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LIQUEFACT

Assessment and mitigation of Liquefaction potential across Europe: a holistic approach to protect structures/infrastructure for improved resilience to earthquake-induced Liquefaction disasters.

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Guidelines for the use of ground improvement technologies to mitigate the liquefaction risk on critical infrastructures

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TREVI	Trevi Società per Azioni	Italy

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GLOSSARY

Acronym	Description
AGI	Associazione Geotecnica Italiana
AOS	Apparent Open Size
ASCE/G-I	American Society of Civil Engineers/ Geo-Institute
ASTM	American Society for Testing and Materials International
BC	Boundary Conditions
CAV	Cumulate Absolute Velocity
CDC	Canterbury Development Corporation
CDM	Cement Deep Mixing
CEN	European Committee for Standardization
CEN TC 288	European Committee for Standardization -Technical Committee 288
CEN TC250/SC7 – EG14	European Committee for Standardization - Technical Committee 250 / Sub-Committee 7 – Evolution Group 14
CL; CH	Inorganic clays (low plasticity); Inorganic clays (high plasticity)
CPT	Cone Penetration Test
CPTU	Cone Penetration Test with Piezocone
CRR	Cyclic Resistance Ratio
CSR	Cyclic Stress Ratio
DCP	Dynamic Cone Penetrometer Test
DDC	Deep Dynamic Compaction
DDM	Direct Density Measurement
DJM	Dry Jet Mixing
DMM	Deep Mixing Method

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DMT	Dilatometer Test
DSM	Deep Soil Mixing
EC	Explosive Compaction (Blasting Compaction)
EC7; EC8	Eurocode 7; Eurocode 8
EDP	Engineering Demand Parameter
EG	Evolution Group
EN	European Standard
EPA	Environmental Protection Agency (US)
EQ	Earthquake
EU	European Union
FL; FSL	Factor of safety against Liquefaction
FS	Factor of safety
GM; GC	Silty gravel; Clayey gravel
GW; GP	Well graded gravel; Poorly graded gravel
HEIC	High Energy Impact Compaction
IM	Earthquake Intensity Measure
IPS	Induced Partial Saturation
JG	Jet Grouting
JGS	Japan Geotechnical Society
LBS	Liquefaction-induced Building Settlement
LGM	Low Mobility Grout
LPI	Liquefaction Potential Index
LSN	Liquefaction Severity Number
LT	Loading Test

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MBIE	Ministry of Business, Innovation and Employment of New Zealand
ML; MH	Silts and clays with w_L < 50; Inorganic silt with moderate to high plasticity
MPT	Menard Pressumeter Test
MSF	Magnitude Scaling Factors
NZGS	New Zealand Geotechnical Society
OH; PT	Organic silts or clays with moderate to high plasticity; Peat soils with high organic contents
OSU	Oregon State University
P(L)	Probability of Liquefaction
PGA	Peak Ground Acceleration
PL	Liquefaction potential index
PVD	Prefabricated Vertical Drains
QA; QC	Quality Assurance; Quality Control
RAIF	Resilience Assessment and Improved Framework
SBT	Soil Behaviour Type
SC	Sub-Committee
SCEC	Southern California Earthquake Center
SMM	Shallow Mixing Method
SMW	Soil Mixed Wall
SN	Suitability Number
SPT	Standard Penetration Test
SW; SP; SM; SC	Well graded sand; Poorly graded sand; Silty sand; Clayey sand
TC	Technical Category or Technical Committee
ULS; SLS	Ultimate Limit State, Service Limit State

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US	United States
USCS	Unified Soil Classification System
VC	Vibro Compaction
VR	Vibro Replacement
WP7	Work Package 7
Symbols	Description
$(N_{1,60})_{cs}$	Equivalent clean sand normalised number of blows
$(q_{c1N})_{cs}$	Equivalent clean sand normalized cone tip resistance
A	Parameter that affect the shape of build-up curve ($\partial u_g/\partial N$)
a; b	Coefficient and exponent of CRR(N) expression
a_{max}	Maximum horizontal acceleration at the ground surface
a_r; A_r; A	Replacement ratio; Area of columnar reinforcement; Total area of treated soil
B; c	Structure width; Foundation aspect ratio correction
C	Cement
c; c_u	Cohesion; Undrained shear strength
CAV_{dp}	Standardised Cumulate Absolute Velocity
CN	Overburden correction factor
CN; CE; CB; CR; CS	Correction factors for SPT number of blows
CRR_s^{ctx}; CRR_s^{css}	Cyclic Resistance Ratio for saturated soil (ctx from cyclic triaxial tests, css from cyclic simple shear tests)
CRR_{un}^{ctx}; CRR_{un}^{css}	Cyclic Resistance Ratio for unsaturated soil (ctx from cyclic triaxial tests, css from cyclic simple shear tests)
CSR^*; CSR_{rs}	Cyclic Stress Ratio for the untreated portion of the reinforced soil; Cyclic Stress Ratio for the reinforced soil
CSR15	Cyclic Stress Ratio corresponding to 15 cycles

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D	Effective depth of treatment
D; M	Nozzle diameter; Number of nozzles
d; s	Diameter of the drain; Spacing between drains
D_{15}; $d_{15,}$ d_{85}	Diameter of passing particles at 15% by weight, for the filter material; Diameter of passing particles at 15% and 85% by weight, for the natural soil
$d_{85\ (or\ 95)}$	Diameter of passing particles at 85% (or 95%) by weight, for the suspension
d_e	Diameter of the influence zone
D_f; $D_{S,1}$	Depth from the surface to the bottom of the foundation; Depth to the centre of the uppermost susceptible layer
D_i	Diameter of passing particles at i%
D_m	Mean diameter of the column
Dr	Soil relative density
D_{ref}	Reference diameter
Dt; Ds; De; Dv	Liquefaction-induced building settlement; Shear-induced building settlement; Settlement (sediment ejecta); Volumetric-induced building settlement
E	Average applied energy
E'_n	Specific kinetic energy at the nozzles
e_0	Void ratio (initial)
E_{tot}; $E_{v,liq}$; $E_{s,liq}$	Total specific energy of deformation; Volumetric specific energy; Deviatoric specific energy to reach liquefaction
$E_{v,sk,liq}$; $E_{w,liq}$; $E_{air,liq}$	Specific work done respectively to cause the deformation of the soil skeleton, the flow of water and the flow of air into the pores network
FC	Fine content
fs	Sleeve friction
FS_{deg}	Degraded static factor of safety
FS_{ff}	Factor of safety against Liquefaction "free field"

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f_{so}; f_{fnd}; f_{st}	Functions that capture effects due to the characteristics of the soil profile, foundation and the structure
g	Gravity acceleration
G; G_0; G_{eq}; G_s; G_r	Shear modulus; Shear modulus of the soil at small strain; Equivalent shear stiffness; Shear stiffness of the original soil; Shear stiffness of reinforcement
H	Drop height
H(-); $H_{S,i}$; $D_{S,i}$	Heaviside step function; Thickness of the i^{th} susceptible layer; Depth from the bottom of the foundation to the centre of the i^{th} susceptible layer
H_{liq}; H_{crust}	Thickness of Liquefiable Layer; Non-liquefiable crust thickness
h_{eff}	Effective height of the structure
HN	Hopkinson's number
I_c	Soil Behaviour Type Index
k; $k_{(intrinsic)}$	Permeability; Intrinsic permeability
k_0 ; φ_p	Coefficient of earth pressure at rest; Peak friction angle
k_0; k_m ; k_w	Absolute permeability; Permeability to mixtures; Permeability to water
K_α; K_σ	Corrected term for influence of static shear stress; Corrected term for overburden pressure
M; M_w	Magnitude; Moment Magnitude
M_{st}	Inertial mass
$m_{v,3}$	Volumetric compressibility
n	Empirical factor
N	Number of drops
N; N_c	Groutability ratios
N; N_{liq} or N_L	Number of cycles; Number of cycles needed to reach liquefaction for a given value of CSR
N^*_γ; N^*_q	Bearing capacity coefficients computed as a function of the degraded friction angle

$N_{1,60}$	SPT normalised number of blows
N_{SPT}	SPT number of blows
O_{95}; O_{50}	AOS of filter; Size which is larger than 50% of the fabric pores
P	Number of passes
p'	Mean effective stress
Pa or pa	Atmospheric pressure
p_g; p_a; p_w	Grout pressure; Air pressure; Water pressure
PI	Plasticity index
P_{max}	Maximum injection pressure
q; $q_{ult,deg}$	Bearing pressure; Degraded bearing capacity
q_c; q_{c1}; q_{c1N}	Tip resistance in cone penetration test; Tip resistance corrected with the overburden stress; Tip resistance normalised with the atmospheric pressure
q_d	Cyclic deviatoric stress
Q_g; Q_a; Q_w	Grout flow rate; Air flow rate; Water flow rate
r	Distance from the charge
Rd	Depth-dependent shear stress reduction coefficient
r_d	Shear stress reduction factor that accounts for the dynamic response of the soil profile
R_p; R_w	Rotational speed during penetration; Rotational speed during withdrawal
r_u; $r_{u,ff}$; $r_{u,str}$; $r_{u,foot}$; $r_{u,max}$; ; $r_{u,mean}$	Pore Pressure Ratio; Pore Pressure Ratio "free field"; Pore Pressure Ratio "structure"; Pore Pressure below the foundation; Maximum value of excess pore pressure ratio; Mean value of excess pore pressure ratio
S	Soil Factor or Settlement
s; S_r; S_{r0}; S_{rd}	Matric suction; Saturation degree; Saturation degree at the beginning of the cyclic phase; Design value of the saturation degree
T	Blade rotation number or representative period of the motion

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t; t_d; T_{ad}	Time variable; Significant duration of seismic shaking; Time factor
u; u_g	Pore pressure; Pore pressure increment generated during shaking
u_a; $u_{a,0}$;u_w; χ	Pore air pressure; Pore air pressure at the beginning of the cyclic phase; Pore water pressure; Material parameter accounting for the effect of the degree of saturation
v_{h50}	velocity index
V_p; V_w	Penetration velocity; Withdrawal velocity
v_r; ω	Average lifting speed; Rotational velocity
V_s; V_{s1}; V^*_{s1}	Shear wave velocity; Normalized shear wave velocity; Limiting upper value of V_{s1}
W	Water
W	Tamper mass or charge mass delay or foundation-weighting factor
W/C	Water-cement ratio by weight
w_L	Liquid limit
W_p; W	Amount of binder injected during penetration; Total amount of injected binder
Z; z	Depth
Z_{liq}	Thick of liquefiable layer
α_E; Λ^*	Coefficient related to the influence of the shrouding air jet on boundary dissipation; Hydrodynamic coefficient (for JG)
β	Parameter which depends physical and mechanical properties of soil in $r_{u,ff}(N/N_L)$ equation
β, δ	Coefficients calibrated on experimental data (for JG)
γ_m ; γ_w	Specific weight of the mixture; Specific weight of water
$\Delta(N_1)_{60}$; Δq_{c1N}	Additional resistance to consider the influence of fines content (SPT); Additional resistance to consider the influence of fines content (CPT)
δ_{AB}; α; w; L_{AB}; η_{AB}	Differential settlement; Tilt; Overall settlement; Distance A-B; Deflection ratio
ΔCRR^{ctx}; ΔCRR^{css}; $\Delta CRR_{,Nliq}{}^{ctx}$; $\Delta CRR_{,Nliq}{}^{css}$	Increments of Cyclic Resistance Ratio between unsaturated and saturated conditions (ctx from cyclic triaxial tests, css from cyclic simple shear tests)

Δs	Lifting step
Δu; Δu_{ff}	Excess pore pressure; Excess pore pressure "free field"
ε_{shear}	Shear strain on the free-field
ε_v; ε_s	Volumetric strain; Distorsional strain
μ; ρ	Water viscosity and density
μ_m ; μ_w	Dynamic viscosity of the mixture; Dynamic viscosity of water
ρ	Mass density
ρ_{dyn}	Dynamic settlement
σ	Normal stress
σ'_0	Effective confining stress prior to the earthquake
σ_h or σ_{h0}	Total horizontal stress
σ_h' or σ_{h0}'	Effective horizontal stress
ΣM	Total number of mixing blades
σ_v or σ_{v0}	Total vertical stress
σ_v' or σ_{v0}'	Effective vertical stress
τ; τ_{eq}; τ_c; τ_s; τ_r	Shear stress; Equivalent shear stress; Cyclic stress amplitude; Shear stress in the untreated portion; Shear stress for reinforced portion
φ'; φ *'; φ_0	Friction angle; Degraded friction angle; Initial friction angle
γ; γ_r; γ_s	Angular distortion; Angular distortion for reinforced portion; Angular distortion for the original soil
σ'_c	Confining effective stress
σ'_{un}; $\sigma'_{un,liq}$	Effective stress in unsaturated conditions; Effective stress in unsaturated conditions at liquefaction

CONTENTS

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SUMMARY

This deliverable is dedicated to the ground improvement technology with reference to the mitigation of liquefaction risk on critical infrastructures. As widely shown and recalled in all deliverables of Liquefact, seismic liquefaction may affect buildings (public or private) and infrastructures generating extensive physical damage and jeopardizing for long time their functionality. Impact, quantified as direct and indirect repair expenses, the costs necessary to bring the structure and its content to the original conditions, and as a reduced functionality of the structure, may turn into severe financial and social losses for the affected communities. In this framework particularly attention must be paid to the critical infrastructures, i.e. those systems and organizations that deliver goods and services fundamental for the life of the society and for the economy of the productive asset (Macaulay, 2009).

The ground improvement technology, with its large variety of solutions nowadays available, offers the possibility to mitigate the effects of liquefaction on soil and, with these, the above recalled physical and economic impact. However, their application requires an adequate knowledge of their basic principles, particularly of the response of the modified soil to the seismic excitation. This analysis must necessarily converge into the definition of design rules that enable engineers to quantify minimum capacity of the improved soil, i.e. the liquefaction resistance able to sustain the seismic demand. Once this question has been positively addressed, the further step concerns execution of treatments, i.e. the definition of the set of operative parameters necessary to reach the prescribed goal. This step implies to predict the effects of treatment, i.e. the relation between executive parameters and the modification of ground properties. Only in a few cases this analysis can be accomplished with theoretical/sound theoretical models, basically due to the lack of adequate mechanical reconstruction of the phenomena taking place during treatment. More often this goal is achieved with the aid of experiments, performing prior the execution of treatments the so-called field trial. It consists of a real scale reproduction of the treatment, where the operative parameters are varied within prescribed ranges and their effects measured. Even this process must be understood and disciplined, defining the rules for quantifying the effects of treatments. Lastly, as also stated in the international standards, the efficacy of treatments must be proven with experiments able to confirm the correct execution of treatments (Quality Assurance) and the achievement of the prescribed results (Quality Control).

The above sequence of topics is mirrored into the organization of the present deliverable. After an introductory chapter that illustrates the main effects of liquefaction on the buildings and infrastructures together with the possible mitigation strategies, the second chapter presents an overview of the available techniques, describing for each of them how the specific ground modification inhibits the effects of liquefaction, the practical execution and the list of treatment parameters, advantages and drawbacks. The third chapter presents the design principles of each category of ground improvement technique with reference to the performance in terms of liquefaction mitigation. This theme is presented in terms of capacity vs demand, introducing the necessary mechanical variables able to describe the phenomenon and include the effects of ground improvement. The fourth chapter is dedicated the experimental verification of treatments in the framework of Quality Assurance/Quality Control strategy. Finally, all information regarding each of the considered ground improvement techniques is synthetically presented in a card given in the appendix A of the present document.

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1. INTRODUCTION

1.1 Objective

In the general scope of the project, aimed at defining an operative strategy to quantify and mitigate the liquefaction risk on critical infrastructures, the Work Package WP7 has the role of validating the defined procedures with the retrospective analysis of past events and of summarizing the outcomes into guidelines that enable operators to implement methodologies for risk assessment and the EU Commissions to produce technical standards. Bearing this goal in mind, the action has been focused on two complementary targets, i.e. evaluate the liquefaction risk of a generic system and standardize the use of ground improvement technologies for mitigating the liquefaction risk.

As widely shown in the literature, liquefaction rarely produces the dramatic and shocking number of casualties typical of other earthquake effects like building collapse, landslides and tsunamis. Only in few cases liquefaction affected massively the territory, like in the flow failure examples occurred in 1964 in Alaska, that caused 32 casualties, or in the more recent 2018 earthquake occurred in Indonesia. This mechanism occurs when the static shear stresses on sloping ground exceed the frictional shear strength of the soil deteriorated by the pore pressure build-up. Displacements in this case can be very large, in the order of tens of metres or even more, and very fast disrupting buildings and infrastructure over wide areas.

Apart from this extreme case, the more frequently noticed impact concerns the foundation of buildings and infrastructures. Briefly, damage occurs in the form of horizontal ground displacements caused by liquefaction of loose granular soils (e.g. 0'Rourke and Hamada, 1992; Hamada and O'Rourke, 1992) or as transient loss of bearing capacity that result in large vertical displacements or overall instabilities. However, earthquakes all over the world (Turkey, Greece, Taiwan, India, Japan, New Zealand, Italy) have highlighted a complex behaviour of the structure-foundation system subjected to earthquake vibrations. For example, the Adapazari area (Turkey) suffered extensive liquefaction with buildings that rotated significantly and others that underwent quite uniform settlements of several dozen centimetres. In other cases, ununiform settlement caused deformation of the superstructures up to intolerable levels. Other effects can be seen on horizontal infrastructures like breakage or disconnection of pipelines or uplift of sewer manholes. These examples highlight the importance of understanding the mechanism triggering liquefaction and the response of soil subjected to ground shaking together with the interaction with the overlying or embedded structures.

Ground improvement is widely used worldwide to mitigate the negative effects of liquefaction. Listing all the accomplished solutions is nearly impossible, because of the very large number and variety of examples. Most frequently, ground improvement techniques are based on a conceptual scheme only and are implemented with rule of thumb approaches. The partial or nil knowledge of the quantitative relation existing between the characteristics of the technique (the so-called executive parameters) and

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the mechanical modification induced in the original soil has the effect that the mathematical formulations supporting rational design methods are weak.

The need is thus felt to comprehend the existing knowledge into mechanical schemes that characterise the mechanical principle of the technique and reproduce the effects of treatments. The support of deeper experimental investigation is sometimes necessary to develop more robust rules and enable to optimally design treatments.

Despite the mechanisms to mitigate liquefaction continue to be areas of ongoing research, there is a relative success of ground improvement methods in preventing damage caused by liquefaction events. From an engineering viewpoint, there is the need to choose suitable liquefaction remediation measures. The choice of a ground improvement method shall be determined considering for each class of structure and situation issues like effectiveness, reliability, technical feasibility, economic conveniences etc.

The variety of ground improvement solutions nowadays available has been considered in the present report selecting the most frequent and well-established techniques. For each of them, the basic principles have been addressed examining the response of the soil to the seismic excitation and the effect of modification. This analysis converges into the definition of design rules that enable engineers to quantify minimum capacity of the improved soil, i.e. the liquefaction resistance able to sustain the seismic demand. Once this question has been positively addressed, the further step concerns execution of treatments, i.e. the definition of the set of operative parameters necessary to reach the prescribed goal. Governing this process implies to predict the effects of treatment, i.e. the relation between executive parameters and the modification of ground properties. Only in a few cases this analysis can be accomplished with theoretical or sound theoretical models, basically due to the lack of an adequate mechanical reconstruction of the phenomena taking place during treatment. More often this goal is achieved with the aid of experiments, performing prior the execution of treatments the so-called field trial. It consists of a real scale reproduction of the treatment, where the operative parameters are varied within prescribed ranges and their effects measured. Even this process must be understood and disciplined, defining the rules for quantifying the effects of treatments. Lastly, as also stated in the international standards, the efficacy of treatments must be proven with experiments able to confirm the correct execution of treatments (Quality Assurance) and the achievement of the prescribed results (Quality Control).

1.2 Typical damage on buildings and infrastructures

The damage caused by liquefaction on buildings and infrastructures can be very impactant for the life of community. Although not the only case of such a size, the 2010-2011 earthquake sequence in Christchurch (New Zealand) is probably the most impressive example of liquefaction induced damage over an urban environment. It is estimated that about 15.000 families lost their homes and 8.000 were permanently displaced (see Figure 1-1 as an example), 70% of the buildings in Central Business District had to be

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demolished, 900.000 tons of liquefied soil were removed from the ground surface after the events (Tonkin and Taylor, 2016).

Figure 1-1: Evolution of land use in the red zone of Christchurch across the February 2011 earthquake.

The reconnaissance of damage performed in Japan after the 2011 earthquake by the Oregon State University (OSU, 2011) provides a typical sequence of the liquefaction effects on buildings and infrastructures. Areas affected by liquefaction typically undergo extensive production of sand ejecta at the ground level (Figure 1-2) that often occur in open unused areas, but sometimes invade the basement of buildings. This phenomenon arises from the excess pore pressures in the ground that exhaust through holes formed in the upper less permeable crust dragging sand particle to the surface with the high generated hydraulic gradient (piping). When this phenomenon occurs, underground cavities are formed that lead to sinking of the ground level (Figure 1-3) and to extensive damage on nearby buildings or infrastructures. Absolute prevention of this occurrence with ground improvement is not simple, basically due to the large volume of material potentially involved. Feasibility should be estimated with a careful cost-benefit analysis that considers this countermeasure versus other possible mitigation strategies, limiting action to the areas where risk is major.

The impact on buildings depends significantly on the characteristics of the subsoil coupled with the dimension, shape and load distribution at the foundation level. Bray & Macedo (2017) distinguish three different possible mechanisms occurring in the subsoil capable of producing settlements on buildings: shear deformation given during the earthquake by the increase of pore pressures and the contemporary reduction of the mean effective stress and enhanced by the load (static and dynamic) applied from the superstructure; post-earthquake volume deformation given by the dissipation of excess pore pressure and restoration of the original overburden stress state in the soil skeleton; shrinking of underground cavities due to sand ejecta. While the last phenomenon is associated with the attainment of a liquefied state in the subsoil (i.e.

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nullification of the effective stress), the former two mechanisms may occur even when nil effective stresses are not attained. The manifestation of the above phenomena at the building level includes rigid movements such as uniform settlements or rotation, but also deformation of the superstructure.

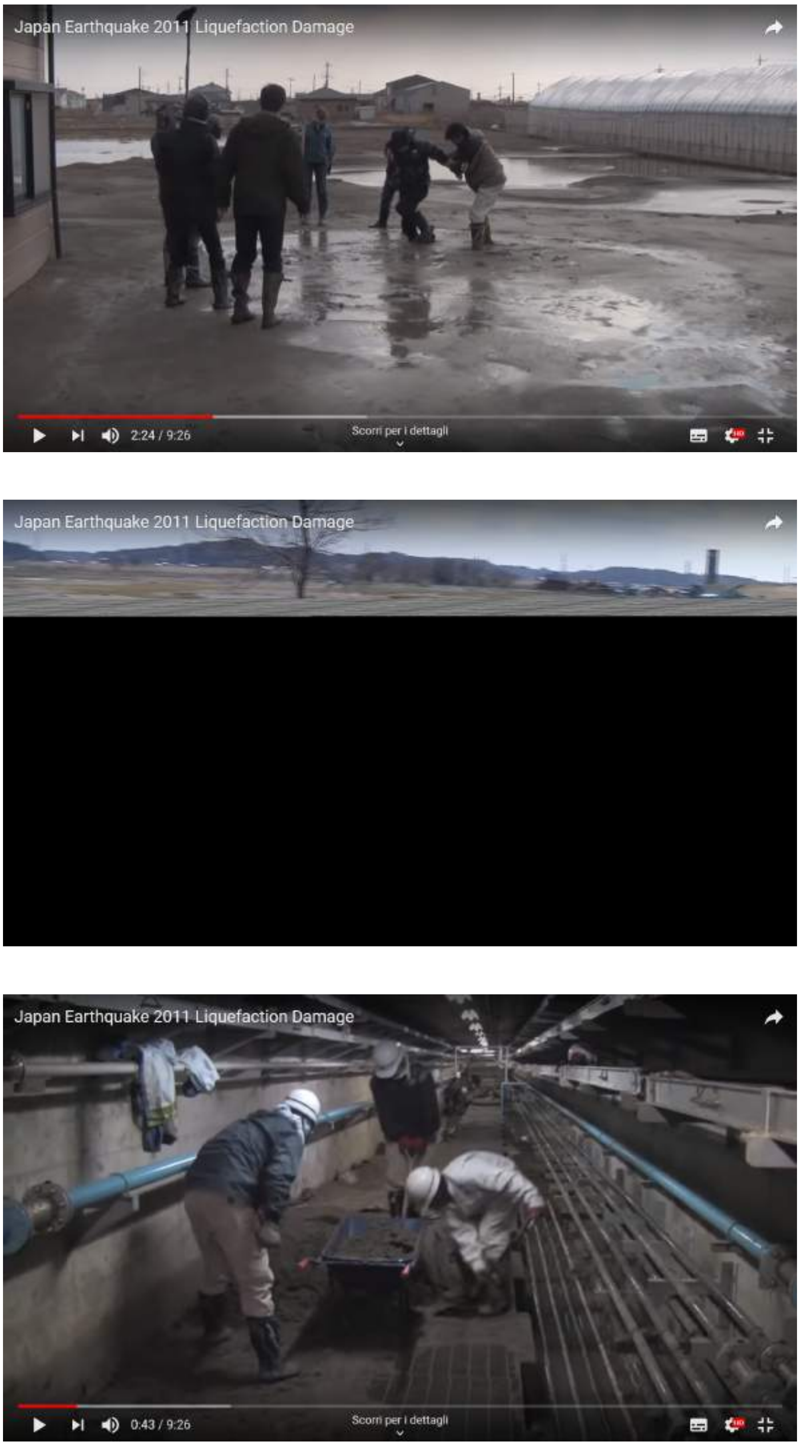

Figure 1-2: Extensive sand ejecta in open areas and at the basement of a treatment plant (OSU, 2011).

Figure 1-3: Sinking of the ground (OSU, 2011).

The response changes significantly (see for instance Figure 1-4) depending on the following factors:

- *Load transferred onto the soil* - shear and volume deformation of the subsoil depends significantly on the load transferred from the top. Light buildings undergo settlements mostly because of liquefaction effects (dissipation of pore pressures and sand boiling); the initial deviator stress produced in the soil by heavy buildings is responsible of the significant shear deformation occurring at the side of the foundation.
- *Height and slenderness of the building* – Non-uniform load transfer at the foundation level contributes to differential settlements. Tall structures with a small footprint may undergo rotation when subjected to the directional dynamic loading (Figure 1-5), while low structures with wide footprint tend to settle more homogeneously (Figure 1-4).
- *Relative position of the buildings in comparison with nearby structures* – titling tends to occur more frequently on isolated structures than on buildings whose subsoil is confined by nearby structures.
- *Subsoil heterogeneity* - the spatial variation of soil properties along the foundation plane determines a heterogeneous response at the ground level that may exaggerate differential settlements.
- *Foundation type* - the flexural stiffness of the foundation plays a predominant role on the deformation of the superstructure. Continuous foundation systems (strip footings, mats or rafts) tend generally to produce rigid movements like uniform settlements or tilting, while isolated footings produce a distortion of the superstructure.

The above effects may be prevented with ground improvement interrupting the chain of underlying mechanisms, i.e. reducing the soil deformation caused by earthquakes, inhibiting the onset of pore pressures, transferring the load to deeper competent strata etc.

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Figure 1-4: Uniform settlements, deflection and tilting of buildings caused by liquefaction (OSU, 2011).

