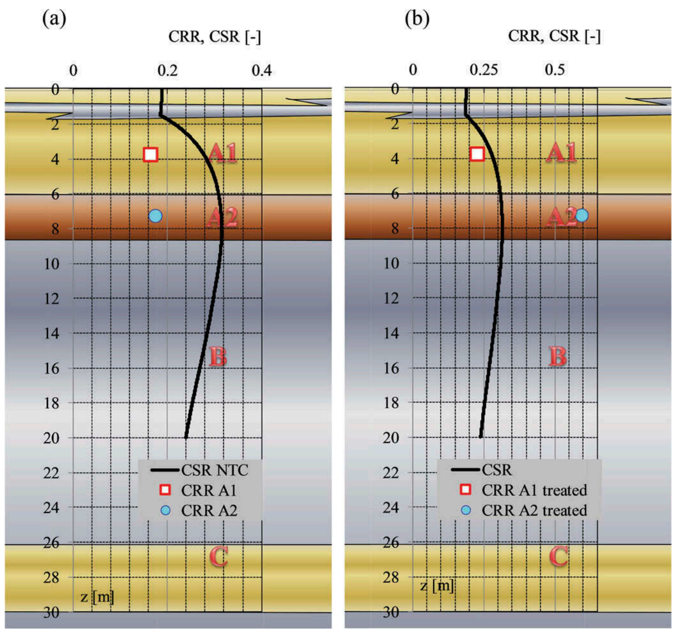

Figure 6. Soil cyclic resistance ratio CRR and earthquake induced stress ratio CSR as a function of depth, from undrained cyclic triaxial tests: (a) untreated soils and (b) treated soils

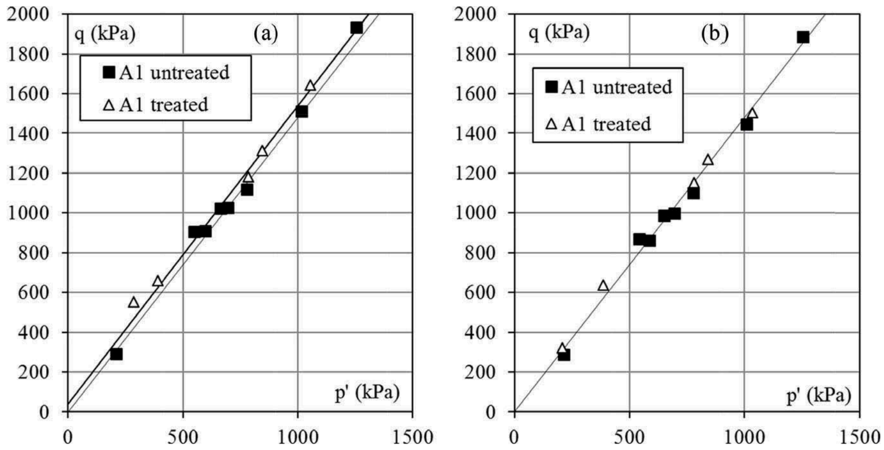

Figure 7. Strength states at (a) peak and (b) critical state for treated and untreated A1 sand.

with air tends to gel in 60-90 min. After gelling the samples were left to cure for about 1 day to allow the development of cementation bonding. The cured samples were then saturated in the triaxial cell using back pressure, isotropically normally consolidated at a mean effective stress p'_c = 100 kPa and subjected to both monotonic and cyclic undrained loading.

The failure envelopes at peak and critical states for treated and untreated A1 sand are shown in Figures7a and b, in the stress deviator, q and mean effective stress, p' plane. The envelopes at peak are almost parallel (shearing resistance angle at peak φ'_p= 37ű) but treated A1 sand has a cohesion intercept c' = 37 kPa. The shearing resistance angle at critical state of treated and untreated A1 sand is the same, i.e. φ'_{cs} = 36ű.

Figure 8 shows on the right side the cyclic stress deviator q versus the axial strain ?a and on the left side the stress paths in the q-p' plane measured during two undrained CTX tests