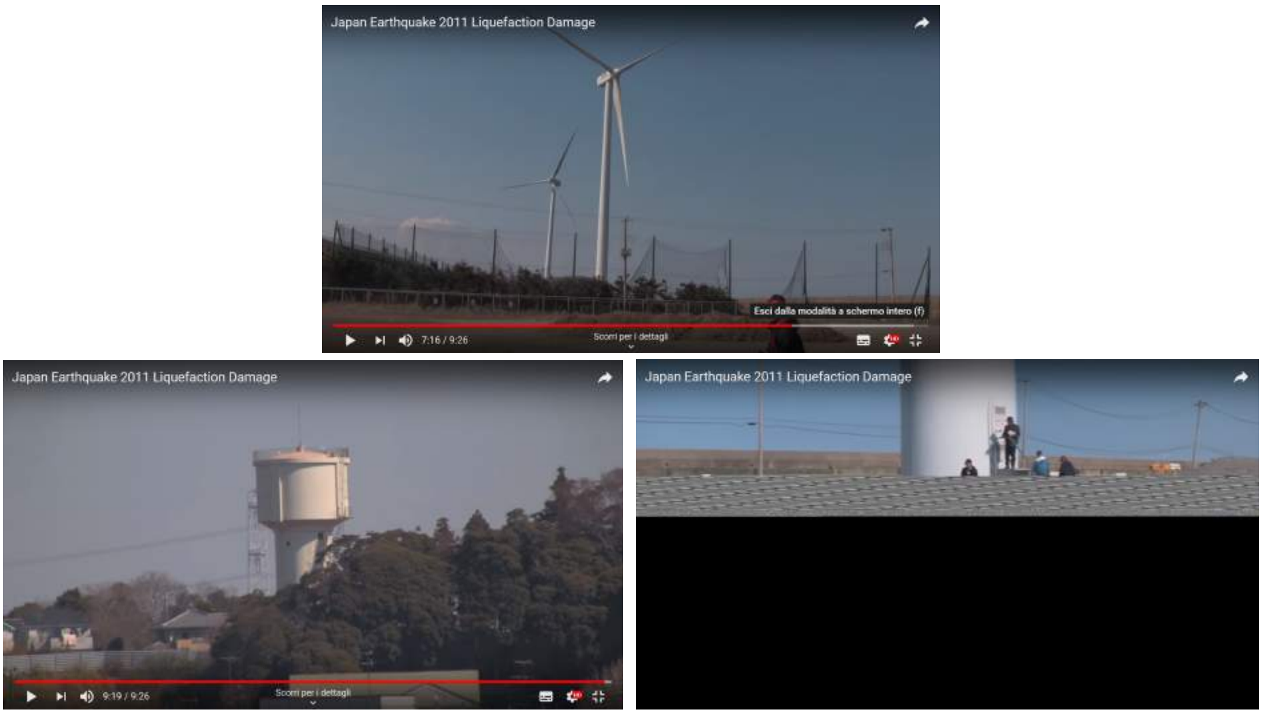

Figure 1-5: Titling of tall structures (wind turbine, water tank) due to liquefaction.

Earth retaining structures may collapse during earthquake due to liquefaction occurring at the foundation level or in the backfill material. Lateral movement, tilting or foundation failure are typical mechanisms observed during severe earthquakes (Figure 1-6). While for foundation collapse, the criteria defined above may be recalled, liquefaction of the backfill material increases the horizontal forces acting on the retaining structure. An accurate investigation of the composition and of the state (density, saturation) of the soil located at the foundation of the earth retaining structure and of the backfilled material enables to identify the possible critical mechanisms and choose an appropriate ground improvement solution. The intervention may be relatively simple or difficult and costly depending on the local conditions, extension (height, width) of the structure and presence of nearby sensitive buildings.

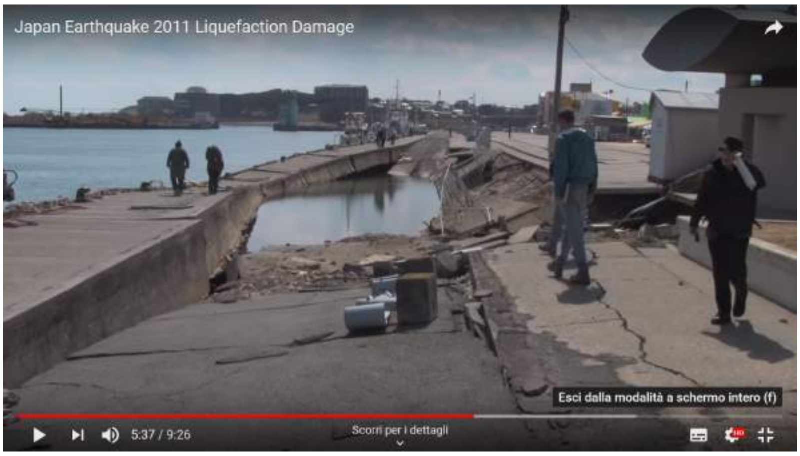

Figure 1-6: Collapse of earth retaining structures (OSU, 2011).

Fresh and wastewater lines are among the most critical and vulnerable infrastructures for a community, not only because works and costs necessary to repair the effects of liquefaction are very severe due to the wide distribution of these systems over the urban areas. It must be additionally considered that these infrastructures are fundamental for the community life and their damage has a tremendous impact on the population in terms of indirect costs related with the lack of service that makes entire urban areas inhabitable. Damages (Figure 1-7) may derive from the truncation of pipes in the displaced sections or more often from the disconnection of joints. All these phenomena are triggered by the relative displacement along

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the lines. Ground improvement provides a solution to reduce them, but an alternative countermeasure consisting in the adoption of more flexible pipes to reduce stresses should be considered as well.

Another frequent effect on wastewater systems is the uplift of manholes that occur due to the attainment of a liquefied state in the soil used to fill the gap with the surrounding hole. Ground improvement aimed at reducing the onset of liquefaction provides an excellent solution to mitigate this effect.

The map of Figure 1-8 shows the distribution of damage occurred in the water distribution system of Urayasu (Japan) after the big 2011 earthquake. Damaged trunks are denoted with red lines to distinguish them from undamaged pipelines coloured with blue. This area has been reclaimed in the Tokyo bay area filling the seabed with sandy soils in different ages. Damage is negligible in the north-western portion forming the old (continental) part of the city, extensive in the southern (reclaimed) more recent areas. It is also interesting to note that some small areas in the reclaimed land did not undergo damage even though fully surrounded by damaged pipelines. They correspond to sites where ground improvement was achieved (vibro compaction). Damage mainly occurred due to disconnection of joints. Preventing these effects with ground improvement poses a serious concern on the technique to be used, that must be chosen considering depth of involved subsoil and the extension of the area.

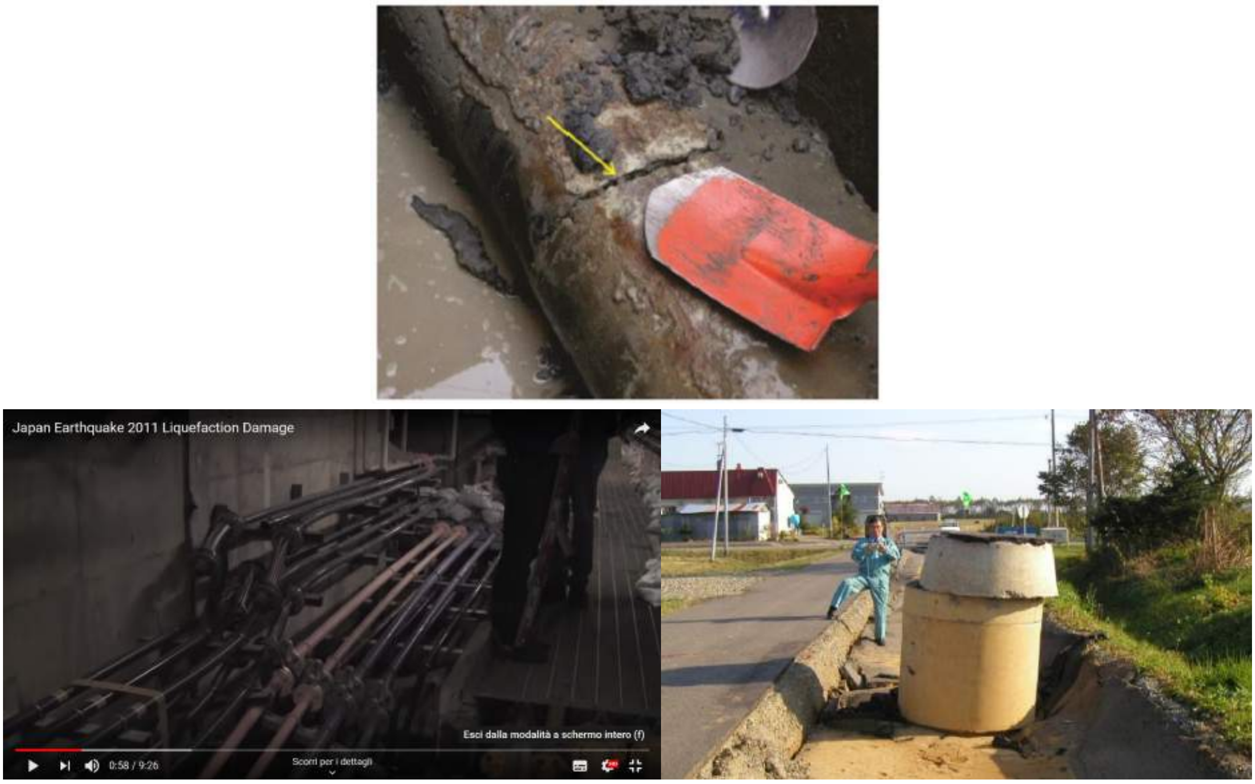

Figure 1-7: Liquefaction damage on water distribution and sewer system.

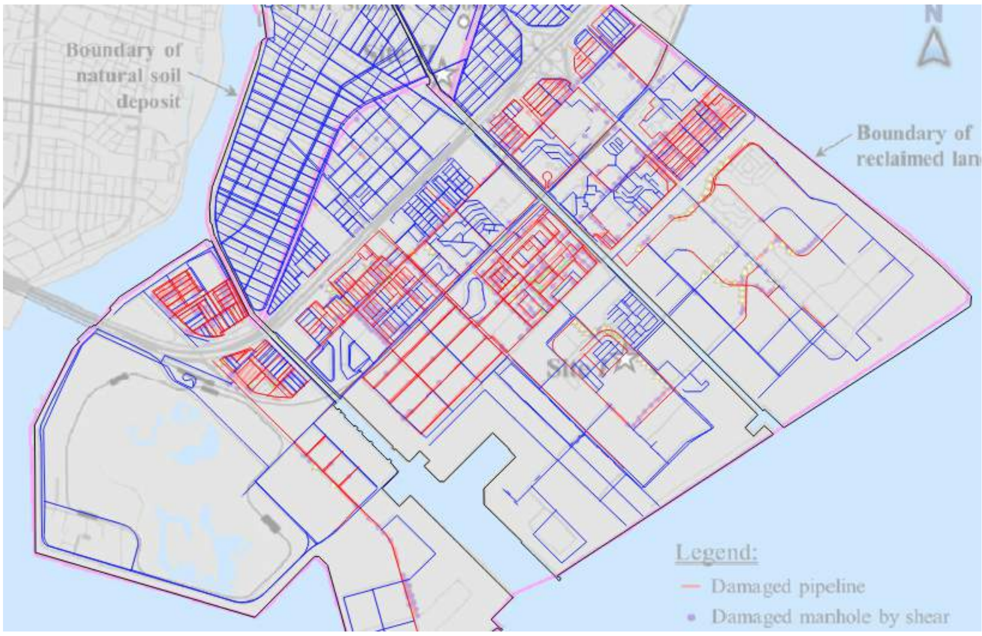

Figure 1-8: Liquefaction damage on water distribution and sewer system.

Levees are one of the major systems to protect lowland areas from flooding. Liquefaction occurring at the toe of the embankment may determine sliding collapse and cracks in the upper crest like the one shown in Figure 1-9. The phenomenon may take place more extensively, over the whole foundation determining an overall reduction of the crest level that may turn to be problematic for the future management of the territory. Remediation in this case is very invasive and costly and is aimed at hampering the onset of liquefaction improving/reinforcing the ground.

One of the fundamental assets for a community is the transportation network, with all the different communication lines (roads and railways etc.) working at the intra or inter urban system. Embankments, viaducts and bridges may suffer phenomena like those shown in Figure 1-10.

Embankments may settle at a level that serviceability is reduced or totally hampered. Abutments and foundation of piers may collapse jeopardizing the safety of the entire structure and causing an interruption of the function. The latter event may be particularly critical considering that bridges are often the only communication way between the two riversides. In all cases, the effect for the population is an increased time for the commuting travel. The social and economic cost for this type of damage may be very severe considering the long time necessary for restoration and the disadvantage for the communities. Ground improvement is by no doubt opportune for singular situations (e.g. bridges) whereas its convenience must be estimated with a cost-benefit analysis for more distributed systems (e.g. embankments).

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Figure 1-9: Cracking of levees (OSU, 2011).

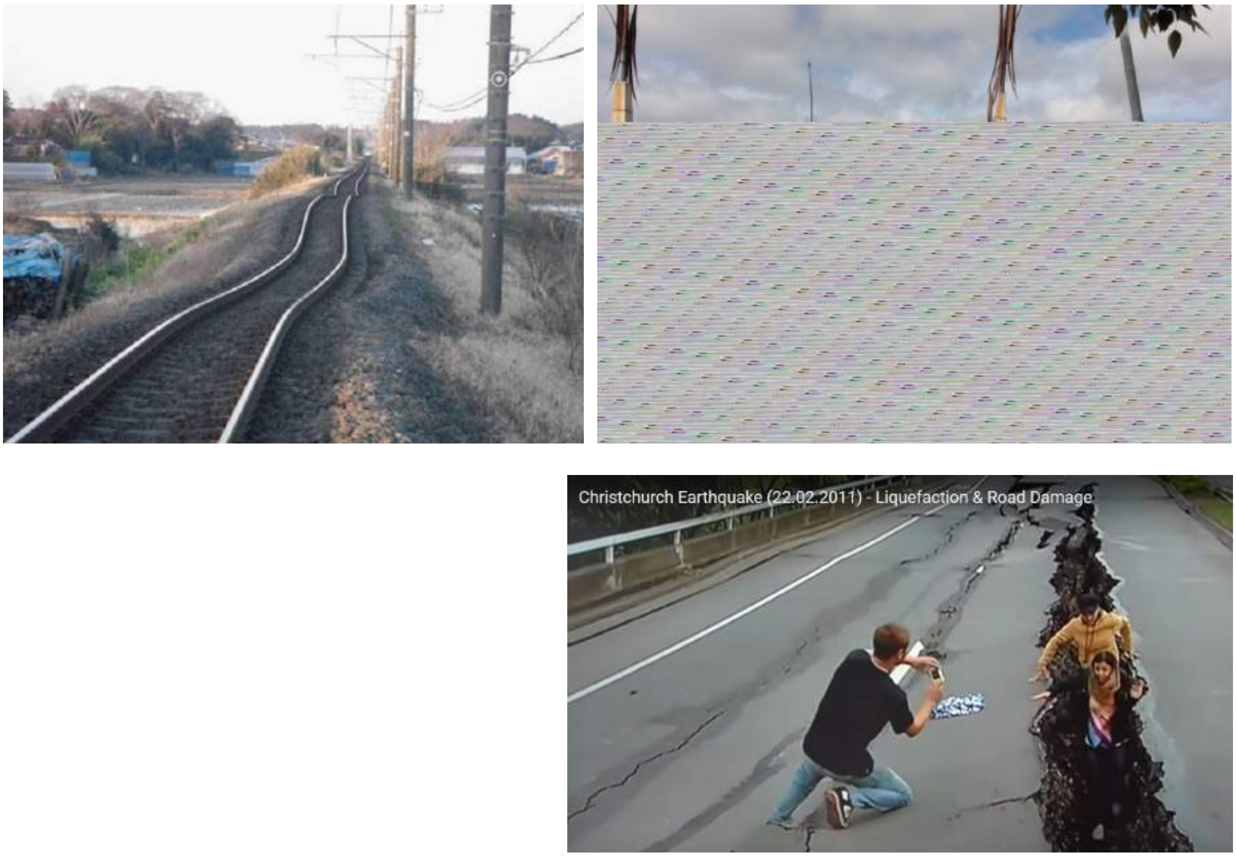

Figure 1-10: Transportation infrastructures.

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1.3 Mitigation strategies for reducing liquefaction risk and improve resilience

As shown in deliverable 7.1, ground improvement represents one of the possible solutions to mitigate risk and thus its role can be seen in the above defined holistic model including the multiscale connections outlined in Figure 1-11. Briefly recalling its fundamental steps, liquefaction is triggered when a relatively high seismic hazard combines with susceptible subsoil (loose, non-plastic in saturated conditions); the phenomenon may turn or not into damage of buildings and infrastructures depending on their physical fragility; damaged systems become progressively unable to withstand their function and thus, depending on its severity physical damage turns into lack of serviceability; the consequences for the society depend on the relevance of the function provided by the infrastructure for the served community, on the repairability/replaceability of this function or, in more general terms, on the preparedness of the community to withstand its absence.

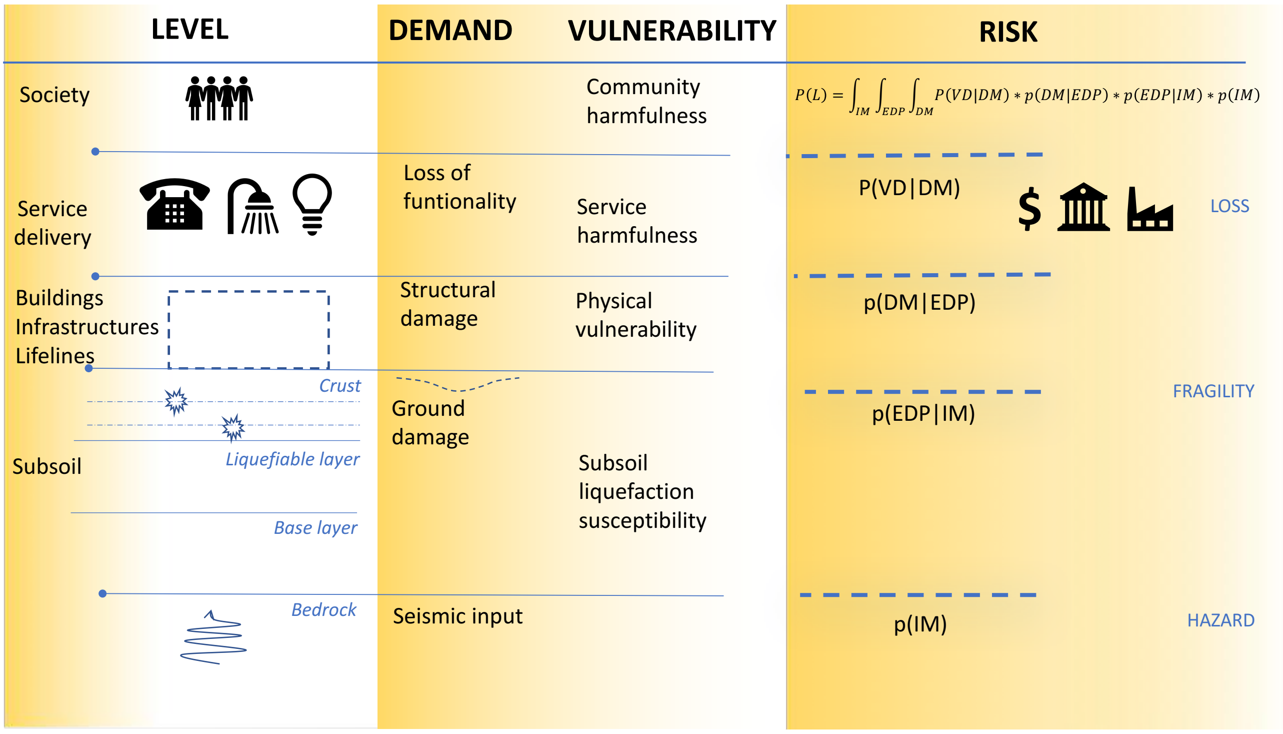

Figure 1-11: Liquefaction risk model.

Interrupting this chain is the scope of mitigation, acting separately on one component of the system or undertaking a holistic strategy aimed at reducing the overall impact on the society. The Japan Geotechnical Society (JGS, 1998) envisages three different classes of intervention (Figure 1-12), acting respectively on auxiliary facilities to support/replace the function of the concerned infrastructure, on the physical reinforcement of the structures or on the improvement of ground properties. Ground improvement proved its effectiveness in several situations that did not undergo liquefaction during the big 2011 earthquake.

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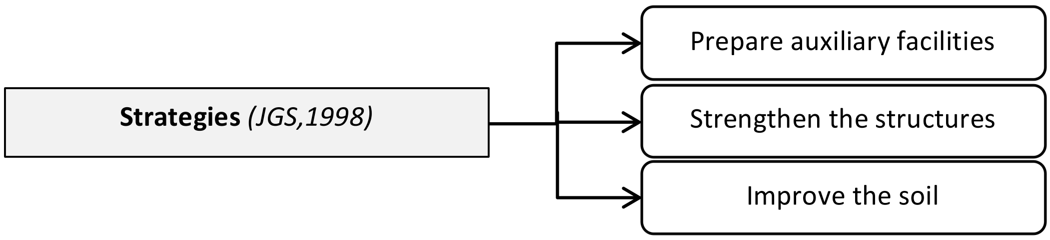

Figure 1-12: Strategies for liquefaction risk mitigation.

Therefore mitigation actions can be subdivided in two main categories, strategic or non-technical when aimed at improving the functionality of the considered system with the creation of auxiliary facilities or with a modified management to face critical situations, or technical when operating on the physical systems with structural reinforcement or ground improvement.

Resilience Assessment and Improved Framework (RAIF) to improve preparedness of community, urban planning to reduce exposure to risk, management of infrastructures to reduce indirect costs or insurance to cover expenses in the event of liquefaction can be seen as non-technical countermeasures.

Concerning technical solutions, risk on new residential buildings can be mitigated both with ground improvement or with strengthening of structure and foundation. In some sites, practical constraints like soil conditions, site access, flooding, lateral spreading potential, dewatering requirements and building type/layout may limit to undertake ground improvement. Where any of these constraints apply, foundation reinforcement has generally to be chosen for a new residential building. This option involves constructing new residential buildings on land that is vulnerable to liquefaction and didn't underwent ground improvement.

As an example, in 2012, the Ministry of Business, Innovation and Employment of New Zealand has issued a set of Guidelines entitled "Repairing and rebuilding houses affected by the Canterbury Earthquakes", organized in different parts. The logic of this document is to subdivide the area of Christchurch in three different zones assigning flats into three foundation technical categories based on the expected future liquefaction performance:

- *TC1*: Liquefaction damage is unlikely in future large earthquakes. Standard residential foundation assessment and construction is appropriate.
- *TC2*: Liquefaction damage is possible in future large earthquakes. Standard enhanced foundation repair and rebuild options in accordance with MBIE guidance are suitable to mitigate against this possibility.
- *TC3*: Liquefaction damage is possible in future large earthquakes. Individual engineering assessment is required to select the appropriate foundation repair or rebuild option.

As a general guiding principle, the code suggests building using light materials rather than heavy materials. Light construction (roof, walls and floors) significantly reduces the imposed loads on the subsoils and

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therefore the potential for liquefaction-induced settlement. For the buildings of TC3 category, the countermeasures listed in Figure 1-13 are suggested to reinforce foundations.

Type	Objectives	Dwelling Constraints	Land Constraints
Deep piles	Negligible settlement in both small and larger earthquakes	No height and/or material constraints likely	Not suitable where either *major or severe* global lateral movement likely or dense non-liquefiable bearing layer not present
Site ground improvement	Improving the ground to receive a TC2 foundation	Limits on some two storey/heavy wall types and plan configurations	Some ground improvements can be specified to accommodate *major* lateral stretch
Surface structures/ shallow foundations	Repairable damage in future moderate events	Only suitable for light and medium wall cladding combined with light roofs, regular in plan	In the absence of ground improvement, Type 1 & 2a options only suitable for minor to moderate vertical settlement and varying lateral stretch, Type 2b can accommodate up to 200 mm SLS settlement Type 3 (specific design) concepts can be designed for major lateral stretch and some for potentially significant vertical settlement

Figure 1-13: Proposed TC3 foundation types (MBIE, 2012).

Depending on the performance under Serviceability (SLS) and Ultimate Limit State (ULS) the different foundation types are proposed (Figure 1-14).

	Vertical Land Settlement (SLS)		Lateral Stretch (ULS)	
	<100 mm (Moderate)	>100 mm (Potentially Significant)	<200 mm (Moderate)	<500 mm (Major)
Type 1 – light-weight platform (standard solution) Enhanced NZS 3604 subfloor	Yes	No[1]	Yes	No
Type 2 – underslab platform (standard solution) **Type 2A** – 150 mm underslab on gravel	Yes	No[1]	Yes	Yes
Type 2B – 300 mm underslab on gravel		Up to 200 mm[1]		
Type 3 – concepts for specific design **Type 3A** – Re-levellable platform	Yes	Subject to design	Yes	Yes
Type 3B – Stiff platform				

Figure 1-14: Summary of foundation types proposed for TC3 structures (MBIE, 2012).

1.4 Ground improvement for reducing liquefaction risk and improve resilience

As repeatedly shown in the previous chapters, seismic excitation on loose sands triggers volume contraction that, in saturated conditions, turns into an accumulation of pore water pressures. When the natural drainage capacity of the system is unable to exhaust these pore pressures, the total overburden stress transfers, totally or partially, on water with the result that the effective stresses, index of the contact forces between grains, reduces or eventually nullify.

The reduction of normal effective stresses enhances shear deformation in soil subjected to deviator stress due to initial or earthquake induced loading; thereafter, the exhaust of pore pressures after the seismic event produces volumetric deformation that cumulates at the ground level as consolidation settlement.

When the effective stresses become nil, the sand matrix loses its shear resistance and starts behaving like a viscous fluid. This effect may have dramatic consequences in case of significant initial deviator stresses, like when sandy layers are sloped (flow failure) or in the active zone of the foundation below heavy superstructure. The presence of an upper impermeable crust contributes on one side to trigger the phenomenon reducing the subsoil drainage capacity, but its increased thickness attenuates the effects at the ground level.

The above phenomenology can be simply schematised with the chain of mechanisms described in Figure 1-15, determined by the simultaneous presence of different mechanical conditions. Ground improvement aims to interrupt this chain modifying one or more triggering conditions to prevent the onset of liquefaction.

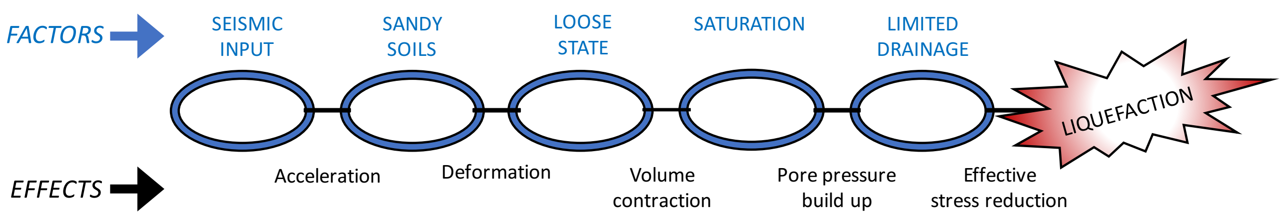

Figure 1-15: Chain of mechanisms determining liquefaction.

Being the phenomenon ruled by the concurrence of different factors, i.e. non plastic soil in a loose state, saturation, hampered drainage, various mitigation techniques may be carried out to interrupt the chain of mechanisms responsible for the phenomenon. Soil susceptibility may be reduced decreasing the contractive tendency upon cyclic loading, e.g. by means of dynamic compaction (Mayne, 1984), vibratory techniques (Kirsch & Kirsch, 2016) or blasting (Lyman, 1941) or adding a finer plastic material (El Mohtar et al., 2013) to reduce the mobility of grains upon shaking. Triggering may be avoided by preventing the excess pore pressure build-up with induced partial desaturation (Mele et al., 2018) or facilitating its exhaust with horizontal and vertical drains (Chang et al., 2004). Other possible countermeasures consist in limiting the impact on the superstructure by reinforcing foundations with piles, columnar or lattice wall inclusions created with jet grouting (Yamauchi et al., 2017), deep soil mixing (Nguyen et al., 2012) or stone columns

(D'Appolonia, 1954). Reinforcements have the twofold scope of reducing shear strains in susceptible soils and transfer loads to deeper non liquefiable strata.

A list of possible ground improvement solutions describing principles, drawbacks and costs is provided by the JGS (2011) (Table 1-1). From a purely mechanical viewpoint, the function of ground improvement can be classified as follows, being the single ground improvement technique able to reach one or more of the following goals:

- *Densification*: reducing the volume contraction tendency of the soil upon shaking;
- *Stabilisatio*n: reducing the mobility of grain and volume contraction tendency of the soil upon shaking;
- *Drainage*: reducing the pore pressure build-up;
- *Desaturation*: preventing the pore pressure build-up;
- *Reinforcement*: reducing the shear strain into liquefiable soil and transferring loads to more competent strata.

As for any application of ground improvement, suitable techniques should be chosen scrutinizing the problem from different perspectives, i.e. not only considering mechanical effectiveness, but also other factors including cost-effectiveness, executability, possible side effects and environmental restrictions. A cost-benefit analysis is normally the best rule to make choices, but the scenario changes significantly if ground improvement is addressed to new structures or to the remediation of previously existing ones.

For new structures there is generally less limitation on the choice of the ground improvement solution as the disturbance of previously existing conditions is rarely of concern. An exception is represented by those situations where the structure to be built is located close to existing buildings or infrastructures. In this case, side effects dictated by invasive ground modification and by its execution must be carefully assessed. There are techniques inapplicable in some context because of the large dimension of the equipment, incompatible with the available space, or due to side effects (movements or vibration) induced by installation. Minimisation and/or monitoring of side effects should accompany application of the technique in any case.

Environmental restrictions represent the last non negligible side effect that must be considered in the application of ground improvement. Some techniques may produce wastes incompatible with sensitive environment (e.g. spoil) of modify the local conditions (e.g. groundwater regime) in a way that becomes intolerable for the regulation.

Apart from the above recalled peculiar cases, the adopted solution becomes a compromise between desired degree of safety and cost-effectiveness. An appropriate design should be able to tune the intervention on the expected intensity of the shaking and on the effects on the structural response of the building over its lifecycle (Performance Based Design). Provided this issue is possible, the other non-secondary issue concerns cost-effectiveness. From this viewpoint, the variety of possible scenarios prevents to establish a single rule, but ad hoc analyses must be carried out for each single case. For instance, the situation can be largely different even within a single category (e.g. buildings) depending on the structural characteristics, vulnerability, covered functions and lifetime of the considered structure. Assessing risk, with a reliable estimate of the direct and indirect costs induced by damage and of the occurrence probability of the seismic events, estimating the advantage given by mitigation and comparing it with the sustained cost is generally the most appropriate way to proceed. This comparison can be performed on an annual basis as described in deliverable 7.1 (see chapter 6), but with the indubitable principle that ground improvement becomes more and more advantageous with the covered time length (lifetime of the structure). This analysis leads to consider advantageous and undertake mitigation, even with massive ground improvement intervention, in case of important structures having a long lifecycle.

The balance changes dramatically for structures and mitigation intervention having just a temporal function. In this case, the reduction of benefits leads to consider ground improvement less advantageous, if not cost-ineffective. The above principle applies somehow to pre-existing structures, whose lifetime has partially expired. However, in this case the lost advantage may be compensated considering the higher direct and indirect costs necessary to demolish the structure before rebuilding or, moreover, by other possible values (e.g. historical, cultural) possessed by the structure that would be preserved with mitigation.

Still from the viewpoint of cost-effectiveness, another important issue regards the territorial extension of the considered asset. Differently from buildings or from those infrastructures having a restricted geographical location, horizontal lifelines (e.g. water and gas distribution) are distributed over large areas. This situation may render ground improvement particularly costly and the benefits given by local treatments limited. In this case, an overall strategy should be then conceived limiting ground improvement to the most critical zones and combining intervention with a rational design of the network aimed at reducing malfunctioning and risk. This situation is more complex for existing lifelines, but the above strategy cab be included in the long-term management plan of the network. Examples of calculation are provided in chapter 6 of Deliverable 7.1.

Table 1-1: Classification of ground improvement methods for soil liquefaction countermeasure (JGS, 2011).

Basic principle	Technique	Outline	General cost (JPY/m³)	Noise, Vibration	Disturbance of soil	Machine size	Displacement control
Densification	Sand compaction method	Compacted sand piles are installed by driving down and extracting up a vibrating steel shaft.	1,000 ~ 2,000	Higher	More	Big	More
	Vibro compaction	A vibrator implemented at tip of extension tube carries out compaction at designated depths.	1,000 ~ 2,000	Higher	More	Big	More
	Quiet compaction	Casing pipe is penetrated and withdrawn a little at a time with rotational force to achieve soil compaction.	2,000 ~ 3,000	Lower	More	Big	More
	Compaction grouting	A very stiff grout mix, with an almost zero slump, is injected under relatively high-pressure to compact surrounding soil.	10,000 ~ 15,000	Lower	More	Small	More
Drain	Gravel drain	Gravel piles are vertically installed into the ground to accelerate the dissipation of excess pore water pressure induced by seismic events.	2,000 ~ 4,000	Lower	Less	Big	Less
	Artificial drain	Artificial drains such as PVD are installed into the ground to accelerate the dissipation of excess pore water pressure induced by seismic events.	2,000 ~ 4,000	Lower	Less	Middle	Less
Replacement	Pre-mixed soil	Stabilized grounds are constructed using soils mixed with chemical agent such as cement prior to placing in dry or slurry form.	3,000 ~ 4,000	Lower	More	Big	More
	Light weight soil	Stabilized grounds are constructed using soil mixed with cement along with foam or light weight material.	8,000 ~ 12,000	Lower	More	Big	More
Solidifying	Deep soil mixing	In-situ soils are mixed with a binder, such as cement or lime supplied in dry or slurry form, using rotating blade.	4,000 ~ 6,000	Lower	Less	Big	More
	Jet grout	A high pressure fluid is jetted out from the tip of extension rod, to allow in-situ soils eroded and mixed with cement grout.	20,000 ~ 60,000	Lower	Less	Small	More
	Chemical grouting	Chemical grouts, composed of additives such as sodium silicate or polymer, are injected into ground to improve its strength or lower its permeability.	20,000 ~ 30,000	Lower	Less	Small	Less
	Permeation grouting	Durable chemical grouts, specially manufactured to remove anti-durable factor, is injected into ground to increase liquefaction resistance.	20,000 ~ 30,000	Lower	Less	Small	Less
Reinforcing	Additional piles	Additional pile installation or reinforcing around pile heads increase structural resistance.	(20,000 ~ 50,000)	Lower	Less	Small	More
	Sheet pile reinforcing	Sheet piling surrounding structures works to protect its foundation; also effectively works as a stopper of the excess pore water pressure during seismic events.	(20,000 ~ 50,000)	Lower	Less	Small	More
	Solidifying reinforcing	Solidifying soils around foundations works, in some degrees, as structural reinforcing.	(20,000 ~ 50,000)	Lower	Less	Small	More

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Whatever the adopted technique, the international standards (e.g. EN1997-1, 2004) state the following basic principle: "the effectiveness of the ground improvement shall be checked against the acceptance criteria by determining the induced changes in the appropriate ground properties". Although general, this sentence features a strategy that may adopted to drive in a consistent framework the three phases of ground improvement application, design, execution and control of treatments (Croce et al., 2014). Therefore, depending on the scope of ground improvement, the hydraulic/mechanical performance should be identified with a property (or more than one), originally inadequate and modifiable with ground improvement, and its adjustment motivated with quantitative design analyses.

Usually ground improvement techniques bring advantages, producing positive modification to the ground properties. Sometimes they are accompanied by limitations and drawbacks that must be seriously considered as they may hamper the effectiveness of the technique. One of the main aspects to be considered is the applicability of the candidate technique on existing structures. Some techniques produce in fact significant disturbance to the surrounding soil at a point that their execution is impossible near or below existing buildings or infrastructures, while others can be conveniently applied due to low invasiveness.

Apart from the induced modification, another relevant issue concerns the execution of the technique, i.e. the setting of the optimal treatment parameters necessary to achieve a prescribed goal. Each technique is achieved with a treatment that can be characterised with a set of geometrical and mechanical parameters (e.g. the intensity and duration of shaking and the spacing between boreholes for vibratory compaction, the injection pressure and spacing between holes for grouting etc.).

The choice of parameters dictates the cost of the treatment, which is a relevant issue to judge economic convenience. In some cases, charts exist to define the above parameters starting from the characteristics of the soil to be treated and to the desired goal. In some other cases, a significant degree of uncertainty remains that must be necessarily solved with an experimental assessment (field trial) to be performed before treatment is executed. This preliminary activity has the twofold scope of ensuring the feasibility of treatment and establishing the best procedures for execution.

Finally, but not less important, the effectiveness of ground improvement must be proven with simple, fast, reliable and non-invasive control tests. The controlling technique must be chosen depending on the modification applied to the soil. Most commonly, penetration resistance tests (SPT, CPT) executed prior and after treatment are suitable for assessing improvement, also because they are the widely used for liquefaction assessment. Sonic tests based on the propagation of compression and shear waves can be also used, provided the technique determines an increase of the propagation velocity.

The main factors characterising the use of a ground improvement technique for liquefaction mitigation can be synthetically described in charts, an example of which is given in Table 1-1. Normally, ground improvement requires a protocol procedure to choose, design and apply the selected technique. For risk assessment the fundamental choice concerns the economical convenience of mitigation, that should be estimated performing a cost/benefit analysis as described in the flow chart of Figure 1-16. One of the main variables that must be known and considered to determine a sufficiently approximate estimate of costs is the volume of subsoil to be treated. This information, together with the unit cost of treatment (typically expressed as cost/volume) forms the total cost of mitigation.

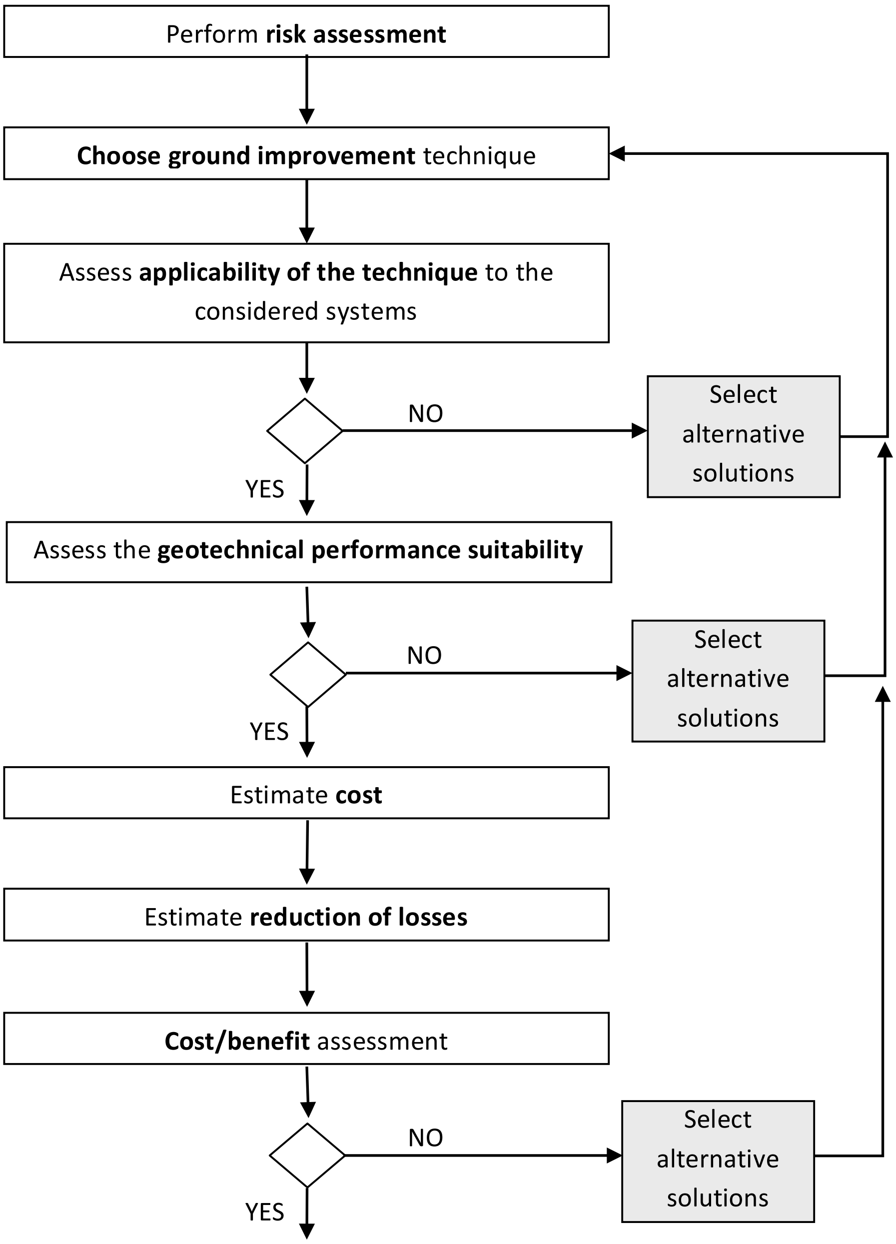

Figure 1-16: Flow chart describing mitigation analysis for risk assessment.

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1.5 Existing codes and guidelines

The most suitable technique should be selected considering the best compromise of effectiveness, technical feasibility, costs and environmental sustainability. A significant effort to standardize design procedures has been made in countries that have suffered severe liquefaction (e.g. Han, 2015; JGS, 1998; Kirsch & Bell, 2013). The complexity of standardization in Japan is depicted in Figure 1-17 that reports the specifications adopted in Japan by institutions responsible for different infrastructures (JGS, 2011). It is immediate to see that the codes differ from each other, being criteria based on safety factor FL, liquefaction potential index PL or limit SPT blow counts alternatively adopted. Once limits are exceeded, remediation criteria are required by the promoting institutions. An attempt to unify the approach to liquefaction mitigation in US is proposed in SCEC (1999). Here a procedure is introduced to quantify hazard and implement ground improvement techniques for mitigation. According to this procedure, mitigation projects should contain the following documents:

1. Project description;
2. Description of the geologic and geotechnical conditions at the site;
3. Evaluation of the site-specific liquefaction hazard;
4. Recommendations for appropriate mitigation measures;
5. Logs of field explorations (SPT and CPT);
6. Description of laboratory tests on soil samples and summary of test results;
7. A summary of the assumptions used in analysis
8. Calculation and results.

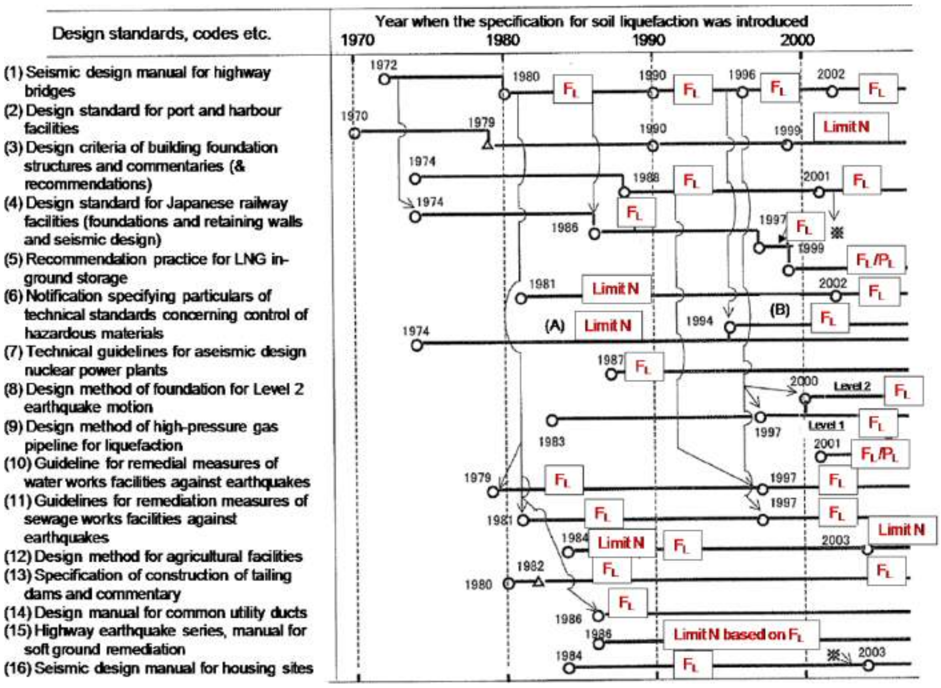

Figure 1-17: Specifications for countermeasures against liquefaction in Japanese design standards and codes for infrastructures (JGS, 2011).

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Following the same strategy, a more detailed approach has been recently developed by the New Zealand Geotechnical Society (NZGS, 2017). According to this guideline, analyses should be aimed at progressively evaluating the liquefaction susceptibility of the subsoil, triggering caused by likely events and effects on the structures. Assessment is thus articulated with the following subsequent steps:

1. Determine performance requirements for the building and foundation system;
2. Assess site seismicity, local seismic response and susceptibility to liquefaction/lateral spreading based on geotechnical investigation;
3. Assess severity and free field effects of liquefaction at the site considering lateral spreading hazard and potential for differential lateral displacement across the building footprint;
4. Assess the effects of liquefaction on the structure and compare them with the performance criteria.
5. Consider structural options to reduce susceptibility to damage from liquefaction or, where they are not sufficient, consider ground improvement options;
6. Select suitable methods for ground improvement;
7. Design the extent (depth and size in plan) of improvement needed to meet design objectives considering soil-ground improvement-structure interaction;
8. Design the size and arrangement of the ground improvement determining material requirements where necessary;
9. Determine quality control (QC) and quality assurance (QA) requirements.

Despite this document does not enter in the details of the analysis for each ground improvement technique, it certainly represents the most complete and up to date methodology for the design of ground improvement to mitigate liquefaction. The effectiveness of the ground improvement techniques is checked against quantitative acceptance criteria based on the performance requirements of the buildings, identifying in this way the relevant ground properties to be modified.

1.6 Ground improvement and liquefaction in Eurocodes

In the European Union, the topic of ground improvement is treated by two different types of standards:

- "Execution of Special Geotechnical Works" produced by Technical Committee CEN TC 288;
- Geotechnical Design Eurocode 7 (EN 1997), drafted by Sub-Committee CEN TC250/SC7.

The execution standards provide definitions and rules to contractors in order to obtain safe and reliable products. They define construction procedures including testing, control methods and required material properties. Execution standards are available for the following techniques:

- Grouting (EN 12715);
- Deep Mixing (EN 14679);
- Ground Treatment by Deep Vibration (EN 14731);
- Jet Grouting (EN 12716);

- Vertical drainage (EN 15237).

Indication on design is briefly recalled in the execution standards, but the topic is thoroughly covered by the codes for design EC7. Its current version contains a very brief chapter (5.5) on ground improvement and reinforcement and provides only generic principles. The need for a more extended and specific chapter on ground improvement has thus been recognized by the Sub-Committee CEN TC250/SC7 and a new version is foreseen in the revised geotechnical design Eurocode, expected in 2020. The debate on this new edition started in 2012, when the SC7 created a specific working group on Ground Improvement (Evolution Group EG14). This Evolution Group has issued its final report on December 2015, providing a draft of the forthcoming chapter on Ground Improvement to be developed by the Project Team who has now the responsibility of writing the new chapter on Ground Improvement Design.

The discussion on Ground Improvement Design Rules has been very lively from the beginning, and still is, starting from the definition of the term “Ground Improvement” and proceeding with the interaction between technological issues and design principles and/or methods. However, bearing in mind that the new code has yet to be written, it seems useful to report the main indication provided by the Evolution Group EG14 who has stated that the design of ground improvement can be undertaken by two possible methods (Figure 1-18):

1. Diffused Ground Improvement;
2. Discrete Ground Improvement.

Diffused Ground Improvement design is applicable when the behaviour of the improved ground can be modelled by conventional soil or rock models. In this case the designer should evaluate the change of ground properties (i.e. cohesion, friction angle, permeability, etc.) and consequently define “Improved Characteristic Values”. Design rules for foundations, retaining structures, embankments, slopes etc. are then applied according to the relevant sections of the Eurocode. The Improved Characteristic Values may be evaluated using testing, empirical methods, comparable experience or analytical/numerical modelling. Discrete Ground Improvement design can be applied when ground improvement relies on inclusions, i.e. discrete elements created in the ground, physically disconnected from any structure, provided with prescribed geometry and mechanical properties. The overall performance of the improved ground is calculated by considering separately the characteristics of the inclusions and their interaction with the soil/rock. In such a case, design rules for foundations, retaining structures, embankments, slopes etc. are applied according to the relevant sections of the Eurocode. In the present version of Eurocode 7 (ENV 1997) it is generally stated that the “effectiveness of the ground improvement shall be checked against the acceptance criteria by determining the induced changes in the appropriate ground properties”.

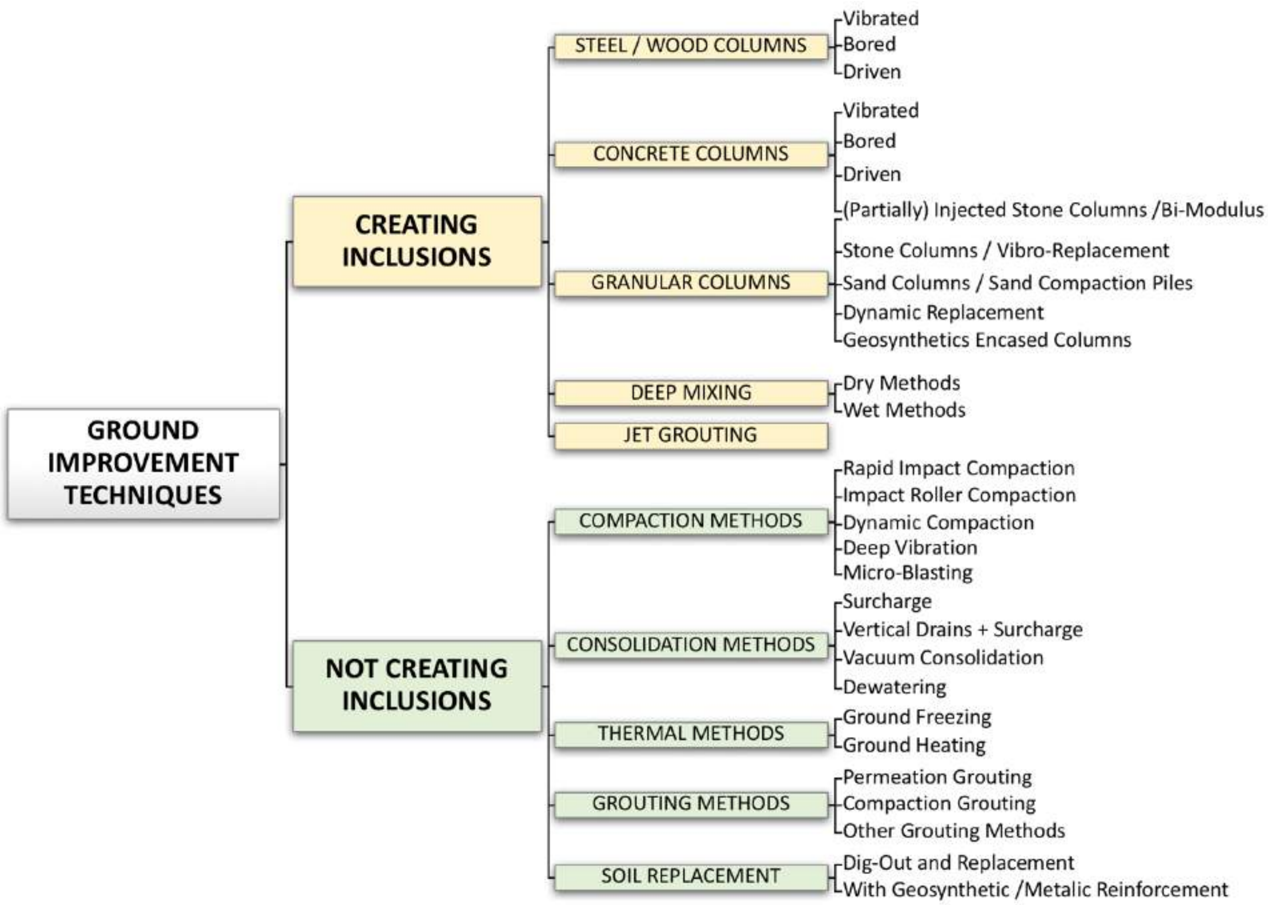

Figure 1-18: Ground improvement techniques defined by the evolution group EG14.

Although very general, this statement requires to implement a design method for structures likely to undergo ground improvement, to identify weak relevant properties of the soil to be modified by ground improvement, to fix acceptance criteria and appropriate experimental methodologies to assess the quality of execution and the performance of improved soil. While the performance requirements for foundations are generally defined in terms of ULS and SLS in section 6.2 of the Eurocode 7, the assessment of performance against liquefaction is only recalled for the SLS of spread foundations (section.6.6.4 Vibration analyses) and in the ULS of earth retaining structures (section 9.7).

Specific liquefaction analyses are dealt in section 4 (Requirements for siting and for foundation soils) and in Annex B (Empirical charts for simplified liquefaction analysis) of the Eurocode 8 part 5. Assessment is aimed at evaluating susceptibility of the considered subsoil and triggering caused by the earthquake. Susceptibility is defined considering the simultaneous existence of the following conditions:

- saturated sandy soils at depths greater than 15 m from ground surface, peak ground acceleration a_g higher than 0.15g and at least one of the following conditions:
  - the sands have a clay content lower than 20% with plasticity index PI > 10;

- the sands have silt content lower than 35% and a normalised SPT blow count value $N_{1(60)} < 20$;
- sands are clean and have normalised SPT blow count value $N_{1(60)} < 30$.

Once susceptibility is ensured, triggering is evaluated by comparing the cyclic stress ratio (CSR) induced by earthquakes with the cyclic resistance ratio (CRR). The former is expressed by the following formula:

$$CSR = \frac{\tau}{\sigma'_{vo}} = 0.65 \cdot \frac{a_g}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} \qquad \text{Equation 1-1}$$

where a_g is the peak ground acceleration evaluated considering the seismic hazard and local site conditions, σ_{v0} and σ'_{v0} are respectively the total and effective overburden pressure. The cyclic resistance ratio CRR is evaluated with empirical charts illustrating field correlation with different types of in situ measurements (Figure 1-19 shows an example extracted from the Annex B).

This assessment is carried out for depths lower than 20 m and the response is considered negative if CSR>0.8·CRR, i.e. assuming a safety coefficient equal to 1.25.

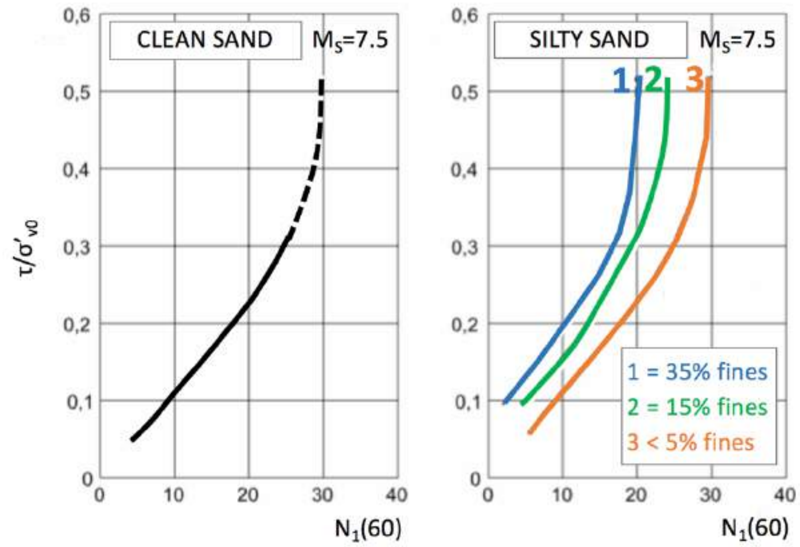

Figure 1-19: CRR as function of $N_{1(60)}$ for M_S=7.5 earthquakes (modified from EC8 part 5).

Then in chapter 4.1.4 (potentially liquefiable soils) it is stated that *"if soils are found to be susceptible to liquefaction and the ensuing effects are deemed capable of affecting the load bearing capacity or the stability of the foundations, measures such as ground improvement and piling shall be taken to ensure foundation stability"*. It is also specified that *"ground improvement against liquefaction should either compact the soil… or use drainage to reduce the excess pore-water pressure generated by ground shaking"*.