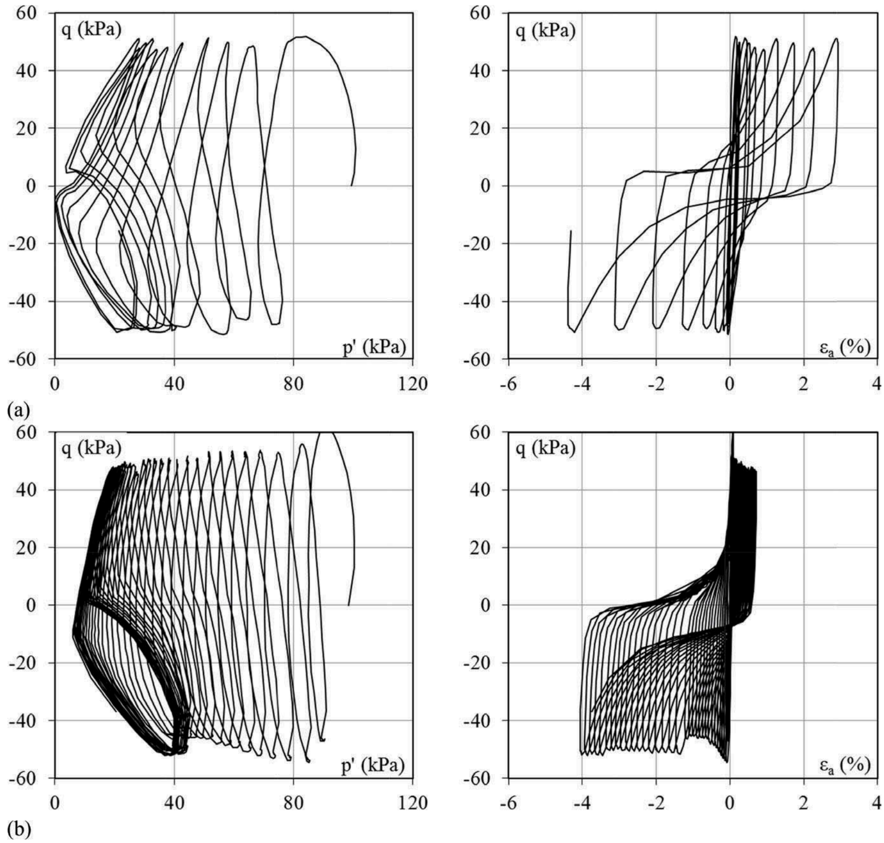

Figure 8. Stress-strain relationships (right side) and stress paths (left side) from undrained CTX tests on (a) untreated and (b) treated A1 sand.

carried out applying the same cyclic stress ratio ($CSR^{TX} = 0.25$) on (a) an untreated and (b) a treated A1 samples. The results in Figure 8 highlight the effect of the cementation on the cyclic behaviour of A1: the untreated sample reach failure between the 9th and 10th cycles and large axial strains develop quite quickly. In the treated sample the development of axial strains and the decrease of mean effective stress is slower and the liquefaction condition is achieved after more than 30 cycles.

The cyclic resistance of treated A1 and A2 soils are shown in Figure 5 and compared with the liquefaction resistance of the untreated materials. For a number of equivalent cycles N = 7, the treated A1 sand has a liquefaction resistance in triaxial condition 35% higher than the untreated sand; the increase in cyclic resistance in treated A2 is instead very much higher and at N = 7 thesilty sand is no more liquefiable. The effect of these results on the in situ resistance are shown in Figure 6b, where the cyclic resistance ratio CRR in simple shear condition estimated for treated A1 and A2 for N = 7 are compared to the earthquake induced cyclic stress ratio CSR.

It's worth noting that the actual injectability of the in-situ A1 and A2 layers and the level of improvement attainable (especially as far as the A2 silty sand is concerned) must be checked directly on site, so a trial test will be performed. A possible scheme of the trial tests consists of nine vertical drillings, located at the vertex of three triangular meshes with different spacing, from which the silica grout will be injected using tube a manchettes. In situ and laboratory tests will allow to check thecapability of the silica-grout to permeate the sandy layers and its efficacy in reducing the liquefaction susceptibility.

5 FINAL REMARKS

Cyclic liquefaction is a phenomenon during which granular uncemented saturated soils (gravel, sand and low plasticity silt) lose much of their strength and stiffness for a short interval of time, but long enough to cause significant failures. The selection of a mitigation technique for a specific site depends on several aspects, among which the accessibility of the area to be treated can be determining. At open sites a wide range of methods can be employed to reduce the liquefaction risk. At developed sites, where it may be difficult or impossible to apply conventional remediation technique, permeation grouting can be particularly advantageous, allowing to avoid disturbing of the natural soil and existing foundations.

Permeation grouting using a colloidal solution has been selected to stabilize against liquefaction the sandy subsoil of a school building located in the town of Cesenatico, Italy. The building houses about 600 students and will be operating during the remediation works.

A series of undrained cyclic triaxial tests was carried out on sandy samples, reconstituted at the site void ratio and permeated using a sodium silicate solution. The tests indicated a very satisfactory increase of the soil liquefaction resistance.

The laboratory results will be checked on site through a trial test in order to calibrate the grout injection procedure and the grout mixing process.

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