It is noted that the above criterion quantifies the triggering of liquefaction but does not consider extent and depth of the liquefiable layer that would certainly play a predominant role on determining different effects

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at the ground level and on the upper structures. Additionally, general requirements are given for ground improvement without referring them to the performance of structures.

For this and other limitations, EC8 is undergoing a thorough revision, as all the design Eurocodes are, and it is hoped that some specific guidelines on the use of ground improvement against liquefaction will be incorporated in the revised edition.

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2. GROUND IMPROVEMENT TECHNIQUES FOR LIQUEFACTION MITIGATION

2.1 Principles and techniques

In geotechnical engineering, ground improvement techniques are very useful to modify the characteristics of the subsoil and provide a better performance both for new or pre-existing structures.

Ground improvement may have several purposes, including: increase of strength to improve the bearing resistance of a foundation soil; reduce of the compressibility to control settlements; reduce permeability to stop the water flow or isolate contaminated sites; increase drainage to accelerate the consolidation process or to assist other techniques; compensate for ground movements resulting from excavation; mitigate the mitigation of liquefaction potential. Focusing specifically on the latter purpose, ground improvement can be implemented to interrupt the schematic chain reported in Figure 2-1, where the different factors concurring to generate liquefaction are coupled with the mechanical effects.

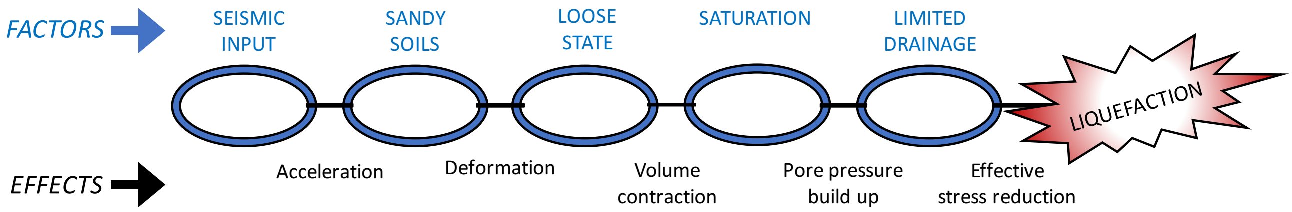

Figure 2-1: Schematic representation of liquefaction.

Nowadays there is a wide range of ground improvement techniques grouped in different types of classification in the literature, as summarised in Table 2-1.

Table 2-1: Ground improvement classifications.

REFERENCE	GROUND IMPROVEMENT CLASSIFICATION
Michell (1981)	Technologies based classification (compaction, consolidation, grouting, stabilisation using admixtures, thermal stabilisation, reinforcement)
Van Impe (1989)	Permanent - Temporary improvement classification
Hausmann (1990)	Type of modification (hydraulic, mechanical, physical-chemical, inclusions and confinement)
Evangelista (1995); Burghignoli (1995); Flora & Lirer (2011)	Ground improvement – Ground reinforcement
CEN TC250/SC7-EG14 (2015)	Creating inclusions – Not creating inclusions
Kishida et al. (2009); NZGS (2017).	Replacement, densification, solidification, reinforcement and drainage

The classification adopted in the present document is based on the effects produced on the soil:

- *Densification*;
- *Stabilisation*;
- *Drainage*;
- *Desaturation*;
- *Replacement*;
- *Reinforcement*.

Densification methods are among the most common ground improvement techniques; they determine a rearrangement of the particles in a denser configuration, without changing the original composition of the soil. The main mechanical effect is an increase of strength and stiffness, a reduced tendency to contract upon cyclic loading and, finally an increase of the liquefaction resistance. Among the main limitations of this class of techniques are the noise and vibration produced during treatments, that makes this solution not suitable for existing structures.

Stabilisation methods imply the filling of the in-situ soil pores with cementitious material or other additives to freeze the original structure of the soil. Treatment bonds the particles reducing their mobility or fills the pores reducing in this way the tendency of soil to contract and preventing the onset of excess of pore pressure. These methods are minimally invasive, as their implementation cause limited disturbance, vibration and noise, for this reason are suitable for application below or near existing structures. On the other hand, they are not particularly convenient from an economical viewpoint, are not applicable to low permeability soils and leave some uncertainty on the volume of soil really treated.

Drainage is normally used to speed up consolidation in fine grained materials; in liquefiable soils this principle can be similarly used to dissipate the excess of pore water pressure generated during the earthquake. Technical solutions include arrays of vertical drains located at a prescribed distance, when ground improvement is referred to existing structures, or horizontal drains applicable below existing buildings. The main limitation is related with the inclusion of drains below existing structures that requires uncommon equipment like directional drilling and with some uncertainty in the effectiveness of drainage ruled by the permeability of the surrounding soil.

Desaturation methods are the new frontier of mitigation against liquefaction. Treatment is based on the formation of gas bubbles into the soil, by means of chemical, biological or mechanical methods, with the aim of reducing the stiffness of the interparticle fluid an inhibit the onset of excess pore pressures. Treatments appears to be particularly cost effective, although affected by some uncertainties on their durability and on the possibility to control the degree of saturation in the soil.

Replacement methods involve the removal of in-situ materials and replacement with more suitable soils. It is very invasive and can be only applied to shallow portions of the subsoil.

Reinforcement methods involve the construction of relatively stiff inclusions into the soil. Columns created with different techniques (pile installation, deep mixing, jet grouting) can be placed individually in the soil,

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as a regular grid or to form lattice walls). In liquefiable soils, this typology of treatment aims to reduce the shear deformation of the soil and thus decrease the development of excess of pore water pressure during the earthquake. Furthermore, in the extreme event of liquefaction, reinforcements may work as a support for the overlying structures. This methodology can be applied on foundation before the construction of new buildings or on small structures.

Although other types of ground improvement techniques are currently available, this document focuses on ten more used techniques:

- *Deep dynamic compaction;*
- *Vibro compaction;*
- *Blasting compaction;*
- *Compaction grouting;*
- *Low pressure grouting;*
- *Earthquake drains;*
- *Induced partial saturation;*
- *Vibro replacement;*
- *Deep mixing;*
- *Jet grouting.*

Since some of the considered techniques can produce more than one effect on the ground, it is difficult to classify them rigidly. Therefore, the following matrix (Table 2-2) is created to summarise the main effects induced by each technique.

The techniques shown are then summarised in the Appendix A, in the *Technical Charts* which show the main features of each technique including the suitable types of soil, the benefits and drawbacks, the treatment parameters, QA and QC.

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Table 2-2: Matrix of techniques-effects.

		EFFECTS							
		Densification	**Stabilisation**	**Drainage**	**Desaturation**	**Replacement**	**Reinforcement**	***CREATING INCLUSION***	***NOT CREATING INCLUSION***
TECHNIQUES	**Deep dynamic compaction** * for Deep dynamic replacement	✓				✓*			✓✓
	Vibro compaction	✓							✓
	Blasting compaction * for treatment in soft fine-grained soils	✓✓		✓*✓		✓	✓	✓*✓	✓✓
	Compaction grouting	✓✓		✓		✓	✓✓	✓✓	✓
	Low pressure grouting	✓	✓	✓		✓	✓	✓	✓✓
	Earthquake drains	✓		✓✓		✓	✓	✓✓	✓
	Induced partial saturation	✓		✓	✓	✓	✓	✓	✓✓
	Vibro replacement	✓		✓		✓	✓	✓✓	✓
	Deep mixing		✓✓				✓✓	✓✓	✓
	Jet grouting		✓✓				✓✓	✓✓	✓

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2.2 Deep dynamic compaction

2.2.1 Principle

Deep dynamic compaction (DDC) is a technique that densifies the soil by means of high energy tamping, therefore classified as an *impact method* for dynamic compaction by Kirsch & Kirsch (2010). A weight lifted by a conventional crane is repeatedly dropped on the ground surface densifying the soil at depth (Figure 2-2). The dropped weight is usually a toughened steel plate, a box-steel and concrete, or a reinforced concrete mass. The number of drops and the height of drop depending on the desired compaction level and the type of soil to be treated.

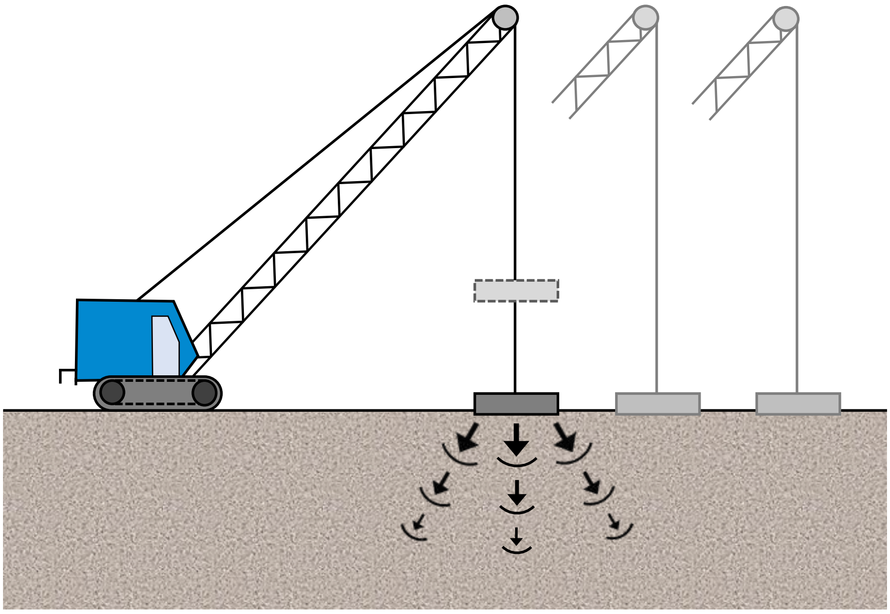

Figure 2-2: Deep dynamic compaction procedure.

The dynamic impact caused by DDC creates compressive and shear waves that propagates through the soil mass attenuating with distance. This principle is used to densify the loose soil and increase strength and stiffness. In dry conditions the physical displacement of the soil particles results in a void ratio reduction and an increased relative density. In liquefiable soils, DDC is capable to reduce or eliminate the liquefaction susceptibility breaking the "loose state" link in Figure 2-3. In particular, for granular materials below the water table the liquefaction mitigation can be performed by adopting two different approaches: inducing liquefaction during treatment or avoiding it (Slocombe, 2013). In the first case, the drops induce increasing pore water pressure and subsequent liquefaction, that in conjunction with the effective surcharge results in a denser configuration after dissipation of pore water pressure. In the second case, soil compaction is achieved by displacement, preventing the development of high pore water pressure in order to avoid liquefaction; in this case, a small number of drops and low drop height are used.

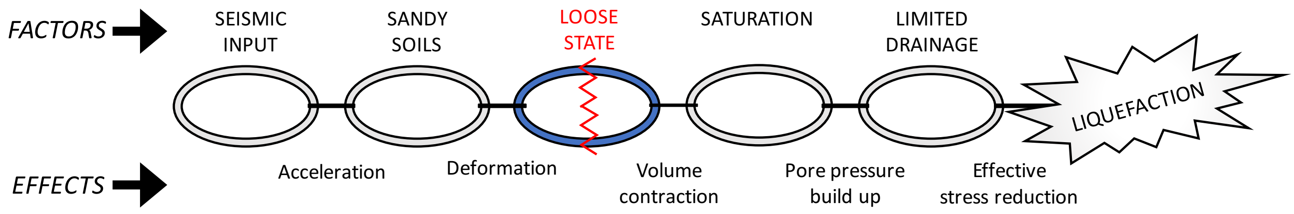

Figure 2-3: Deep dynamic compaction for liquefaction mitigation.

In DDC three or more layers of treatment are generally considered:

- the deepest layer corresponding to the first tamping pass (wide grid, high number of drops, full drop height);
- the middle layers corresponding to intermediate tamping passes (intermediate grid, lesser number of drops and drop height);
- the surface layer corresponding to the last tamping pass (continuous tamping, small number of drops, low drop height).

The maximum depth of treatment is of the order of 10 meters (Schaefer et al., 2017a).

2.2.2 Applicability

DDC is suitable for the treatment of natural soils and fills and usually performed for granular materials. The extension to finer soils is called *dynamic consolidation*, characterized by expulsion of water for the dynamic loading. Moreover, in the case of soft shallow soils, the treatment can be performed by adding imported granular materials in order to form columns (*dynamic replacement*).

According to Slocombe (2013), the technique is best suited to granular materials. Mixed soils are more suitable than cohesive soils, and the lower performance is for refuse-contaminated soils treatment. Schaefer et al. (2017a) classify the suitability of soil with regard to this technique based on the range of grain size distribution (Figure 2-4). If the soil fall into zone 1 it is most suitable for DDC treatment, the soil of zone 2 is less suitable, the soil in zone 3 is not recommended for DDC treatment.

The effective depth of treatment, D, originally proposed by Menard using the energy input expression, can be related to the weight of the tamper, W, the drop height, H, and to the empirical factor, n, as reported in Equation 2-1.

$$D = n \cdot (W \cdot H)^{0.5} \qquad \text{Equation 2-1}$$

where:
D = effective depth of treatment (m)
W = tamper mass (ton)
H = drop height (m)
n = empirical factor n≤1; in general, n=0.4÷0.8 and it is equal to 0.5 for most soils (Schaefer et al., 2017a)

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Furthermore, Slocombe (2013) reported that higher factors as 0.9 are suitable for shallow loose soils and lower factor as 0.25 for deep treatments (Figure 2-5).

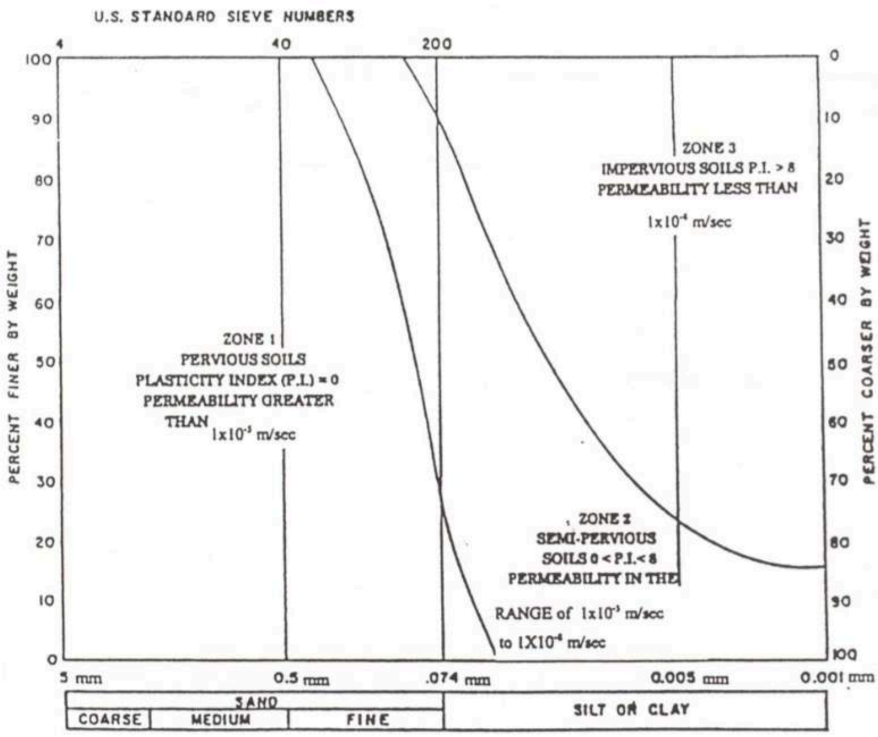

Figure 2-4: Suitability to deep dynamic compaction (from Schaefer et al., 2017a).

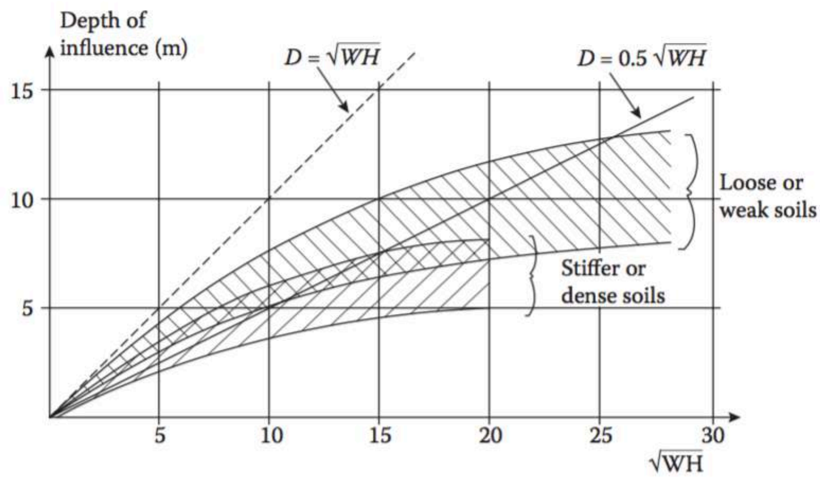

Figure 2-5: Effective depth of treatment (from Slocombe, 2013).

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The treatment is performed using a design grid pattern and the level of energy applied can be evaluated by means of Equation 2-2. In particular, for treatments involving different drop heights, tamper masses and/or grid spacing, the average applied energy can be evaluated as the sum of the different levels.

$$E = \frac{W \cdot H \cdot N \cdot P}{(grid\ spacing)^2} \qquad \text{Equation 2-2}$$

where the grid spacing is in m, and:
E = average applied energy (ton·m/m^2)
N = number of drops (each position)
W = tamper mass (ton)
H = drop height (m)
P = number of passes

Since the apparatus require to be supported by a free-draining surface, if the surface consists of cohesive soils, a granular carpet is needed.

DDC is a considered a good solution from the environmental viewpoint since it does not imply the use of artificial additives (cement, lime or chemical products). During treatment it is recommended the use some protections as moveable screens to intercept flying debris caused by the impacts.

2.2.3 Limitations and drawbacks

The energy transmitted by the treatment can be reduced by the presence of obstructions in the compressible layer, therefore additional numbers of blows could be required.

Deep dynamic compaction apparatus requires sufficient headroom, in particular for the crane.

DDC causes vibrations and noise. Therefore, the use of this technique is preferred in the case of new structures, while close to existing structures is not recommended. The level of disturb near the treatment site has to be verified and monitored. Ground vibrations could be reduced by adopting a higher number of drops combined with lower drop height, lower weight or by means of a cut-off trench. The latter one, intercepting the waves, can reduce the vibration level by half (Slocombe, 2013).

Moreover, DDC can cause not negligible lateral movements dangerous for existing infrastructures (Schaefer et al., 2017a).

Where DDC is used in order to reduce large pores, the extension of the pores has to be evaluated in order to avoid the apparatus falling into the void during the treatment.

Particular attention is required in the presence of contaminants, barrier layers have not to be damaged and possible changes in water level have to be considered.

2.2.4 Treatment parameters

The most important treatment parameters of deep dynamic compaction are:

- tamper mass and size;
- number of passes;
- applied energy;

and, for each tamping pass:

- grid spacing;
- drop height;
- number of drops.

2.3 Vibro compaction

Vibro compaction can be divided into two categories, *deep* and *shallow*, depending on the depth of the layer to be treated. *Deep-vibro compaction* requires the use of probes that penetrate the ground and induce vibrations that densify the soil to be treated, while *shallow - vibro compaction /replacement* requires replacing the native shallow soil with soils with better characteristics that are compacted with the use of rollers.

2.3.1 Deep – vibro compaction

2.3.1.1 Principle

Deep vibro compaction, also known as vibro compaction (VC), is a compaction technique applied to granular soil that involves the use of a deep torpedo vibrator probe (Kirsch & Kirsch, 2010). The probe is lifted by a crane and consists of a motor with a rotating eccentric mass, that can be varied to increase the power of the vibrating action. Once the probe reaches the required depth, exploiting its weight and facilitating penetration with vibrations assisted by air and/or water jetting, treatments are carried out at prescribed intervals retracting the vibrator to the top as summarised in Figure 2-6. The horizontal vibration of the probe generates an artificial motion that propagates radially and attenuates with the distance from the probe. With this action the intergranular forces between the grains are temporarily reduced and gravity may play its role in enabling a denser configuration of the soil. Relative densities of the soil in the range of 70-85% are normally reached. During compaction, a backfill material (clean granular or in situ materials are normally used with this scope) is added at the top to fill the crater caused by compaction.

VC gives a rather homogeneous reduction of the soil void ratio all around the probe, with the effect that density, shear strength and stiffness are increased. Treatments are affected by a limited reduction of permeability. In liquefiable soils, this densification can be used to reduce (or eliminate) the liquefaction

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susceptibility, as reported in Figure 2-7. In particular, as for other densification techniques based on vibration, the treatment can cause temporary liquefaction in saturated soil, as it generates an increase in pore water pressure. Finally a denser configuration is reached after the dissipation of the excess pore water pressure.

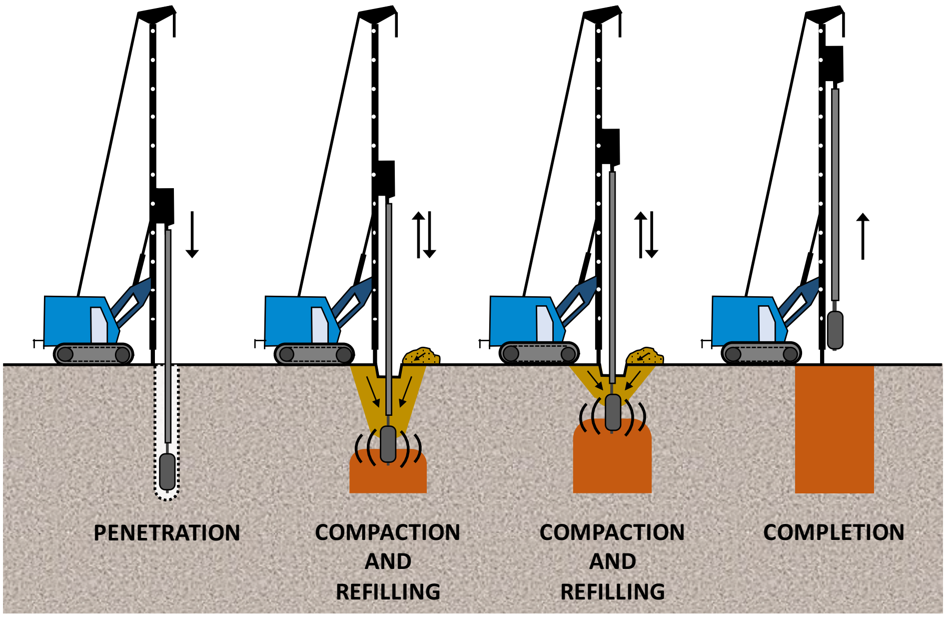


Figure 2-6: Vibro compaction procedure.

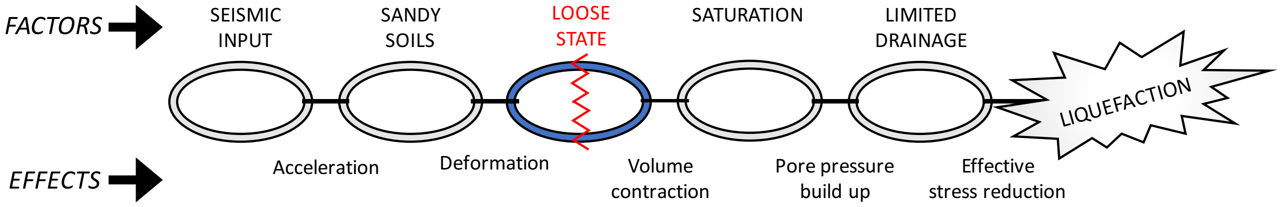


Figure 2-7: Vibro compaction for liquefaction mitigation.

2.3.1.2 Applicability

Vibro compaction is a ground improvement technique suitable for densify clean cohesionless soils. Some quick indications about the suitability of soils for VC were given by Degen (1997) on the basis of Unified Soil Classification System – USCS and reported in Table 2-3. In particular, well-graded gravel and sand are the most suitable soils for VC. In poorly graded granular soils the compaction is only marginal and compaction trials are recommended. Silt content larger than 8÷10% and/or clay content larger than 2% can inhibit the compaction.

Focusing on the grain size distribution of soils, Kirsch & Kirsch (2010) and Degen (1997) reported that the most suitable soils for VC are those that fall into "zone B" in Figure 2-8.Moreover, Brown (1977) proposed a *suitability number* SN, based on the grain size distribution of the soil to be treated (Equation 2-3). In particular, soils with low SN are the most suitable for VC, while for SN > 40 the soil is not suitable for the treatment.

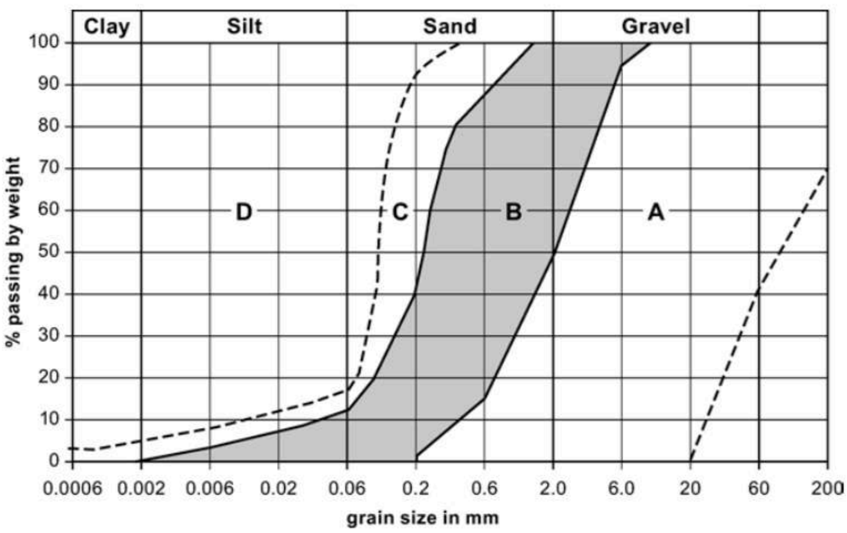
$$SN = 1.7 \cdot \left(\frac{3}{D_{50}^2} + \frac{1}{D_{20}^2} + \frac{1}{D_{10}^2} \right)^{0.5}$$ **Equation 2-3**

where:
SN = suitability number
D_{50}, D_{20} and D_{10} = diameter (mm) of passing particles at 50%, 20% and 10%

The suitability of soils for VC can also be estimated by comparing the results of in-situ tests with literature charts. An example of soil compactability chart based on CPT (cone penetration tests) results is reported in Figure 2-9, classifying the soil in *compactable*, *marginally compactable* and *not compactable*.

Table 2-3: Suitable soils for VC (modified from Kirsch & Kirsch, 2010; Degen, 1997).

SOIL TYPE	USCS	SUITABILITY FOR VC
Well graded gravel	GW	Well suited for vibro compaction, potential penetration difficulties with less powerful machines
Poorly graded gravel	GP	If $D_{60}/D_{10} \leq 2$ compaction only marginal (trial compaction recommended)
Silty or clayey gravel	GM, GC	Compaction not possible if clay content > 2% and silt content > 10%
Well graded sand	SW	Ideally suited
Poorly graded sand	SP	If $D_{60}/D_{10} \leq 2$ compaction only marginal (trial compaction recommended)
Silty sand	SM	Compaction inhibited if silt content > 8%
Clayey sand	SC	Compaction inhibited if clay content > 2%



A = well compactable but problems in the penetration for the high permeability of the soils; *Backfill:* surface materials

B = ideally suited for VC (FC<10%); *Backfill*: surface materials

C = VC possible (extended time); *Backfill*: imported coarser materials

D = VC not suited (Vibro replacement may be used)

Figure 2-8: Soil suitable for VC (modified from Kirsch & Kirsch, 2010; Degen, 1997).

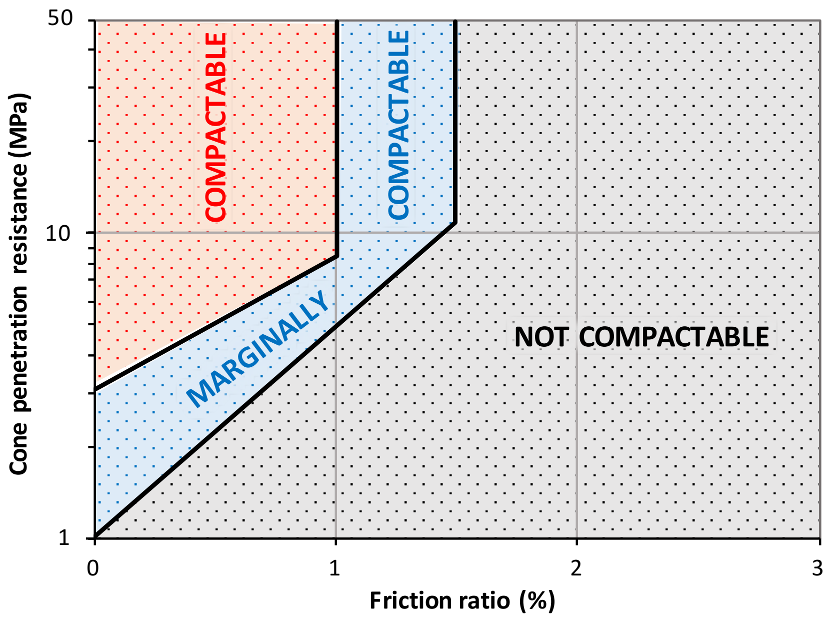


Figure 2-9: Soil compactability based on CPT results (modified from Kirsch & Kirsch, 2010; Massarsch, 1994).

VC is used to improve the soil properties in the design of new structures. The use of VC close to existing structures is restricted by the development of noise and vibration, therefore the acceptability of the disturbances should be verified and monitored.

2.3.1.3 Limitations and drawbacks

Sand and gravel deposits with even small silt and/or clay fraction, are poorly suitable for VC treatments. Cohesive layers in granular deposits can reduce the compaction efficiency. Moreover, very hard layers may need overpassed with pre-boring to allow the penetration of the probe.

For treatments in finer sand, it is suggested to use imported coarser materials as backfill material, while for treatments in coarser materials it can be taken from the surface.

The speed and effectiveness of the densification are related to the permeability of the soil to be treated. For soils with low permeability, the penetration rate of the vibrator will be low. Compaction is increasingly inhibited for decreasing permeability ($k<10^{-5}$ m/s). For increasing amount of gravel and cobbles, thus for high permeability ($k>10^{-2}$ m/s), the loss of water can obstruct the penetration of the vibrator (Kirsch & Kirsch, 2010).

In the case of carbonate sands, since the mineralogy can influence the evaluation of the density from the results of CPT, additional in-situ density tests or laboratory investigation should be carried out (Kirsch & Kirsch, 2010).

The treatment apparatus used for VC requires sufficient headroom.

Vibro compaction is potential sources of noise and vibration. VC can cause settlements close to the treatment location, thus a possible reason for damage to adjacent structures. Vibration measurements should be carried out to evaluate the minimum distance to the closest structures.

The turbid water coming from the penetration process should be purified from the sediments before being discharged.

Since VC can disperse contaminants, if there is a contaminations risk, alternative treatment method should be evaluated.

2.3.1.4 Treatment parameters

The most important treatment parameters for VC are summarised below:

- penetration depth of the probe;
- mean extraction intervals;
- vibration frequency;
- duration of compaction;
- pressure of the water/air jets;
- grid of treatment.

2.3.2 Shallow – vibro compaction / Replacement

Shallow – vibro compaction / Replacement involves the removal of the native soil and replacement with a soil with better characteristics, and the compaction by means of vibratory rollers.

In the case of shallow liquefiable soils, they can be removed and replaced with soils not susceptible to liquefaction: generally granular mixed material. Crushed stone, well graded gravel or soil mixed with cement or other additives are commonly used for replacement in liquefaction remediation. These soils are generally compacted by vibration with vibratory rollers (although can be also used impact rollers for High Energy Impact Compaction, HEIC) (Figure 2-10 and Figure 2-11).

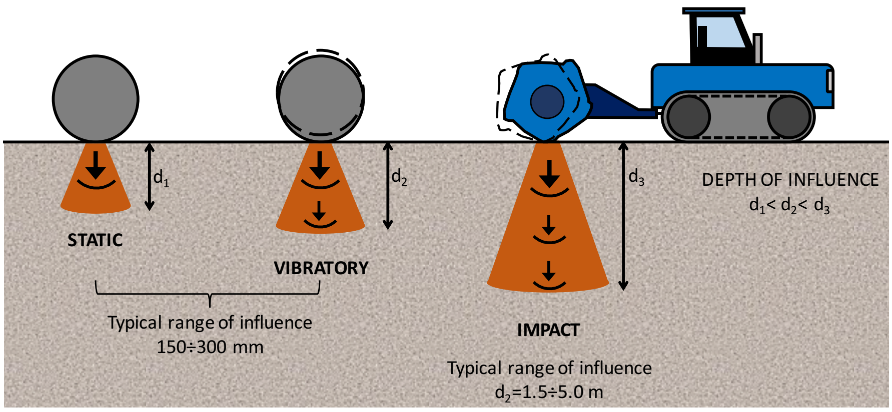


Figure 2-10: Static, vibratory and impact compaction.

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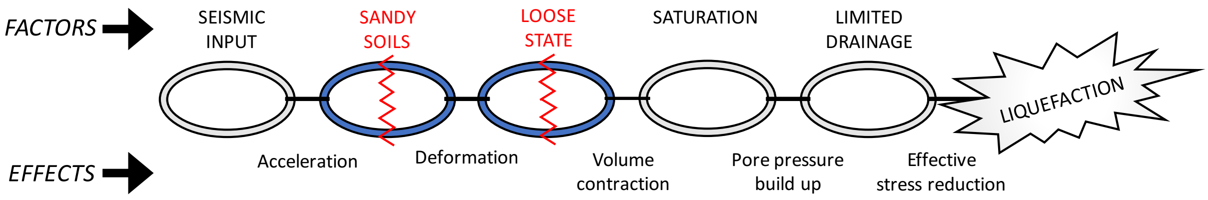

Figure 2-11: Shallow-vibro compaction / Replacement for liquefaction mitigation.

The treatment procedure includes the following steps:

- lowering the groundwater level;
- excavation and removal of the subject soils;
- thin surface spreading of replacement material (usually 30 cm) and rolling (repeated alternatively);
- completion.

Replacement with dense granular fill has been used as ground improvement technique in the rebuild of Christchurch following the 2011 Christchurch earthquake NZGS (2017).

Replacement is most suited for shallow liquefiable layers, but it is also possible to carry out partial replacement of the ✓depth of liquefiable soils according to an acceptable performance criteria. Shallow-vibro compaction / Replacement can be used to treat sands and silts.

The depth of the treatment is generally limited by:

- the feasibility of excavation;
- the dewatering for placing and compacting the soils below the water table;
- the cost of temporary excavation support to protect neighbouring structures from damage (for site near existing structures).

Usually a filter or geogrid is placed below the granular replacement fill to facilitate compaction, to reduce the migration of fine and to provide some protection against lateral stretch.

Shallow - vibro compaction / Replacement involves moderate levels of noise and vibration (possible nuisance or damage to neighbouring properties).

2.4 Blasting compaction

2.4.1 Principle

Blasting compaction, or explosive compaction (EC), adopts the detonation of explosive charges to densify the surrounding soil. The detonation of the explosive causes a shocks wave and an expansion of the high-pressure given by the release of gas. The treatment can be performed in loose granular soils in which the dynamic load of blasting causes a densification process. In particular, in saturated cohesionless soils, the explosion causes an increase of the pore water pressure, with subsequent liquefaction, and destruction of the soil structure. After the dissipation of excess of pore water pressure, a denser configuration is achieved. Moreover, after detonation, a cavity expansion (displacement) and collapse are observed.

The treatment allows an increase of the relative density of about 15÷30% and the compaction is achieved up to depth deeper of the charge of 20÷50%.

The charges can be placed on the ground surface or at some depth in the borehole. For deep treatments, the boreholes are supported by a bentonite slurry or by a plastic casing. Then, the explosive is loaded in the hole, or on the ground surface, and the detonation of explosive is performed (Figure 2-12).

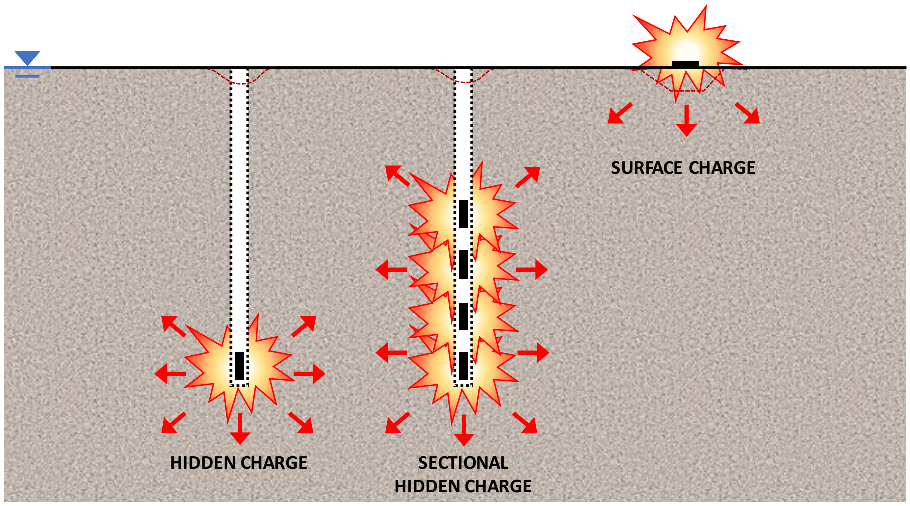

Figure 2-12: Blasting compaction procedure.

The position of the charges is determined by the impact range, from 10 m for low charges of 10÷15 kg TNT, to 20 m for 30 kg. The energy of 1kg of TNT is equal to the energy of 5 tons of tamper with 100 m of free-falling height (Bell & Kirsch, 2013). The explosion of the charges is usually sequential, the delay between the explosions allows a cyclic loading and a minimum of peak acceleration (Gohl et al., 2000).

Usually, to increase the final homogeneity and densification, several passes of explosions are used (generally two): the first pass destroys the bonds between the particles and causes the main settlements, subsequent passes cause additional densification for the cycles of strain. The treatment is usually performed in a triangular grid pattern with 3÷8 m spacing or staggered rectangular grid with of 4÷9 m spacing at multiple depths.

Compaction does not occur immediately after detonation; when explosion takes place only a small ground settlement is produced while the highest densification and settlement occur after several hours. Although the settlement produced after treatment indicates an increase of density, the penetration resistance shows time-dependent behaviour, in some cases immediately after explosion no increase in resistance was observed, in other cases a reduction and sometimes a very small increase, but in most cases after two weeks a high increase is observed (initial value is doubled). This is probably due to the destruction of the bond between the particles or to changes in soil structure and effective stress states as reported by Gohl et al. (2000).

The treatment aims at increasing density, strength and stiffness and reduce compressibility upon cyclic loading of the treated soil. In liquefiable soils, blasting compaction is capable to reduce or eliminate the liquefaction susceptibility for the densification process that occurs, as previously mentioned for loose cohesionless soils, breaking the "loose state" link as reported in Figure 2-13.

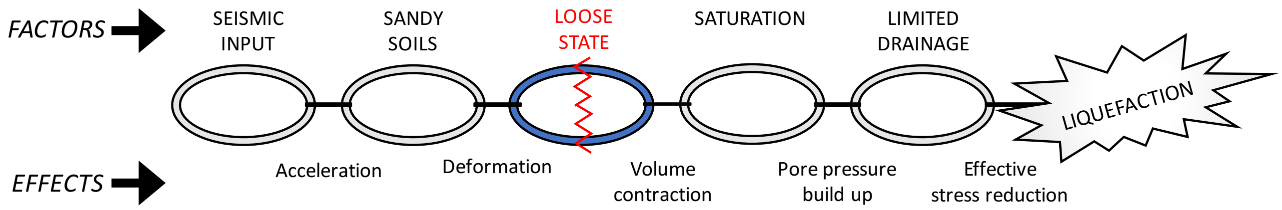

Figure 2-13: Blasting compaction for liquefaction mitigation.

2.4.2 Applicability

The most suitable soils for blasting compaction are cohesionless loose saturated soils (usually sand to silty sand or sand and gravel), while a lower effect is obtained in dense deposits and in dry conditions. The treatment can also be performed in soft fine-grained soils in order to improve drainage. In this case, before the explosion, an additional sand layer is placed on the top. The explosion causes a partial displacement resulting in a cavity, and liquefaction of the upper sand layer that fills the cavity in the fine soil, forming a sand column and a depression on the ground surface.

The design of the treatment is largely based on empirical basis, performing field trials, that can be an obstacle to the use of this ground improvement method for several reasons such as the lack in the method knowledge or the risk factor, for the owner, the contractor and the engineer (as reported by Gohl et al. (2000)).

Blast effectiveness is related to the soil conditions, the type of explosive, the charge length, the layout of the blast hole and the sequence of detonation. Gohl et al. (2000) proposed a formula based on the Hopkinson's number to estimate the radius of influence of an explosion in the hole and reported in Equation 2-4.

$$HN = \frac{(W/\rho)^{0.33}}{r} \quad \text{Equation 2-4}$$

where:
HN = Hopkinson's number
W = charge mass delay (kg)

r = distance from the charge
ρ = mass density of the explosive

Blasting compaction is suitable to treat soils underwater, such as in harbour area, with charges usually placed closely to the bottom.

Blasting compaction is economically advantageous to treat large volumes of cohesionless soils. The depth of treatment is generally higher than other ground improvement techniques and the treatment requires small equipment (e.g. geotechnical drill or wash boring rigs).

The explosion of the charge is not dangerous for human life and health if the treatment is correctly designed and performed by experienced staff.

2.4.3 Limitations and drawbacks

Clay particles can reduce the efficiency of the blasting treatment hampering the mobility of grains with a reduction of the drainage. The environmental impact of the technique is particularly high for the emission of noise and vibration; additionally, gases and fumes emitted during the explosion are deemed responsible for a contamination of the surrounding environment (air and groundwater). Since the treatment induces high rate of vibration and noise, a minimum distance from surrounding structures and building is required, and this occurrence limits the applicability of treatments to sites far from the built environment. In any case, the induced vibration has to be monitored. Explosions taking place within 30÷40 m from structures should require a reduction in the charge (Gohl et al., 2000). Blasting compaction cause settlement of the surrounding ground, so it is important to continuously monitor the settlement of the area.

2.4.4 Treatment parameters

The most important treatment parameters are reported below:

- charge in each hole;
- depth of charge;
- scattering pattern of charges (in height);
- distance between the holes;
- phasing and number of blast stages;
- sequence of explosions.

Some useful information for the design of blasting explosion can be found in Narsilio et al. (2002) and are summarised in Table 2-4

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Table 2-4: Blasting compaction design (from Narsilio et al., 2002).

Charge size	Depth	Horizontal spacing	Detonation interval	Detonation sequence	Number of events	Related observations	References
$M=0.10\times z_w^{2.46}$	$1.8\cdot M^{1/3}$	$s=4\cdot M^{1/3}$					Ivanov (1967)
1–12 kg 8–850 gr/m^3 10–30 gr/m^3	$>\frac{1}{4}$ depth to bottom of layer to be treated (usually $\frac{1}{2}$–$\frac{3}{4}$)	5–15 m	Hours to days		1–5 (usually 2–3)	$\Delta S=0.02H$ to $0.10H$	Mitchell (1981)
3.6 kg (30% special gelatin dynamite)	3, 6, and 11 m	6 m	4 h			Porosity changed from 47 to 43% at Karnafuli Dam (D_{10}=0.18 mm, C_u=2)	Terzaghi et al. (1996)
$\mathrm{HN}=M^{1/3}/R$	R or $\frac{2}{3}$ depth to bottom of layer to be treated	R	Preliminary test needed	From the edges, inwards (Polish experience)		$\Delta S=H[2.73+0.9\ln(\mathrm{HN})]$	Ivanov (1967) van Impe (1989) Narin van Court and Mitchell (1998) Narin van Court (1997, 2003)
$\mathrm{NM}=\dfrac{(M/L_c)^{1/2}}{R}$	$H_B=1.48\cdot Q^{1/2}$ $H_1=2.63\cdot C^{1/3}$	Square or equilateral triangles				May not be valid for different patterns and site conditions	Dembicki et al. (1992) Imiolek (1992) Narin van Court and Mitchell (1998) Narin van Court (2003)
$E_1=\Sigma\left(\dfrac{M_i}{R_{vi}^2}\right)$ with $350<E_1<3{,}500$	Layers >7–8 m should be divided in sublayers of 5–6 m thick	Square grid 4.5–11 m (preferably, 4.5–6 m) $s=1.4\cdot s_{\mathrm{final}}$ $s=2\cdot s_{\mathrm{final}}$ (in the first two events if three events are needed)	Several min (controlled by u-dissipation)	Outward, from center to outside	1–3	Predicted final q_t in CPT: $q_{1,f}=0.195\cdot q_0^{0.478}\cdot E_1^{0.330}\cdot \sigma_v'^{0.175}$ (R^2=0.66) $q_{1,f}=0.404\cdot q_{1,0}^{0.525}\cdot E_1^{0.327}$ (R^2=0.64)	Narin van Court (2003)

Note: M explosive charge mass (kg), z_w ground water table depth, ΔS surface settlement (m), H thickness of the layer treated by blast densification, recommended Hopkinson's number values [as defined by Ivanov (1967)] HN: 0.15 (van Impe 1989), 0.50 (Narin van Court 2003), 0.2–0.5 (Ivanov 1967), 0.5–1.2 for nonconcentrated charges (Ivanov 1967), or chose HN such that $M=10$ kg TNT (Ivanov 1983), R effective radius in plan (m) $=1/2\cdot s$, s grid spacing, recommended normalized explosive charge mass values [as defined by Dembicki et al. (1992)] NM: 0.3 – 0.6 (Dembicki et al. 1992), 0.4 – 0.7 (Narin van Court 2003), H_b minimum distance from ground surface to top of charge (m), Q charge loading density (in kg/m), C concentrated charge (kg). Recommended energy input attenuation E_1 values at the center of the grid using M_i=4–7 kg per sublayer, depending on spacing: $350<E_1<1{,}000$ for very loose soils ($q_t<5$ MPa), $1{,}500<E_1<3{,}500$ for loose to medium soils (5 MPa$<q_t<15$ MPa) or $750<E_1<1{,}500$ for two events, with no mayor dependency on σ_v', R_{vi} minimum distance from charge to a point in the soil mass (in m), $q_{1,f}$ CPT final normalized tip resistance, q_t CPT tip resistance.

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2.5 Compaction grouting

2.5.1 Principle

In the compaction grouting technique, a cavity is previously created in the ground (usually loose cohesionless soils), then a very thick grout (known as *low mobility grout*, LMG) is injected into to cause expansion of the cavity. The thickness of the grout serves to limit seepage in the surrounding soil. This process causes a growth of the bulb of the injected grout that displaces and compacts the surrounding soil. When treatments are performed at different heights, columns are formed that represent a reinforcement for the treated area. In 2010, the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers published the 'Compaction Grouting Consensus Guide' (ASCE/G-I 53-10, 2010), defining the compaction grouting as follows:

"*Compaction Grouting is a ground improvement technique that improves the strength and/or stiffness of the ground by slow and controlled injection of a low-mobility grout. The soil is displaced and compacted as the grout mass expands. Provided that the injection process progresses in a controlled fashion, the grout material remains in a growing mass within the ground and does not permeate or fracture the soil. This behavior enables consistent densification around the expanding grout mass, resulting in stiff inclusions of grout surrounded by soil of increased density. The process can be applied equally well above and below the water table. It is usually applied to loose fills and loose native soils that have sufficient drainage to prevent buildup of excess pore pressure.*"

The most used grouts are soil-cement mixtures (mortars) with low water/cement ratio and high viscosity. The grout is usually injected from open-end pipes into pre-drilled hole. The grout displaces the surrounding soil that densifies and remains as a homogeneous mass without permeating or fracturing the soil. In particular, the grout is pumped into the pipe in several stages, creating a column of connected grout bulbs, following different types of procedure:

- *Stage-up* procedure if the process is carried out upward from the bottom;
- *Stage-down* procedure if the process is carried out downward from the top.

In the *stage-up* procedure (summarised in Figure 2-14) the pipe is installed to the desired depth and then the injection is performed during pipe withdrawal at prescribed intervals. This process is the most common because easier, faster and cheaper. In the *stage-down* procedure (summarised in Figure 2-15) the pipe is installed on the top of the treatment location and the injection is performed at that depth. After the setting time, the drilling is performed downward through the grout injected before, and a new injection is carried out under the previous position. This process is repeated at prescribed intervals.

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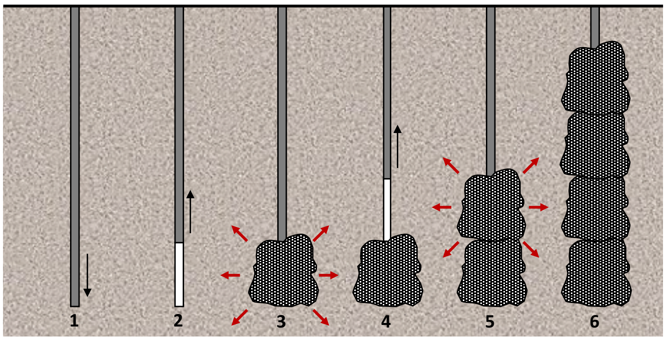
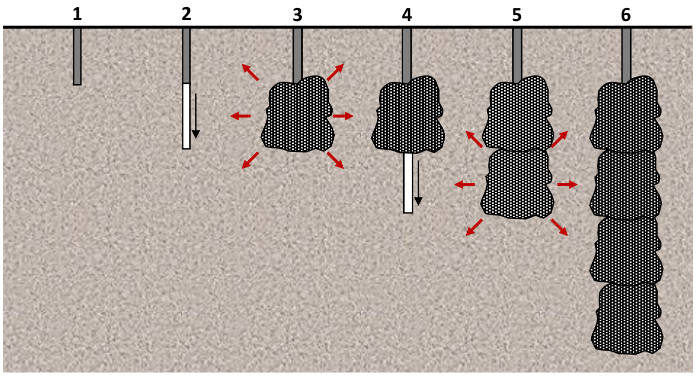


STAGE-UP PROCEDURE (ASCE/G-I 53-10, 2010)

1: Install casing
2: Retract casing to top of deepest stage
3: Grout first stage
4: Raise casing to the top of next stage
5: Grout next stage
6: Repeat steps 4 and 5 to top of improvement zone

Figure 2-14: Compaction grouting procedure: Stage-up method.



STAGE-DOWN PROCEDURE (ASCE/G-I 53-10, 2010)

1: Cement casing into oversized hole that extends to the top of the first grout stage
2: Drill trough casing to extend hole to bottom of intended stage
3: Grout first stage and allow grout to set (usually overnight)
4: Drill through first stage and extend hole to bottom of second stage
5: Grout next stage and allow grout to set
6: Repeat steps 4 and 5 until hole bottom is reached (usually indicated by low grout take or high pressure)

Figure 2-15: Compaction grouting procedure: Stage-down method.

The soil should be in close contact with the pipe to prevent the grout from rising upwards through the soil-pipe annulus and to provide resistance against the pipe lifting during injection. Moreover, before starting the injections, generally, water is introduced into the pipe and then filled with mortar.

According to EN 12715 (2000), the injection process for grouting methods is governed by:

- the grout volume per pass;
- the injection pressure;
- the flow or placement rate;
- the grout rheology.

The injection pressure increases with the soil density and the depth. In fact, the pressure increases as the injection continue, due to the higher density of the treated soils.

During the injection, the grout is pumped at high pressure until one or more refusal criteria are reached. Common examples of refusal criteria are:

- target volume per stage;
- maximum pressure at a given injection rate;
- undesired ground movement or maximum ground heave.

According to EN 12715 (2000), the grout injection is usually preceded by a drilling phase, that can be performed in different ways:

- rotational drilling;
- percussion drilling (with hammer);
- case percussion drilling;
- grab, chisel and bailer borings;
- driving lances;
- vibrating of casing or drill pipes.

Different shapes of the injected grout can be obtained depending on the soil conditions, as reported in Figure 2-16: *sphere*, *irregular shape*, *quasiconical shape* and *columnar shape*.

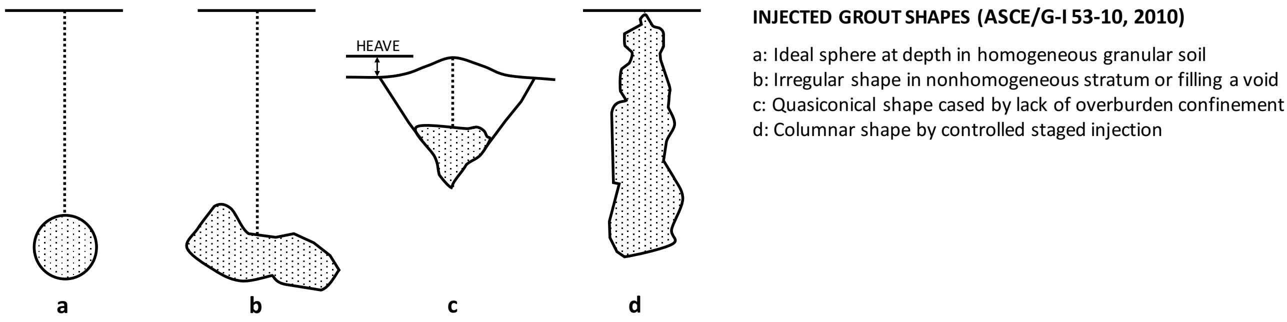


Figure 2-16: Injected grout shapes.

With regard to the grout, the Grouting Committee (ASCE/G-I 53-10, 2010) defined the compaction grouting grout as "*a mixture of silty sand, cement and water to form a mortarlike material with a slump less than 2 in.*". According to the Grouting Committee, the grout must satisfy the following requirements:

- sufficiently pumpable;
- must remain as a growing mass in the ground;
- any bleed water must be able to dissipate into the ground.

In fact, low mobility grout has to be designed to be both pumpable and immobile (Hussin, 2013; ASCE/G-I 53-10, 2010). The amount of water and the aggregate gradation affect these properties. In particular, the amount of water added is the minimum required to guarantee a pumpable grout. For the aggregates, Hussin (2013) reported that a uniform gradation is commonly used, combining gravel to silt material (with 100% finer than 5÷20 mm and 0% finer than 0.001÷0.03 mm). Some suggestions for the aggregate gradation are reported in Figure 2-17.

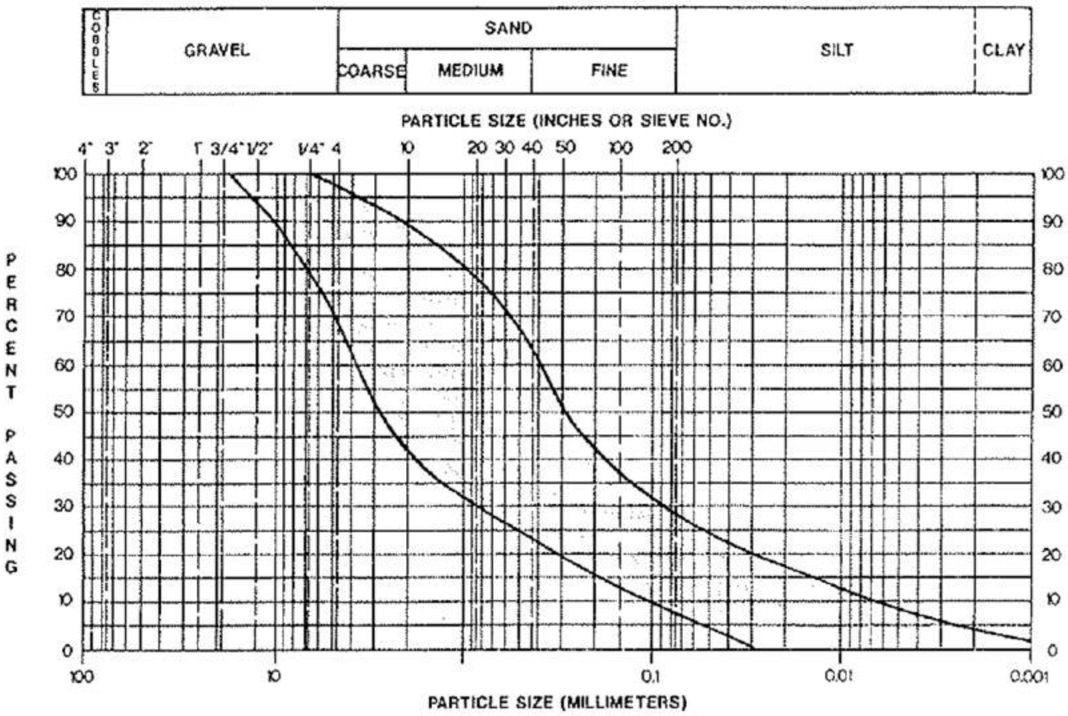


Figure 2-17: Preferred aggregate gradation (from ASCE/G-I 53-10, 2010; Warner et al., 1992).

The coarser particles provide to the grout internal friction and permeability, resulting in an immobile grout when leaves the point of injection. High internal friction allows to preserve the "spheroidal shape" of the bulb and to avoid fracturing and lensing (ineffective treatment). Moreover, the silt size particles provide mobility. The cement is usually included (providing the finer particles to make the grout pumpable), but not required. Moreover, the mineralogy of the aggregate can affect the behaviour of the grout.

The addition of plastic clay, such as bentonite, or concrete pumping additives, are to be avoided because they can cause a fluid-like behaviour in the soil.

Since the purpose of the treatment is the soil densification, the strength of the grout is not important. The design of the treatment is related to the subsoil conditions, type of soil, density and stress state. Typical injection rate for low permeability soils (or low confinement) are 4.2÷28.3 litres per minute and for high permeability (or dry soils, or soil at depths) 113÷340 litres per minute (Hussin, 2013). The Grouting Committee (ASCE/G-I 53-10, 2010) reported typical values on the order of 30÷60 litres per minute. The injection rate (slow) must be balanced by permeability of soils (high), in order to dissipate the excess of pore water pressure caused by injection providing the compaction; otherwise the hydraulic fracturing can occur (treatment with high injection rate and/or low permeability soils). Schaefer et al. (2017b) reported that the grout rate is usually in the range of 1.5÷2.0 cubic feet per minute (42.5÷56.6 l/min). In sensible areas, such as close to retaining walls, it is suggested to use very low rate.

The amount of grout injected depends on the type of soil, the initial and the required density. Hussin (2013) reported typical values of the injected volume of 8%÷12% of the soil volume to compact and Schaefer et al. (2017b) reported values of 3%÷12%.

The treatment is usually performed in rectangular or triangular injection grid, by means of a method defined as "*spilled spacing*" (Figure 2-18) which uses primary and secondary positions, maximizing the confinement (primary injections provide confinement to the secondary ones).

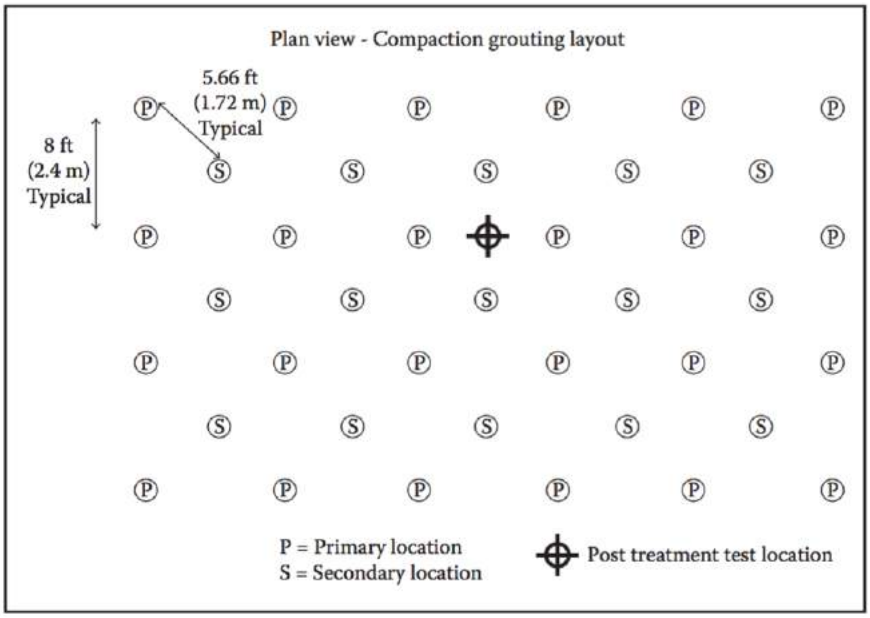


Figure 2-18: Example of compaction grouting layout plan (from Hussin, 2013).

For site improvement Schaefer et al. (2017b) reported that the grout pipes are typically installed at intervals of 6.6 to 16 feet (2÷4.9 m), 3.3 to 10 feet (1÷3 m) for remedial work on existing structures and 8 to 15 feet (2.4÷4.6 m) for tunnelling projects. Hussin (2013) reported space intervals of 0.9÷4.6 m, with typical interval values of 1.5÷2.1 m.

The treated soil is characterised by increased density, increased strength, reduced deformability and permeability. In particular, the densification is characterised by a typical volume reduction of 15%÷20% (Hussin, 2013). The most common applications related to compaction grouting are:

- correction of differential settlements and settlement control (e.g. prevention of tunnel-induced settlement, as reported in Figure 2-19);
- soil densification for ground improvement;
- underpinning and excavation support;
- sealing purpose;
- mitigation of liquefaction potential.

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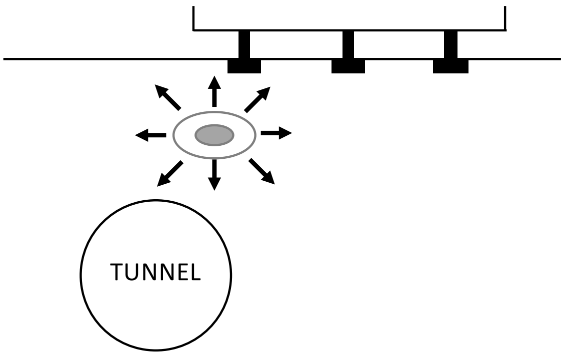

Figure 2-19: Prevention of tunnel-induced settlements.

With regard to the latter one, compaction grouting can be used to mitigate the liquefaction potential as it densifies the soil, moreover, the pseudo-columnar elements create a reinforcement that limits ground deformation (Figure 2-20). Boulanger & Hayden (1995) reported the benefits of compaction grouting in clayey silt and silty sand (increased SPT and CPT resistance) and the time effects on the penetration resistance of the treated soils.

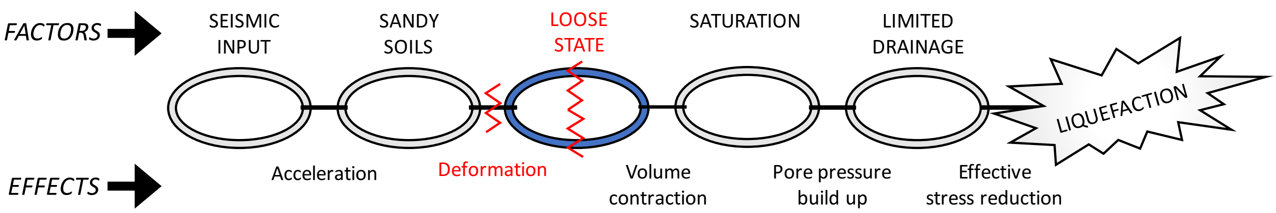

Figure 2-20: Compaction grouting for liquefaction mitigation.

2.5.2 Applicability

The compaction process induces a reduction of the soil porosity requesting the exit of the water. Consequently, compaction grouting is most suitable to treat soils with high permeability (free-draining) and/or with low degree of saturation. The best suited soils are loose cohesionless soils (as gravel, sand and coarser silt), as reported in Figure 2-21. Sands with less than 10% of silt and without clay are very suited to compaction grouting both above and below the water table (Hussin, 2013). In dense soils the process can cause dilation, inducing an increase in volume instead of a reduction. In fine-grained soil the treatment is not suited for the slow rate of pore water pressure dissipation. Slow injection rates are required to treat this type of soil.

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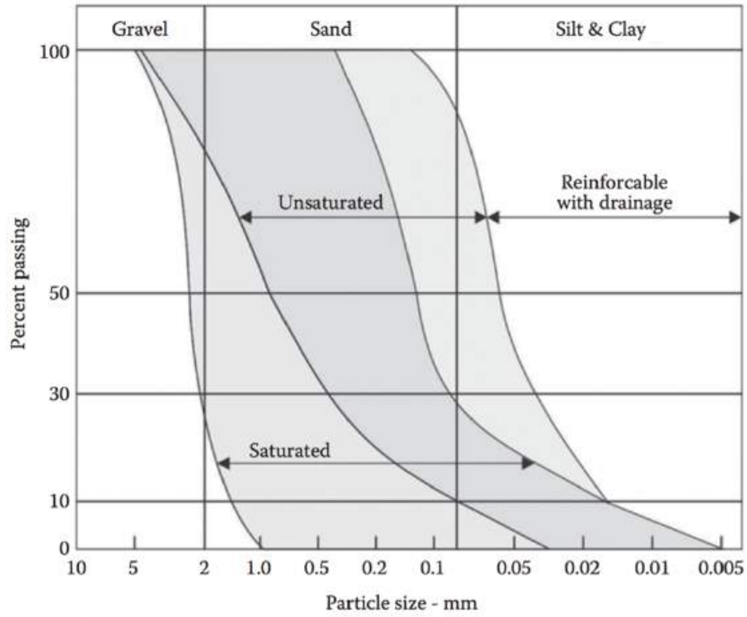

Figure 2-21: Suitable soils for compaction grouting (from Hussin, 2013).

The European Standard (EN 12715, 2000) provides the following indications for the soils suitable for compaction grouting with regard to the permeability and type of grout (Table 2-5):

Table 2-5: Indicative grout type for compaction grouting.

SOIL TYPE	PERMEABILITY TO WATER	GROUT
SAND	$5 \cdot 10^{-5} \leq k \leq 5 \cdot 10^{-3}$ m/s	Cement based suspensions Mortar

As for any other compaction method, the soil moisture content affects the compaction behaviour, in particular, dry soils require more effort to densify than wet soils where water lubricating particles facilitates compaction.

Due to the complexity of the process, the design is based on empirical assessments (some indications based on the literature are reported by Santosuosso and Scarpato (2018). Suggestions for analytical and numerical modelling are provided by the Grouting Committee (ASCE/G-I 53-10, 2010) and Santosuosso and Scarpato (2018) assuming the following mechanisms:

- the injection is idealised as expansion of the grout column in the soil;
- the injection is idealised as a ground movement in all directions (spherical cavity).

The treatment apparatus is composed by a mixer (ensuring complete and uniform mixing of the material), a pump (ensuring injection with appropriate rate and pressure), grout pipes (steel pipes with high strength), hoses and pressure gauges. Sometimes it is required to use ready-mix material delivered in mixer trucks to the treatment position. The treatment apparatus is generally constituted by a small-scale, manoeuvrable and vibration-free equipment, allowing the treatment close to (and below) existing structures. In this case relevelling operations could be necessary (if settlements or heaves of the ground surface occur).

For grouting treatment, and thus for compaction grouting, the European Standard (EN 12715, 2000) reported the following environmental impact risks on site:

- ground movements;
- changes of groundwater level;
- spreading of grout;
- pollution of groundwater;
- dust distribution.

Moreover, chemical reactions between groundwater and hardened grout have to be considered.

2.5.3 Limitations and drawbacks

The treatment carried out in granular soils that contain clay can have a lower efficiency as the clay fraction (1%÷2%) can reduce the permeability and reduce the injection rate.

In surface soils compaction grouting treatments can be ineffective due to a low over-burden pressure (lower than the lateral pressure of the soils), causing a heave of the ground surface instead of the soil densification. For this reason, it is very difficult to treat shallow soils at depths lower than 3 m (Hussin, 2013). In particular, NZGS (2017) reported that shallow treatments in interbedded sand and silty soils tend to dilate soils (due to a low confining pressure), increasing the liquefaction potential.

Sometimes heaves can occur for deep treatments when the surrounding soil is already compacted and resists to further displacements. The heave of the ground surface is a limiting factor because it can damage existing structures. The heave is also an indicator of the occurrence of fracturing processes. For this reason, it is used as a refusal criterion by comparing the real heave with an allowable threshold value previously fixed.

The treatment performed in fine-grained soils adopting high pumping pressure can cause increased pore water pressure and damage to existing structures.

2.5.4 Treatment parameters

The relevant parameters to be fixed for treatment are:

- grout composition;
- grout hole spacing (grid of treatment);
- maximum depth of treatment;
- grouting stage length;
- injection pipe diameter;

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- injection rate;
- limiting injection pressure;
- injected volume.

2.6 Low pressure grouting

2.6.1 Principle

Low pressure grouting (or permeation grouting) consists of low pressure injections of grouts in the soil without altering the original structure, filling most of the porosity (70%÷80%). During the treatment, the injection pressure is kept below the value that causes the fracture of the soil.

According to the European Standard (EN 12715, 2000), the grouts are classified in:

- *Suspensions* (particulate or colloidal), defined as: "a *mixture of liquid and solid materials. Behaves as a Bingham fluid during flow, possessing both viscosity and cohesion (yield strength). Particulate suspensions contain particles larger than clay size, while colloidal suspension contain particles of clay size*".
- *Solutions* (true or colloidal), defined as: *"a liquid formed by completely dissolving a chemical in water to give a uniform fluid without solid particles. Solutions are Newtonian liquids with neither rigidity nor particles and harden in a predetermined period of time, called "setting time". They can be true or colloidal solutions. In the case of colloidal solutions, large molecules are contained in the liquid"*.
- *Mortars*, defined as: "*a highly particulate grout containing sand*".

The injected grouts are usually composed by hydraulic binders or cement (often micro-cements) or chemical products such as silicates (often nano-silicates), acrylic or epoxy resins and polyurethanes. The chemical grouts are also classified as *hard* or *soft gel* that differ mainly in final strength. To select a suitable type of grout, the following proprieties should be considered:

- composition;
- particle size (if applicable);
- stability (no sedimentation or separation of the components);
- rheological properties (viscosity, cohesion and friction angle with time);
- gellification/setting time (time to obtain an increasing of viscosity and reduced workability);
- strength and durability;
- toxicity.

Moreover, some types of silicate solutions are not stable with time and the behaviour can be affected by the temperature, these aspects have to be assessed.

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According to EN 12715 (2000), the main properties that characterise the grouts before and after setting are reported in the following table (Table 2-6), for solutions, suspensions and mortars.

Table 2-6: Parameters characterising grout properties (modified from EN 12715, 2000).

Time	SOLUTIONS	SUSPENSIONS	MORTARS
Before setting	Setting time, density, pH, surface tension, pot life, film time, gel time, viscosity, cohesion, thixotropy	Setting time, density, pH, grain size distribution, viscosity, cohesion, yield, thixotropy, stability, water retention capacity	Setting time, density, pH, grain size distribution, viscosity, workability, water retention capacity
After setting	Hardening after setting, final strength, pH, deformability, durability, shrinkage, expansion, shear strength, syneresis (silicate based solutions)	Hardening time, final strength, deformability, durability, shrinkage, expansion, density, shear strength	Hardening time, final strength, deformability, durability, shrinkage, expansion

The treatment procedure for low pressure grouting is summarised in Figure 2-22. In particular, the injection is usually preceded by a drilling phase, that can be performed in different ways (EN 12715, 2000):

- rotational drilling;
- percussion drilling (with hammer);
- case percussion drilling;
- grab, chisel and bailer borings;
- driving lances;
- vibrating of casing or drill pipes.

Moreover, for unstable ground conditions, can be necessary the use of drilling muds (or grouts or foams), temporary casing, direct injection of sleeve pipes or progressive stabilisation as the borehole advances.

After the initial drilling phase (usually roto-percussive systems), the injection can be performed by means of two ways: by mixing the components before the treatment and injecting the mixture into the soil (*one shot*), or by means of double injections (*two shot*) where the components are injected separately in the soil. The most common is the one-shot injection.

The gellification/setting time and the final properties of the treated soil depend on the proportion of the components, the temperature, the mixing speed of the grout.

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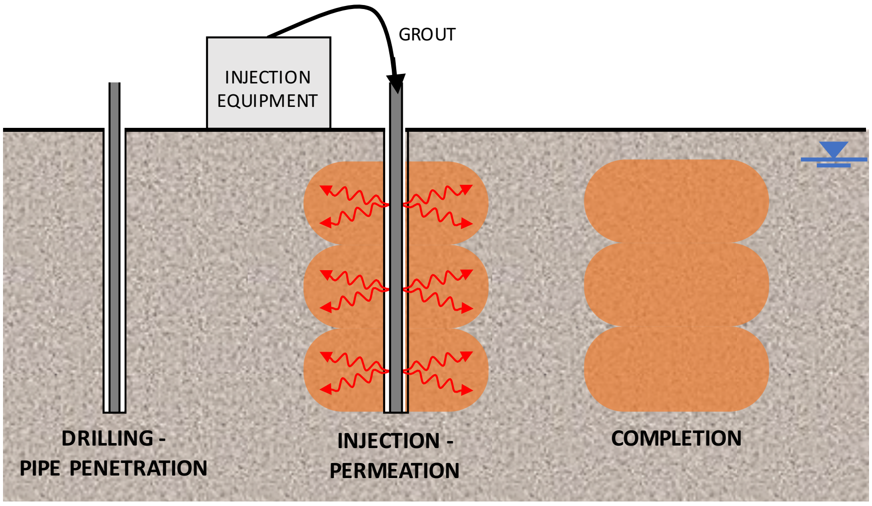

Figure 2-22: Low permeation grouting procedure.

Different types of grout pipes and packers can be used, some examples are provided by Stadler & Krenn (2013):

- manchette pipes, with several ports at fixed intervals (the most used);
- open-ended or perforated lances, used for simple treatments and for limited depth;
- single port outlet (bundles of several individual supply lines);
- multiple packer sleeve pipes (combination of manchette pipes);
- single or double packers or self-inflating rubber packers.

According to EN 12715 (2000), the injection process is governed by:

- the grout volume per pass;
- the injection pressure;
- the flow or placement rate;
- the grout rheology.

The injection can be performed in multiple stages, over many holes, with a sequence of injection passes. Thus, as reported by the European Standard, the design shall specify:

- the type of treatment progression (inward or outward, top-down or bottom-up, etc);
- the grouting phases;
- the number of passes per stage;
- the type of grout injected for each pass.

The treated soil is characterised by reduced deformability, increased cohesion, higher shear and compressive strength, reduced permeability and reduced interconnected porosity. Thus, the most common applications of low-pressure grouting are (Schaefer et al., 2017b; Stadler & Krenn, 2013):

- waterproofing and seepage control;
- slope stabilization;
- reinforcement of soil;
- settlement control, underpinning and excavation support;
- soft ground tunnelling to increase cohesion;
- mitigation of liquefaction potential;
- rehabilitation of structures;
- waste immobilisation.

Sometimes the term *structural permeation grouting* is applied when the aim of the treatment is to improve the mechanical properties of the soil, on the contrary, *waterproofing grouting* is used when the objective is stopping the flow of water.

Low pressure grouting can be used to reduce the liquefaction susceptibility, in particular, one of the most suitable injected mixtures is nano-silicate. Nano-silicate is a colloidal suspension composed by a monomer containing nano-silica, a sodium chloride solution, used as activator, and water. The suspension penetrates the soil filling the voids and gels resulting in a bond with the soil particles. This process reduces the volume contraction of the soil and thus the generation of excess pore water pressure, preventing liquefaction (Figure 2-23). Furthermore, the strength and stiffness of the treated material will increase.

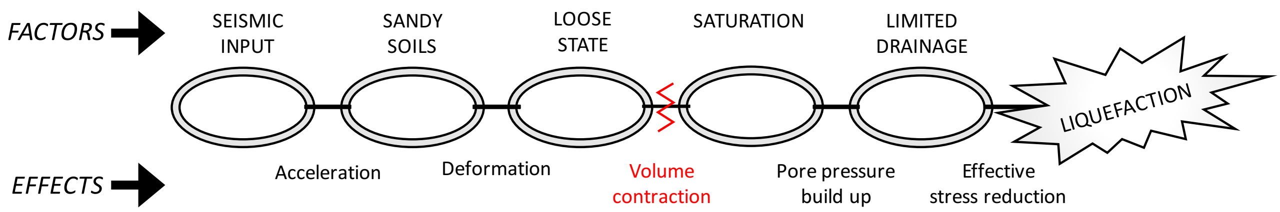

Figure 2-23: Low permeation grouting for liquefaction mitigation.

In cement-based grout, typical water-cement ratios adopted are 0.5÷6. Chemical additives can be added to increase the permeation, to prevent flocculation and to increase the setting time.

In silicate grouts the gel is usually composed by 50÷70% of water and 30÷45% of sodium silicate plus hardener (Stadler & Krenn, 2013).

2.6.2 Applicability

The suitability of soil for low-pressure grouting treatments is primarily affected by the dimension of the soil pores which allow the penetration of the mixtures and by the particles size of the injected mixtures. The suitable soils range between silt to gravel, as shown in Figure 2-24 and Figure 2-25. In particular, for sandy-silty soils the most suitable mixtures are nano-silicates, nano-cements and resins.

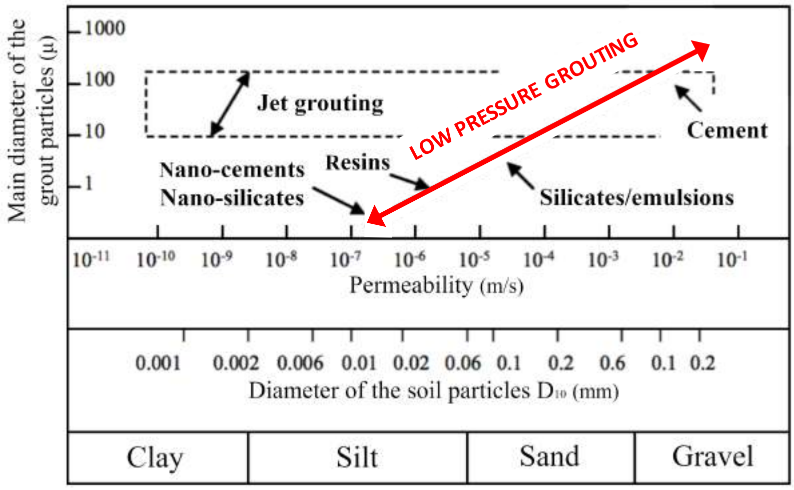

Figure 2-24: Soils suitable for low pressure grouting (modified from Flora & Lirer, 2011).

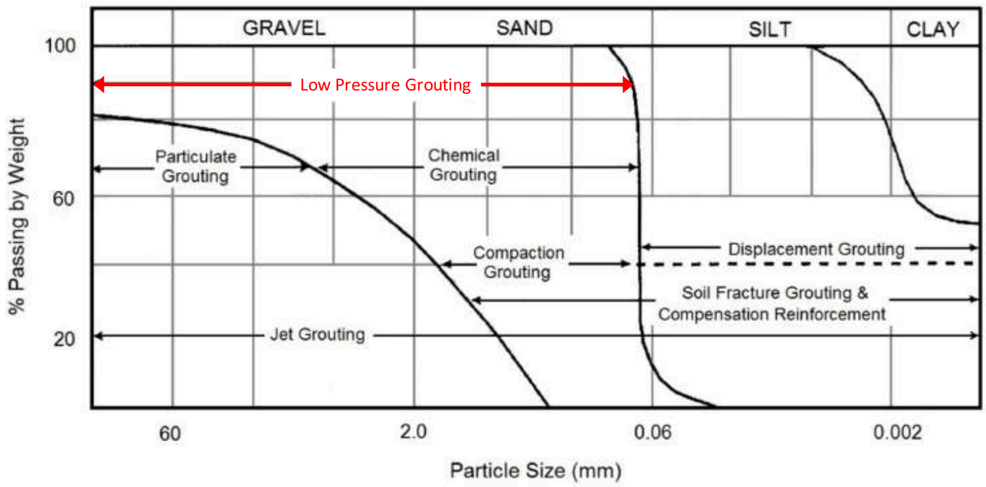

Figure 2-25: Particle size distribution of soils suitable for low pressure grouting (modified from Schaefer et al., 2017b).

Rigorous analysis of the injection process is very difficult as it includes seepage and chemical processes, taking into account the dilution, the dispersion and the sedimentation of the particles in the water. For examples the model proposed by Bouchelaghem & Vulliet (2001) and Bouchelaghem et al. (2001) simulates the seepage processes of a multi-phase fluid media (injection of cement mixtures) in a saturated porous medium, taking into account the mass transport, the dilution and the seepage. Due to the complexity of this type of analysis, the most common approach to assess the suitability of the treatment is empirical, based on the particle size distribution or permeability, as reported above.

Schaefer et al. (2017b) provided some indications for the soils suitable for treatment using cement and bentonite grouts (Table 2-7).

Table 2-7: Soils suitable for treatment using cement and bentonite grouts.

CEMENT/BENTONITE GROUTS	Soils suitable to treatment
PORTLAND CEMENT Type I - II	Soils coarser than 0.024 inches (0.61 mm)
PORTLAND CEMENT Type III	Soils coarser than 0.016 inches (0.41 mm)
BENTONITE	Soils coarser than 0.01 inches (0.25 mm)
MICROFINE CEMENT	Soils coarser than 0.002 inches (0.05 mm)

For undisturbed sandy soils, Lees & Chuaqui (2003) proposed to approximate the injectability of soils to the hydraulic conductivity estimated by Hazen's equation, and the European Standard (EN 12715, 2000) reported the following indications for the injectability of soils (Table 2-8):

Table 2-8: Indicative grout type for low permeation grouting in different granular soils.

SOIL TYPE	PERMEABILITY TO WATER	GROUT
GRAVEL, COARSE SAND AND SANDY GRAVEL	$k > 5 \cdot 10^{-3}$ m/s	Pure cement suspensions Cement based suspensions
SAND	$5 \cdot 10^{-5} \leq k \leq 5 \cdot 10^{-3}$ m/s	Microfine suspensions Solutions
MEDIUM TO FINE SAND	$5 \cdot 10^{-6} \leq k \leq 5 \cdot 10^{-4}$ m/s	Microfine suspensions Solutions Special chemicals

For suspensions, the injectability is related to the ratio of the particles size and the dimension of the pores of the soil to be treated. To evaluate the injectability of a suspension Mitchell (1981) proposed to calculate two parameters N and N_c using Equation 2-5. In particular, the injection is possible if N>24 and N_c>11 and impossible if N<11 and N_c<6.

$$N = \frac{D_{15}}{d_{85}}; \qquad N_c = \frac{D_{10}}{d_{95}} \qquad \text{Equation 2-5}$$

where:
N, N_c = groutability ratios
$D_{15\ (or\ 10)}$ = diameter of passing particles at 15% (or 10%) by weight, for the soil to be treated
$d_{85\ (or\ 95)}$ = diameter of passing particles at 85% (or 95%) by weight, for the suspension

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In low permeability grouting, one of the most important parameter is the soil permeability to mixtures (k_m), related to the permeability to water and to the specific weight and viscosity of the water and the mixtures, as reported in Equation 2-6.

$$k_m = k_0 \cdot \frac{\gamma_m}{\mu_m} = k_w \cdot \frac{\gamma_w}{\mu_w} \cdot \frac{\gamma_m}{\mu_m} \qquad \text{Equation 2-6}$$

where:
k_m = permeability to mixtures
γ_m ; γ_w = specific weight of the mixture or water
μ_m ; μ_w = dynamic viscosity of the mixture or water
k_0 ; k_w = absolute permeability or permeability to water

Typical values of the dynamic viscosity, μ_m, are reported in Table 2-9 (Lirer et al., 2004).

Table 2-9: Typical values of µm for some mixtures (modified from Lirer et al., 2004).

Mixture	Viscosity μ_m (mPa·s)
Silicate	2÷100
Colloidal silica	5÷50
Cementitious	5÷200
Aminoplast	6÷30
Acrylamide	2÷8
Lignins	2÷8
Polyurethane	20÷150

Thus, the following aspects have to be considered, in addition to the size of the porosity and the grout particles:

- the permeability and the penetrability of the grout;
- the chemistry of the groundwater, mix water and the ground;
- ground and grout temperature;
- risk/effect of grout drying;
- environmental impact during mixing, processing and placement;
- pollution potential.

The equipment for the treatment should be capable of supplying, proportioning, mixing, and pumping the grout. The apparatus should be equipped with piping and accessories. Furthermore, the apparatus requires suitable tanks to store the materials. In particular, low permeation grouting plant consists of the following equipment, as reported by European Standard (EN 12715, 2000):

- drilling and driving equipment;
- mixing and proportioning equipment;
- pumping equipment;
- injection piping;

- packers;
- monitoring and test equipment.

The treatment apparatus needs relatively small space and the treatment procedure produces very small vibration compared to other densification allowing the treatment close to existing structures. Particular attention must be given in the case of collapsible soil to be treated.

Some indications about the cost of the treatment with different types of grout are reported in Table 2-10 as suggested by Stadler & Krenn (2013).

Table 2-10: Relative cost of treatment (modified from Stadler and Krenn, 2013).

TYPE OF GROUT	RELATIVE COST (per kg, provided but not injected)
Ordinary Portland Cement	1
Binder	1÷3
Microfine binder - Blaine value 8000 cm^2/g Microfine binder - Blaine value > 12000 cm^2/g	5 10
Silicate gel (hardener: aluminate/acetate)	215
Resin products (e.g. polyurethane, specialised epoxies)	> 30÷150

The most important advantage of the silicate compared to the cement base mixtures are the reduced setting (or gelling) and hardening time. Furthermore, the silicates are non-toxic and environmental friendly.

2.6.3 Limitations and drawbacks

The major limitation of low-pressure grouting is related to the uncertainty of the result obtained by the treatment, in terms of extension and mechanical properties obtained.

Not injectable lenses in the soil to be treated may reduce the effectiveness of the treatment.

Several types of grout are not stable with time or the rheological and mechanical properties change with time. There are some concerns about the permanence in time of some types of chemical grouts, as reported by Schaefer et al. (2017b) for silicate injections. Moreover, the risk of excessive dilution of the grout in groundwater which can prolong the setting time and inhibit chemical reactions, have to be evaluated.

For suspension, the tendency to flocculate, to settle and to bleeding have to be taken into account.

The European Standard (EN 12715, 2000) reported that organic silicate gels *"may lead to the proliferation of bacteria in the ground"*.

The pore water pressure and stress changes caused by the treatment should be considered. The environmental impact must be considered, in particular with regard to the toxicity of the grout (and grout components) and its effect on the ground and on the groundwater.

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Moreover, environmental impact risks on site include:

- ground movements;
- changes of groundwater level;
- spreading of grout;
- pollution of groundwater;
- dust distribution.

Furthermore, the hazardous substances have to be considered during the total process of treatment (from transportation to the grouting). Chemical reactions between groundwater and hardened grout have to be considered. Chemical materials injected have to be environmental friendly and non-toxic. In the past, some incidents with chemical grouts, as water poisoning using acrylamide for treatment in Japan in 1974, has led to no use of hazardous substances.

Moreover, the European Standard (EN 12715, 2000) reports that for the safety of personnel, the following potential problems should be considered:

- dust from chemicals which are toxic for the skin, eyes or respiratory system;
- fumes released from liquid mixtures;
- grouts or grout components harmful on contact with the skin;
- contamination of groundwater;
- mixing of chemical which can cause explosion;
- disposal of refuse or wastewater.

2.6.4 Treatment parameters

The most important treatment parameters are summarised below:

- type of drilling;
- type of injection pipes;
- grout composition (types and proportions) and characteristics;
- number of passes;
- grout volume to be injected, pressure and duration for each pass.

Moreover, it is important to consider appropriate:

- grid of treatment;
- maximum depth of treatment.

2.7 Earthquake drains

2.7.1 Principle

The Earthquake (EQ) drains are prefabricated vertical drains with high flow capacity (Rollins et al., 2004).

The EQ drains provide a dissipation of pore water pressure excess generated into saturated cohesionless soils during the earthquake before liquefaction occurs. For this reason, they can reduce the liquefaction potential.

As reported by Rollins et al. (2004), EQ drains are very similar to smaller prefabricated vertical drains (PVD), but have a greater flow capacity. In particular, 100 mm diameter EQ drain has very large flow volume (0.093 m^3/s), conversely conventional PVD has lower flow capacity ($2.83 \cdot 10^{-5}$ m^3/s for a gradient of 0.25) and 1 m diameter stone column has $6.51 \cdot 10^{-3}$ m^3/s.

The flow into the earthquake drains is governed by open pipe flow equations (Manning's equation), conversely sand drains are governed by Darcy's law.

The drains have not structural function and are usually used in conjunction with other ground improvement techniques (such as vibro compaction or deep mixing methods).

The EQ drains consist of perforated corrugate plastic pipes (common diameter = 75÷150 mm) sheathed in a geosynthetic filter to prevent the particles flow into the drain. Appropriate dimensions of the apparent open size (AOS) of the geotextile are needed in order to avoid the clogging of the core. A common used filter criteria is that proposed by Carrol (1983), and reported as follows:

$$O_{95} \leq (2{\sim}3)D_{85} \qquad and \qquad O_{50} \leq (10\ to\ 12)D_{50} \qquad \text{Equation 2-7}$$

where:
O_{95} = AOS of filter
O_{50} = size which is larger than 50% of the fabric pores
$D_{85;}$ D_{50} = diameter of 85% and 50% of passing soil particles by weight

The function of the plastic pipe is to create a water path with low resistance, support the geotextile and provide resistance to the drain.

The drain is installed by means of a mandrel, inserted by vibration and removed at the end of the procedure (Figure 2-26). At the bottom base of the drain is positioned an anchor fixed plate that prevent the soil from entering into the drain during installation and fixes the drain during the removal of the mandrel. The dimensions of the mandrel are minimized to reduce the soil disturbance caused by its insertion (smeared zone with lower permeability). The installation procedure must guarantee the continuity of the drain in order to ensure its efficiency. The water coming from the drains have to be delivered using appropriate methods.

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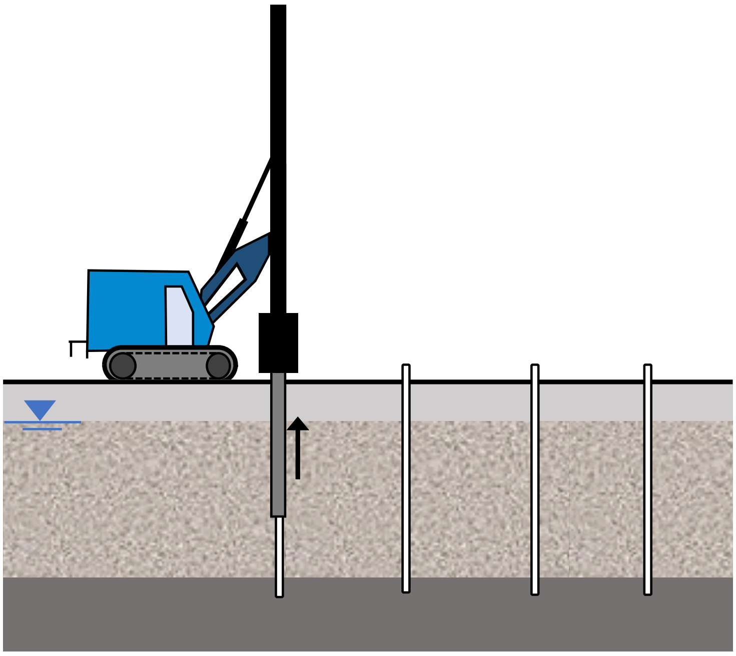

Figure 2-26: Earthquake drains procedure.

The drains have to guarantee a suitable discharge capacity, some suggestions are provided by Chu & Raju (2013) based on discharge factor defined by Mesri & Lo (1991), and reported in Figure 2-27.

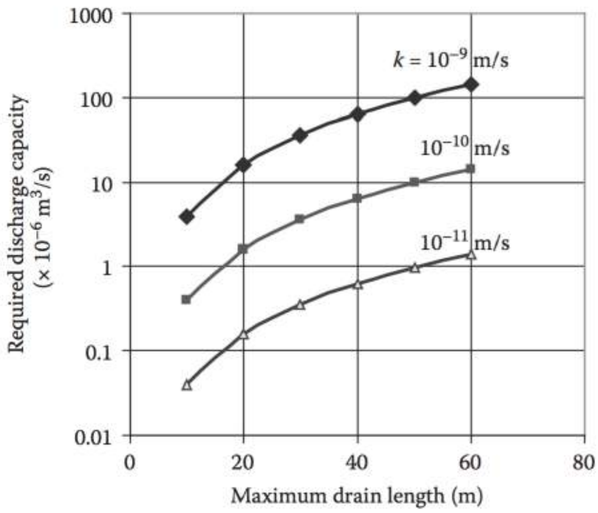

Figure 2-27: Required discharge capacity for PVD (from Chu & Raju, 2013).

Since the influence zone of the drains is affected by their spacing, the EQ drains are usually installed in a triangular grid pattern, with 1÷2 m of space centre to centre intervals, ensuring a more uniform consolidation

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process compared to a square grid pattern. The diameter of the influence zone d_e can be calculated by considering equivalent cross-sectional area, related to the layout of the grid (Figure 2-28).

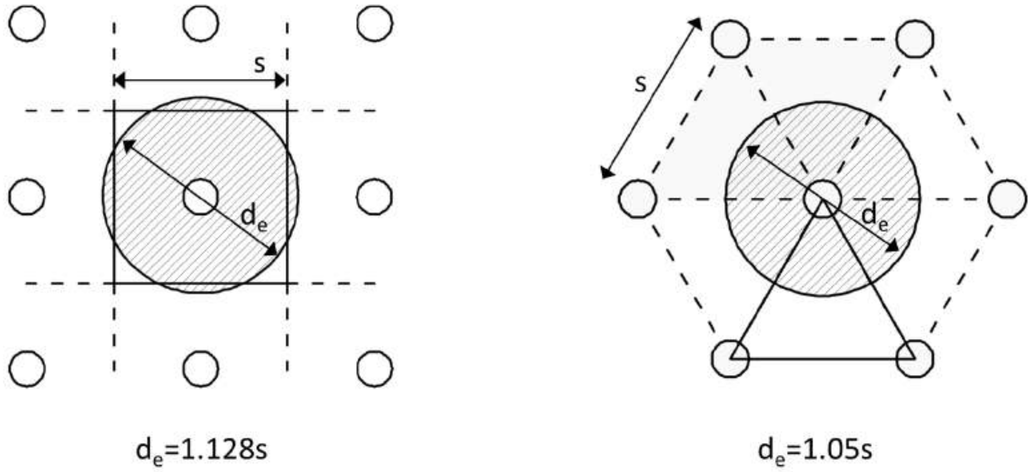

Figure 2-28: Grid of treatment and influence zone.

The European Standard (EN 15237, 2007) provides the specifications for the executions of geotechnical works about vertical drains, including prefabricated band drains (typical dimensions = 100 mm of width and 2÷10 mm of thickness), prefabricated cylindrical drains PVD (typical diameter = 45÷50 mm) and sand drains (typical diameter = 150 mm to 500 mm). In particular, for prefabricated drains the following properties shall be given:

- tensile strength;
- elongation at the maximum tensile force;
- tensile strength of filter, seams and joints;
- velocity index of filter;
- characteristic open size of filter;
- discharge capacity of the drain;
- durability.

Common uses of EQ drains are related to:

- liquefaction mitigation;
- reduction of seismic settlement;
- stabilisation of slope (seismic);
- prevention of lateral spreading;
- groundwater lowering;
- acceleration of consolidation of cohesive soils.

As previously mentioned, the EQ drains, allowing the dissipation of excess of pore water pressure during the earthquake, are an excellent ground improvement method to mitigate the liquefaction potential of susceptible soils (Figure 2-29).

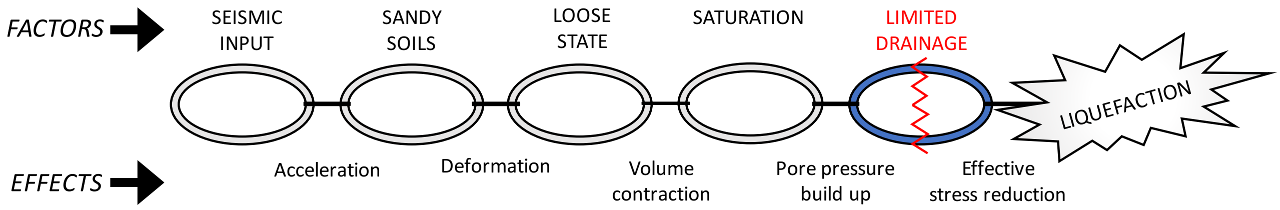

Figure 2-29: Earthquake drains for liquefaction mitigation.

2.7.2 Applicability

The EQ drains are suitable to reduce the liquefaction potential of loose, saturated, cohesionless soils. In particular, the most suitable soils are sands with fine content lower than 5% (NZGS, 2017).

It is recalled that the vertical drains are generally used for highly compressible soils with low permeability (silts, clays, organic silts, organic clays, peat, swamps, sludge) to faster the consolidation process.

The drains modify the hydraulic boundary conditions of the ground and can be considered as zero excess of pore water pressure surface, that accelerate the consolidation process, with beneficial reduction of soil liquefaction susceptibility.

The process can be seen as a sum of vertical and horizontal consolidation process. One of the solutions for radial consolidation was given by Barron (1948). Seed and Booker (1976) developed design charts by means of analytical method, developing and using an infinite element method code (LARF). They considered individual cells (that work as drains) surrounded by an infinite number of identical cells in all directions. Seed and Booker (1976) considered the pore water pressure dissipation only in horizontal direction (throw the drains). The drain does not increase the stiffness of the ground and has infinite permeability. The rate of increase in pore water pressure is introduced using empirical data. More recently, Pestana et al. (1997) developed an improved finite element computer code FEQDrain, introducing composite drainage materials, that can be used for EQ drains design.

The treatment does not produce refusal spoils and is environmental friendly. Moreover, since it produces very small vibration, it can be used close to existing structures, monitoring the settlement induced by consolidation process (in the case of treatment to accelerate consolidation).

The EQ drains are advantageous because their cost is much less than other alternatives and provide a permanent drainage path.

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2.7.3 Limitations and drawbacks

The installation procedure can cause a soil disturbance that can be reduced by adopting smaller mandrel. Installation by static pushing is preferred in sensitive soils but could cause a deviation of the mandrel.

The shorter drainage path causes a quicker consolidation process resulting in a faster settlement.

Site conditions and topography can limit the feasibility of the treatment. In particular, stiff layers or obstructions (i.e. rocks, concrete, wood) may require predrilling to penetrate. If the layer to be drained needs predrilling, the use of the drains is not advisable, because the predrilling phase can cause a void around the drain and thus a soil collapse, resulting in a disturbance of the soil. Installation of EQ drains could not be economically on slope and on not regular and unstable surface (unstable working surface should to be stabilised before the treatment). In very soft layer is difficult to anchor the drains, additional depth may be necessary (with an increase in cost).

The drain material must to be stored properly to prevent sunlight degradation.

Very depth drains require specialized installation apparatus.

Since the EQ apparatus requires sufficient headroom, with limited space the drains can be installed in segment, increasing the cost of the treatment.

In contaminated sites the water coming from drains have to be collected and treated. In such cases, the drains should not penetrate into highly permeable layer.

Before treatment overhead and subsurface utility interference have to be evaluated.

Vertical drains can be installed near existing structures, but not below, limiting their use. For this reason, a good alternative could be the use of horizontal drains (adopting directional drilling) that can be installed below existing structures. An accurate analysis of this type of technique is provided in "Deliverable D4.5" about "Liquefaction mitigation techniques guidelines" (Flora et al., 2019).

2.7.4 Treatment parameters

The most important treatment parameters are shown below:

- flow capacity of the drain;
- dimension of the drain;
- mandrel dimension;
- installation method;
- anchor depth;
- grid of treatment.

2.8 Induced partial saturation

2.8.1 Principle

Induced Partial Saturation (IPS) is an innovative technique that consists of introducing gas bubbles into the soil reducing the degree of saturation. This process allows to increase the liquefaction resistance of liquefiable soils (Copp, 2003; Pietruszczak et al., 2003; Okamura & Soga, 2006; Okamura & Teraoka, 2006; Okamura et al., 2006, 2011; Yegian et al., 2006, 2007; Pande & Pietruszczak, 2008).

Partial saturation of soil increases the liquefaction resistance because the gas/air in the soil has low volumetric stiffness. In particular, during undrained cyclic loading if the soil tends to contract, the gas/air volume decreases reducing the pore water pressure growth. This effect is increasingly evident as the degree of saturation decreases, although it is already evident for small reductions (saturation degree = 99%).

Several methods have been proposed in literature for introducing gas bubbles into the sand, including:

- air injection (Okamura et al., 2011);
- water electrolysis (Yegian et al., 2006, 2007);
- sand compaction pile (Okamura et al., 2006);
- use of sodium perborate (Eseller-Bayat, 2009);
- drainage-recharge (Yegian et al., 2007).

Another potential method for introducing gas bubbles into the soil is to apply biotechnologies that involve the formation of small gas bubbles using a microbial denitrification process.

IPS represents a new technique that is still little used today although it has many advantages compared to other ground improvement techniques (easier, cheaper, environmental friendly and requires a small treatment apparatus). On the other hand, there are numerous experiments carried out in the laboratory.

One of the most advantageous techniques seems to be *air injection* because it is very easy and cheap, therefore the following considerations refer to this method.

Air injection techniques involve the injection of air bubbles in the ground below the water table. The air injection is provided by means of vertical or horizontal pipes, as summarised in Figure 2-30. The latter ones require directional drilling technologies. The air bubbles cause a reduction of the saturation degree of the soil and thus an increased liquefaction resistance compared to the saturated conditions. The air is injected into the soil by means of PVC or stainless steel pipes with a number of small holes. The air injection apparatus therefore consists of injection pipes (EPA (1994) reports typical diameter of 2.54 cm to 12.7 cm), manifold piping and the air compressed equipment. Before air injection, the borehole around the injection pipe have to be refilled with several different materials to prevent the formation of a preferential pathway for the air.

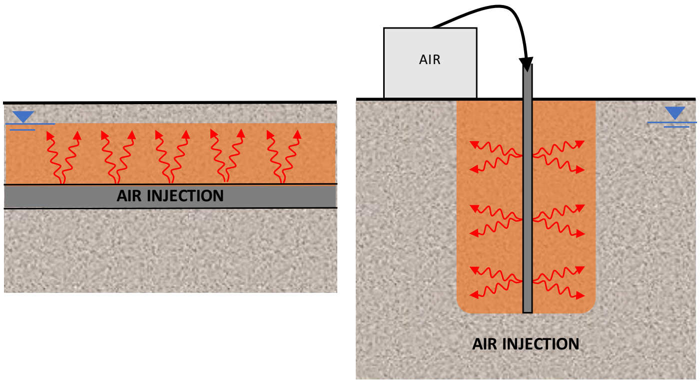

Figure 2-30: Induced partial saturation procedure.

The injection pressure has to be sufficient to displace the water, thus higher than the sum of the hydrostatic pressure and the capillary pressure. Furthermore, the injection pressure must be lower than the in-situ effective stress to prevent soil cracking or fissuring around the injection point; for granular soils the maximum injection pressure (P_{max}) can be calculated as follows (Okamura et al., 2011):

$$P_{max} = min(\sigma'_v; \sigma'_h) \quad \text{Equation 2-8}$$

where:
P_{max} = maximum injection pressure
σ'_v ; σ'_h = effective vertical stress ; effective horizontal stress

According to Camp et al. (2010), air injection technology is commonly used for in-situ environmental treatment to promote volatilization of contaminants (such as solvents or gasoline) and microbial activity to eliminate less volatile contaminants (such as diesel or jet fuel). A schematic representation of typical air sparging system for environmental treatments is provided by EPA (1994) and reported in Figure 2-31.

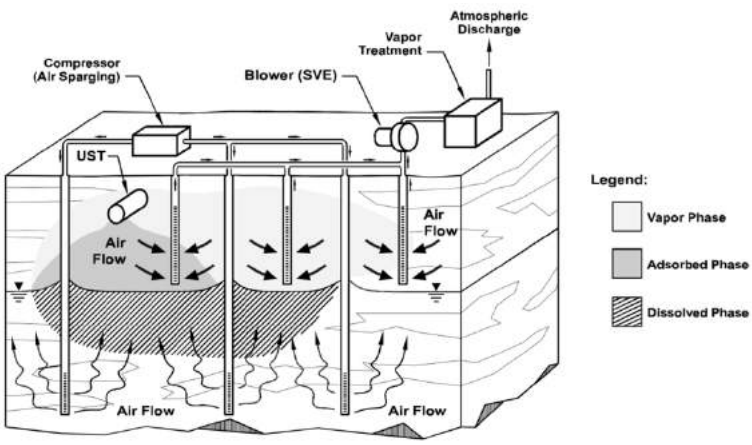

Figure 2-31: Typical air sparging system for environmental applications (from EPA, 1994).

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As mentioned above, the injection of air into the soil (or other IPS methods) reduces the degree of saturation and increases the liquefaction resistance of liquefiable soils (Figure 2-32).

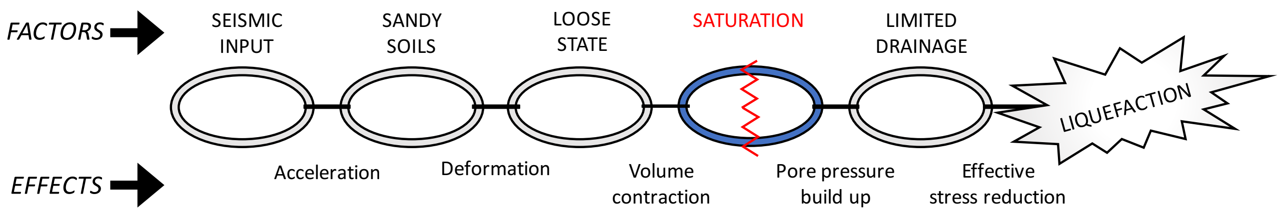

Figure 2-32: Induced partial saturation for liquefaction mitigation.

2.8.2 Applicability

This type of methods is related to the treatment of liquefiable soils in order to the increase the liquefaction resistance, thus to treat saturated cohesionless soil. In particular, air spreading is suitable for treating soils with high permeability (granular soils). Some suggestions are provided by EPA (1994), and reported in Table 2-11, using the intrinsic permeability, $k_{(intrinsic)}$, evaluated by means of Equation 2-9.

Table 2-11: Intrinsic permeability and sparging effectiveness (modified from EPA,1994).

INTRINSIC PERMEABILITY (cm^2)	AIR SPARGING EFFECTIVENESS
$k_{(intrinsic)} \geq 10^{-9}$	Generally effective
$10^{-9} \geq k_{(intrinsic)} \geq 10^{-10}$	May be effective; need further evaluation
$k_{(intrinsic)} < 10^{-10}$	Marginal effectiveness to ineffective

$$k_{(intrinsic)} = k \cdot \left(\frac{\mu}{\rho g}\right) \quad \text{Equation 2-9}$$

where:
$k_{(intrinsic)}$ = intrinsic permeability (cm^2)
k = hydraulic conductivity (cm/s)
μ, ρ = water viscosity (g/cm·s) and water density (g/cm^3)
g = gravity acceleration (cm/s^2)

In the design of this type of treatment the airflow pattern and the zone of influence are very important. The zone of influence of the air injection is affected by soil type and stratification, the injection pressure of the air and the depth. As this is a very complex problem, in situ experimentation and numerical simulations are generally used. A rigorous analysis for the air injection design is provided in the "Deliverable D4.5" about "Liquefaction mitigation techniques guidelines" (Flora et al., 2019).

Since IPS does not generate vibration and requires small treatment apparatus, can be applied near (or below) existing structures.

2.8.3 Limitations and drawbacks

The presence of clay or silt lenses in the soil to be treated can reduce the effectiveness of the treatment.

An important unresolved issue for IPS is whether or not gas bubbles can remain in the soil for a long time, ensuring the effectiveness of the treatment over time. Yegian et al. (2007), reported that under hydrostatic conditions there was little increase in the degree of saturation from the original value (from 82.9% to 83.9%) after of 442 days.

Periodic treatments of restoration may be required because there may be an increase in the degree of saturation over time, for this reason the injection pipes must be well preserved, accessible and reusable over time.

A limitation of these type of techniques is that is no easy to evaluate the effectiveness of the treatment (electrical resistivity and bulk wave velocity may be used to estimate the degree of saturation).

2.8.4 Treatment parameters

The most important treatment parameters for air injection may be summarised as follows:

- depth/length of treatment;
- volume of air to be injected;
- air pressure;
- injection rate;
- injection spacing;
- treatment layout.

Other treatment parameters and information must be considered for the other types of IPS treatment, characteristic for each type of technique.

2.9 Vibro replacement

2.9.1 Principle

Vibro replacement (VR) is a ground improvement technique that involves the formation of dense granular columns into the in-situ soil (stone columns).

Vibro replacement technique extends the limits of application of vibro compaction, including cohesive soils, granular soils with high fine content and layered soils. In fact, in soils that have a fine fraction, the vibrations are not able to separate the particles to achieve a denser configuration, for the cohesive forces. In these cases, the VR technique can improve the in-situ soil introducing dense granular columns.

The deep vibratory process, reported in Figure 2-33.a, is similar to that used for vibro compaction previously described in section 2.3, for which the densification is caused by means of a depth vibrator.

With regard to the backfill supply, two methods are recognized, the *top feed method* (Figure 2-33.a-b) and the *bottom feed method* (Figure 2-33.b), and with regard to jetting medium it is possible to refer to the *dry* or to the *wet method*. The main features are summarized below:

- In the *top feed method*, granular backfill is added from the top. Generally, it refers to the *wet system* because of the water jetting used to remove the soft soil, to stabilise the hole and to help the filling process. Conversely, in the *dry method*, the penetration of the vibrator is helped by compressed air and without water jetting.
- In the *bottom feed method*, granular backfill is added from the bottom, from the tip of the vibrator (called vibrocat). The operations are helped by compressed air and without water jetting (*dry method*).
- The *wet systems* involve the replacement of the in-situ soils by coarse backfill and a little lateral displacement, while the *dry systems* involve only the displacement.

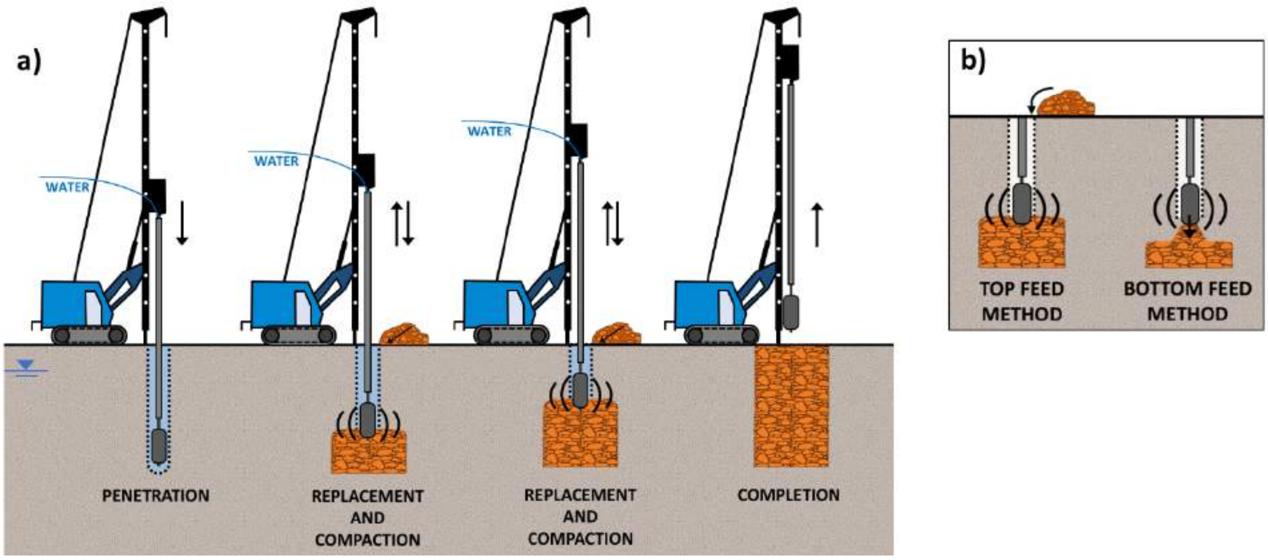

Figure 2-33: Vibro replacement: (a) procedure for wet top feed method; (b) backfill supply methods.

VR treatment builds granular columns characterized by homogeneity, high density and shear strength and reduced compressibility. Furthermore, having high permeability the columns work as drains, dissipating the excess of pore water pressure. The columns and the in-situ soil interact forming a system with increased mechanical properties, resulting in a reduced settlements and consolidation time, increased bearing capacity and reduced liquefaction potential. Thus, the main mechanism is the densification of the soil between the columns (displacement+compaction). Treatment grids can be used to mitigate the liquefaction susceptibility of soils because increase the density of the soils, increase the in-situ lateral stress, replace liquefiable soils with non-liquefiable soils, reinforce the original soils and provide a drainage path for the dissipation of excess of pore water pressure, as reported in Figure 2-34.

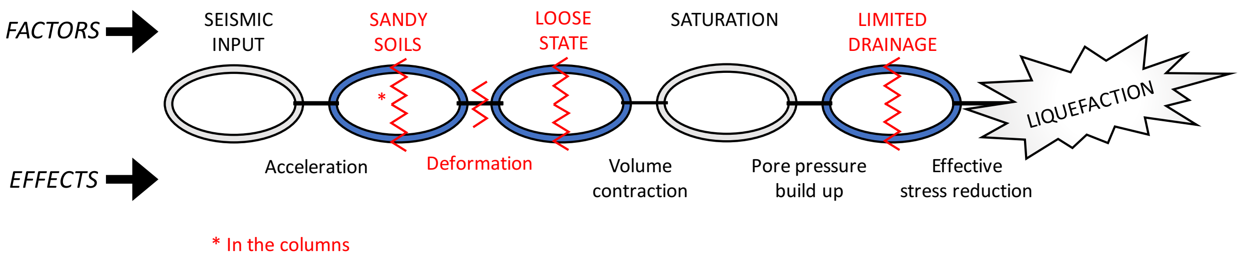

Figure 2-34: Vibro replacement for liquefaction mitigation.

2.9.2 Applicability

Vibro replacement is a technique suitable for all types of soils, extending the range of soils suitable for VC, in particular, soils with grain size distributions falling into the zone C and D of Figure 2-8 (not suitable for VC) can be improved by VR. In fact, contrary to the vibro compaction technique, that is suitable only for granular soils, VR is also suitable for cohesive soils, granular soils with high fine content and in the case of layered soils.

A minimum strength of the in-situ soil (c_u=5kPa) is required to give sufficient containing pressure (Kirsch & Kirsch, 2010).

Pre-drilling is often necessary for stiff soils (crust or layers) to penetrate at depth. Moreover, in stiff soils, the installation process of closely columns (columns with a distance lower than 3 times the diameter) could cause a heave of the ground (Kirsch & Kirsch, 2010).

Some considerations about soils suitable for VR are reported in Table 2-12.

Table 2-12: Suitability assessment for VR (modified from Kirsch & Kirsch, 2010; Degen, 1997).

SOIL TYPE	USCS	COMMENT ON SUITABILITY FOR VR
Silty sands	SM	VR necessary and suitable for silt content > 10%
Clayey sands	SC	VR with marginal overall compaction effect, very fast draining after treatment
Inorganic clays (low plasticity)	CL	$c_u \geq 5$ kPa recommended for upper 3 m, potential difficulties for vibrators to penetrate with $c_u > 50$ kPa (very stiff conditions)
Inorganic clays (high plasticity)	CH	As for CL, but nor suitable when w_n too close to w_L
Silts and clays with $w_L < 50$	ML	Pre-boring necessary when dry
Inorganic and organic silts and clays with $w_L > 50$ and high plasticity	MH CH OH PT	Collapsing soils not suitable Soils generally not or only marginally treatable In excess of 1 m thickness not suitable

(w_L=liquid limit)

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No. 700748

2.9.3 Limitations and drawbacks

The backfill material should have sufficient hardness and strength to resist the action of the vibrator. Furthermore, it should not contain organic or deleterious materials and should be chemically inert. The grain size distribution of backfill should allow a dense configuration and a high permeability of the columns. Usually, for wet method, rounded or subangular stone or gravel (size of 30÷60 mm), uniformly graded, which can pass throw the space between the vibrator and the hole are used. During the treatment, the smallest coarse particles of in-situ materials fill the void of the columns and the fine in-situ materials are transported on the surface by the water. For bottom feed method - dry system, since the backfill is added from the tip of the vibrator, it is necessary to use smaller materials such as finer gravel, crushed stone or well-graded sand with size of 10÷40 mm (Kirsch & Kirsch, 2010).

In the case of stiff soils may be necessary a pre-drilling to allow the penetration of the probe.

When the treatment is performed using the dry bottom feed method, it is recommended to reduce the designed diameter of the columns by 5%, to take into account of a zone with reduced permeability on the boundary of the column.

Any possible deviation from the vertical of the vibrator caused by the variability in the soil strength should be detected and corrected.

When the treatment is performed to prevent liquefaction, a high permeability of the column is required and maintained over time during the earthquake; for this purpose, Saito et al. (1987) proposed a filter criterion for vertical drains used to prevent liquefaction of sands similar to the Terzaghi's filter rule:

$$20 \cdot d_{15} < D_{15} < 9 \cdot d_{85}$$ **Equation 2-10**

where:
D_{15} = diameter of passing particles at 15% by weight, for the filter material
$d_{15,}$ d_{85} = diameter of passing particles at 15% and 85% by weight, for the natural soil

The apparatus used for VR treatment, as for VC, requires sufficient headroom.

The VR technique, as reported for VC, is a potential source of noise, vibration and settlements close to treatment location, thus causing possible damage to adjacent structures. Therefore, vibration measurements should be carried out to evaluate the minimum distance to the closest structures.

When VR is performed using the wet method, the treatment generates turbid water, thus, to avoid pollution, the resulting water needs to be purified before being discharged or reused for the production of other coarse columns. Instead, in the case of the dry method, sludge and water are not produced.

As for the vibro compaction technique, the process of VR can disperse contaminants, thus, if there is a risk of contamination, an alternative treatment method should be evaluated.

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No. 700748

2.9.4 Treatment parameters

The most important treatment parameters are summarised below:

- type and gradation of the backfill;
- penetration depth of the probe;
- mean extraction intervals;
- vibration frequency;
- duration of compaction;
- pressure of the water/air jets;
- grid of treatment;
- diameter of the columns.

2.10 Deep mixing

2.10.1 Principle

Soil mixing techniques involve the mixing of in-situ soil with binder materials, like cement, lime, fly ash, slag or other types of binder. The improvement is based on the chemical interactions of the clayey soils with the binder, the bond between the particles and the filling of the voids with the products of the reactions.

Different types of apparatus have been developed (Schaefer et al., 2017b): single and multiple vertical shaft mixing tools, with rotated mixing shaft and mounted blades or cutting tools, to form columns; horizontal rotating circular cutters, with blades or teeth mounted on two wheels rotating in opposite horizontal direction to form panels; chainsaw type cutters with cutting teeth to create continuous trenches and horizontally rotating, toothed drums attached to the end of an excavator to treat shallow areas.

According to Topolnicki (2013), a general classification for in-situ soil mixing is based on three main factors: the binder form, the mixing principles and the location of the mixing action (as reported in Figure 2-35).

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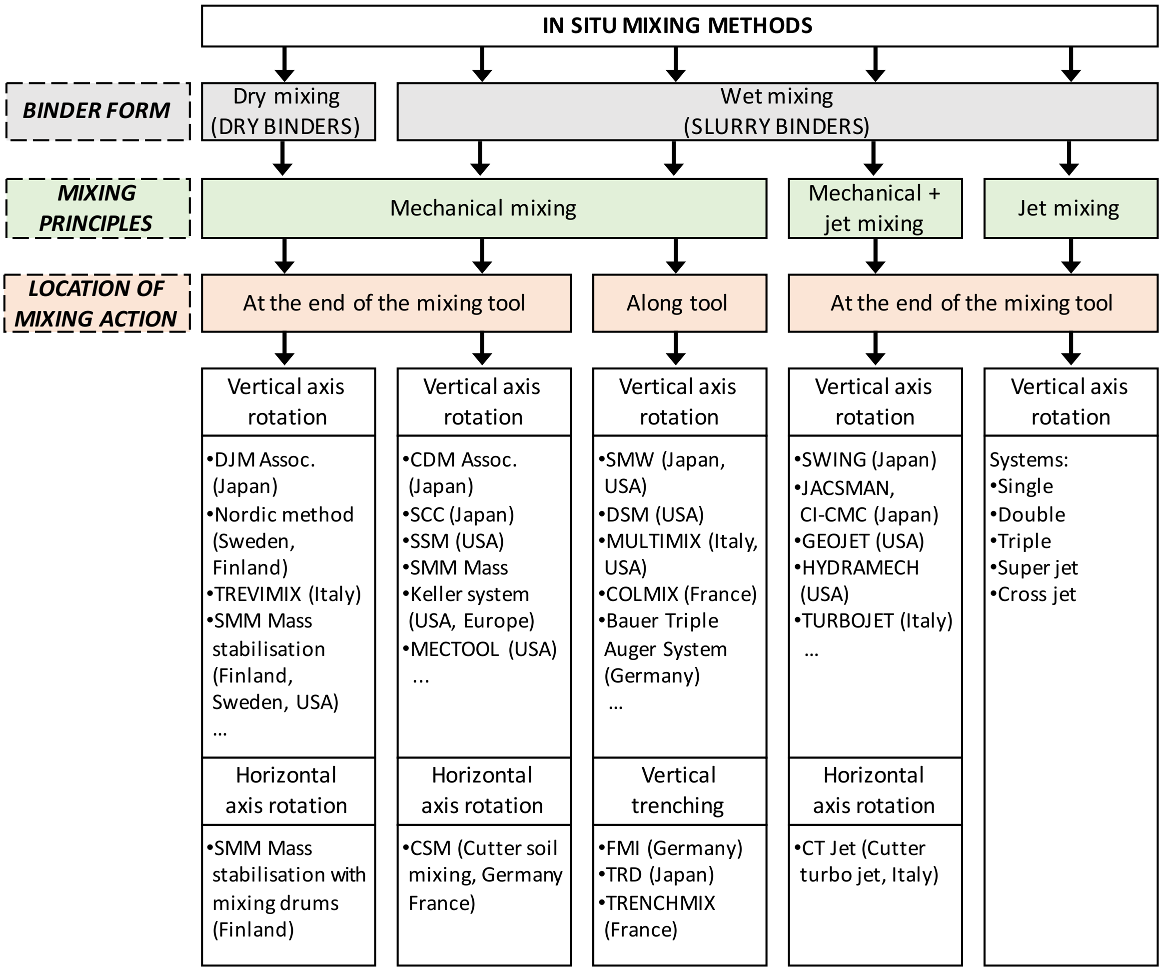

Figure 2-35: Classification of soil mixing (modified from Topolnicki, 2013).

Referring to the depth of treatment, in-situ soil mixing can be classified in *deep mixing method* (DMM) and *shallow mixing method* (SMM). Limit depth of 3 m is introduced for DMM by EN 14679 (2005). In the most common cases deep mixing method refers to the vertical shafts mixing tools, to form column elements. Combining mechanical mixing with other ground improvement techniques, as jet grouting, other methods have been developed and classified as hybrid methods by EN 14679 (2005). Moreover, deep mixing differs from mass mixing, without a precise distinction. As reported by Schaefer et al. (2017b), the main distinctions are based on three aspects: the percentage of the treated area for mass mixing is about 100%, the design strength for the mass mixing and the depth of treatment are lower compared to the deep mixing.

Focusing on deep mixing methods (similar terms, some of which are proprietary, include *deep soil mixing*), two different mixing methods are available:

- *Dry deep mixing*, if the in-situ soil is mixed with a dry binder (powder or granular);
- *Wet deep mixing*, if the in-situ soil is mixed with a binder-water slurry (premixed).

According to Schaefer et al. (2017b), DMM methods can differ for the following characteristics:

- *Mixing method*: dry or wet;
- *Mixing equipment*: vertical-axis rotary (single-axis and multiple-axis), horizontal-axis rotary (cutter wheel soil mixing and toothed-drum mixing) and vertical chainsaw type;
- *Delivery of the binder*: end delivery and shaft delivery (for vertical axis mixing);
- *The pressure of the binder*: low, medium and high pressure.

The most common binders are cement, used both for the wet and the dry method, and lime and lime-cement mixtures used for the dry method.

The binder is usually injected from ports located near the cutting and mixing blades/teeth. For the wet method, the slurry binder can be delivered during the penetration or/and the withdrawal of the mixing machine. In particular, for vertical axis mixing the common practice injects the slurry during the penetration process from nozzles located on the bottom of the shaft. The soil and the slurry are mixed two times, both during the penetration and during the withdrawal of the machine. Good practices often involve double passing of the mixer and stop interval to improve the mixing process. For cutter wheels, the penetration is often performed using water, and then a slurry with low water/binder ratio is injected during withdrawal. For dry method-vertical axis, the penetration is performed using air jets to break the soil and then the powder or granular binder is added during the machine withdrawal from ports above the mixing blades.

Thus, referring to EN 14679 (2005) for column elements, the treatment procedure can be summarised as follows:

- positioning of the mixing tool;
- penetration of the mixing tool to the prescribed depth with disaggregation and injection of the binder (only for wet method);
- the mixing tool is withdrawn, with injection of the binder (for the dry or wet method).

A schematic procedure is reported in Figure 2-36.

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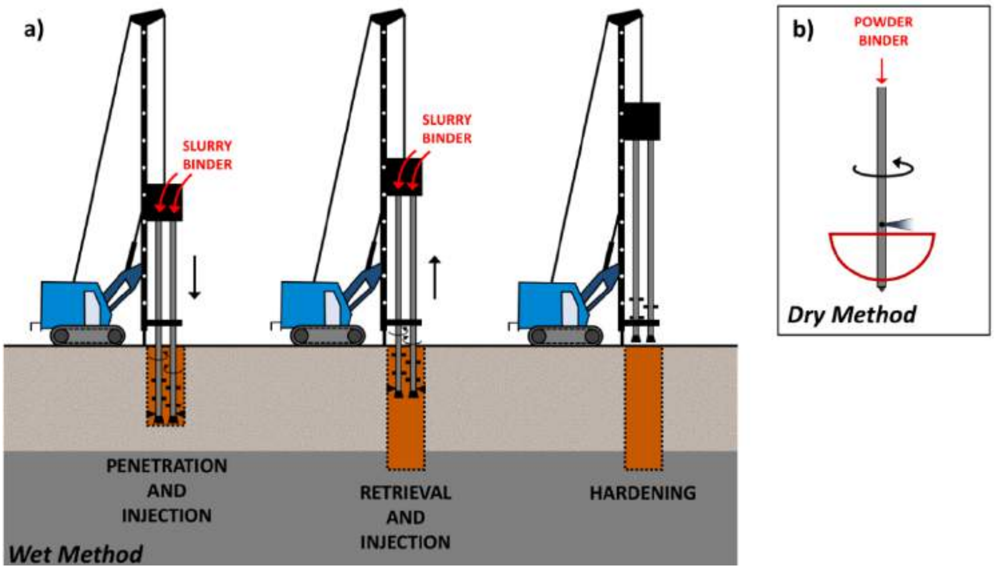


Figure 2-36: Deep mixing method: (a) Wet method procedure; (b) Mixing tool of the dry method.

It is important to highlight that the wet method treatment produces spoils that can be used as filler material, conversely, the dry method produces no spoils, or very little spoils. Some useful information, for wet methods and vertical mixing axis of treatment, is reported in Table 2-13, and for dry methods and vertical mixing axis, in Table 2-14.

Table 2-13: Mixing conditions for several wet deep mixing methods (from Topolnicki, 2013).

	Selected wet DM methods for mechanical mixing about vertical axis(es)							
Technical specification	*CDM (Stand. and MEGA)*	*CDM Land4*	*SCC*	*HB-Keller USA/Europe*	*Bauer*	*SMW*	*DSM*	*COLMIX*
Number of mixing shafts	2 1:older syst.	4	1 possible 2	1, 2, 3	3 possible 1	1-3, 5 usually 3	1–6 usually 4	2, 3, 4
Diameter of mixing tool [m] (shaft spacing)	1.0: S 1.2/1.3: M (variable)	1.0/1.2 (variable)	0.6-1.5 1.2 (2 shafts)	0.5-2.4 :U 0.6-1.2 :E	3 × 0.37 3 × 0.55 3 × 0.88	0.55–1.5 usually 0.9, (variable)	0.8–1 usually 0.9	0.23–0.75 :2 0.36–0.50 :3 0.50–0.75 :4
Realistic maximum penetration depth [m]	50 (55) 30: M 1.2 20: M 1.3	25	20	20	0.37: 10.5 0.55: 15.7 0.88: 25	35 (50)	35	20
Penetration/Retrieval velocity [m/min]	P: (0.3) 0.5-1 R: 0.7–1 (2)	P: 0.7–1 R: 1.0	P: 1.0 R: 1.0	P: 0.3–1 R: 1–2	P: 0.2–1 R: 0.7–1 (5)	P: 0.5–1 R: 1.5–2	P: 0.6–1 R: 1–2	P: 0.8 R: 1.0
Penetration/ Retrieval rotation speed [rpm]	P: 20 R: 40	P: 20 R: 40	30–60	P: 20–25 R: 40–60	20–40 (80)	P: 14–20 R: higher	15–25	8–30
Injection during Penetration and/ or Retrieval	P and/or R, restroking at the bottom	P and/or R, restroking at the bottom	usually P, restroking at the bottom	P (+ R) :U P + R with restroking:E	P and/or R, P (30%–50%) restroking	P and R, restroking common	P (+ R), restroking at the bottom	P (+R) ev. restroking in clays
Water/Cement ratio	0.6–1.3 av. 1.0	0.6–1.3 av. 1.0	0.6–0.8 clays, 1.0–1.2 sands	1–1.5: U 0.6–1.2: E	0.6–2.5	0.7–2.5	1.2–1.75 av. 1.5	0.7–2.5
Footprint area of the mixing tool [m²]	1.5: 2 shafts 0.8: 1 shaft 2.17/2.56: M	2.83–3.14 or 4.21–4.52	0.3–1.75 2.25 two shafts	usually 1.1–4.5: U 0.5–1.5: E	0.44: 3 × 0.37 0.94: 3 × 0.55 2.35: 3 × 0.88	0.7: 3 × .55 1.7: 3 × 0.9 4.7 :3 × 1.5	2.5 (4 shafts, tangential)	0.08–0.95 :2 0.29–0.57 :3 0.76–1.60 :4
Amount of added (dry) binder [kg/m³]	70–300 av. 140–200	70–300 av. 140–200	150–400	150–275: U 250–450: E	80–500	200–750	120–400	100–550
Productivity per shift (one rig)	100–200 m³	500–700 m³	100 m² wall 400 m col.	250–750 m³: U 75–200 m³: E	30–300 m³	100–200 m³	200–300 m² wall	100–300 linear m

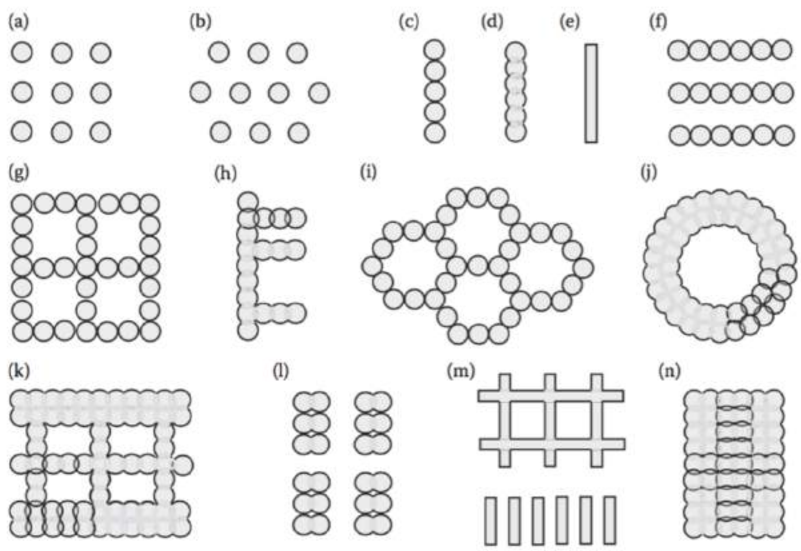
CDM = Cement deep mixing; SMW = Soil mixed wall; DSM = Deep soil mixing.

Table 2-14: Mixing conditions for several dry deep mixing methods (from Topolnicki, 2013).

	Selected dry DM methods		
Technical specification	*DJM*	*Nordic method*	*Trevimix*
Number of mixing shafts	2 (standard), 1	1	2 (more common), 1
Diameter of mixing tool [m]	1.0 (standard) 1.3 (modified)	0.5–1.0 possible 0.6, 0.8 standard	0.8–1.0 (standard)
Realistic maximum penetration depth [m]	33	25 (30)	30
Penetration/Retrieval velocity [m/min]	0.5–3 (4), 7 (1 shaft) typically: P: 1.5, R: 0.7, 0.9 (R: 15 mm/rev.)	P: 2–15 R: 2–6 (R: 15–30 mm/rev.)	P: 0.4 R: 0.6
Penetration/Retrieval rotation speed [rpm]	P: 24, 32 (Electr.) R: 48, 64 (Electr.) P/R 21–64 (Hydr.)	R: 100–220 (150–180 typically)	10–40 P: 20 typically R: 30 typically
Injection during Penetration/Retrieval	R (P used: air/binder)	R (P possible)	R (P used: air/binder)
Footprint area of the mixing tool (max.) [m²]	0.78 : 1 × 1.0 m 1.56 : 2 × 1.0 m 2.65 : 2 × 1.3 m	0.28, 0.5 (0.78)	0.78 : 1 × 1.0 m 1.56 : 2 × 1.0 m
Amount of injected dry binder [kg/m³]	100–400 Cem.:sands 200–600 Cem.: peat 50–300 Lime: clay	70–150 150–250 organic soils	150–300 250 typically
Binder supply capacity per shaft [kg/min]	25–120 standard, up to 200 mod. version	40–230	around 100
Injection pressure [kPa]	P: 100–600 R: 600–100	400–800	600–1,000
Productivity [m³/shift]	300–700	150–300	150–220

DJM = dry jet mixing

The treated elements can be used singularly or overlapping several elements to form grids, walls or blocks. Some examples of deep mixing patterns are provided by Topolnicki (2013) and reported in Figure 2-37.



a) Column-type (square arrangement)
b) Column-type (triangular arrangement)
c) Tangent wall
d) Overlapped wall
e) Trench/CSM wall
f) Tangent walls
g) Tangent grid
h) Overlapped wall with buttresses
i) Tangent cells
j) Ring
k) Lattice
l) Group columns
m) Multiple trenches/CSM walls
n) Block

Figure 2-37: Examples of DMM patterns (from Topolnicki, 2013).

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DMM can increase the shear strength, reduce the deformation, reduce the permeability and the compressibility of the treated area. The main applications related to DMM are:

- foundation support;
- hydraulic cut-off walls;
- excavation support walls;
- environmental remediation;
- ground improvement;
- reduction of liquefaction susceptibility.

In particular, in liquefiable soils DMM treatment is usually performed using a grid of treatment capable of mitigating the liquefaction susceptibility by reinforcing the soil as reported in Figure 2-38; furthermore, in the treated elements, the soil prone to liquefaction (sandy soils) become non-liquefiable mixed soil.

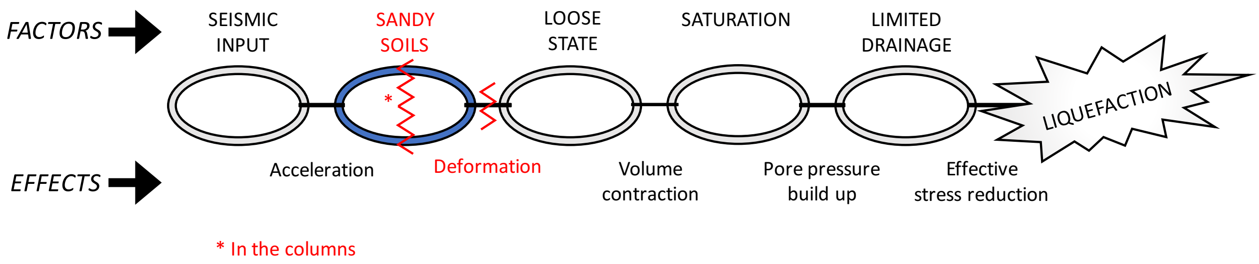

Figure 2-38: Deep mixing for liquefaction mitigation.

If necessary, the treated elements can be reinforced with vertical steel reinforcement, lateral bracing or tie-back anchors.

2.10.2 Applicability

Deep mixing method can be used to improve all type of soils suitable for treatment with binders (cement, lime, other types of binder), as reported in Figure 2-39.

In particular, the wet method is suitable for treating coarse-grained, fine-grained and organic soils and peat, conversely, the dry method is suitable for very week soils as soft fine-grained, organic soils and peat. The dry method is possible in soils with sufficient water content to allow the chemical reaction of the binder with water and soil.

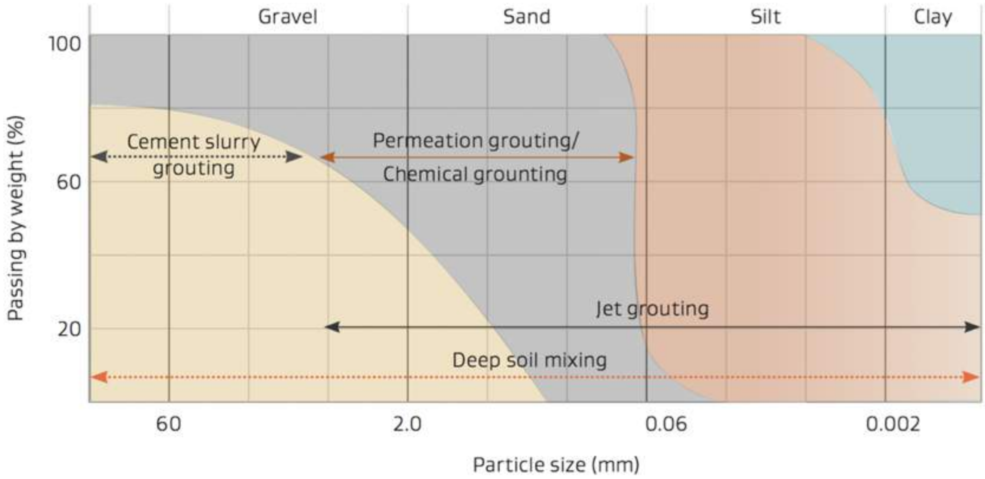

Figure 2-39: Soils suitable for deep mixing method (from NZGS, 2017).

Moreover, the dry method requires more vigorous mixing to obtain the same homogeneity of the wet method. The homogeneity of the mixed soil is an important parameter to this type of treatment, which depends on the mixing time, type of mixer, native soil, binder form and energy of the injection. Thus, as reported by Topolnicki (2013), for mechanical mixing, a simplified index was introduced in Japan to evaluate the degree of homogeneity, named *blade rotation number*, where the soil conditions are considered indirectly. The blade rotation number can be obtained with the equations reported in Table 2-15 for different types of injection (i.e. case 1, 2 or 3).

Table 2-15: Blade rotation number.

	Case 1	Case 2	Case 3
INJECTION	During penetration	During withdrawal	Partial injection during penetration Main injection during withdrawal
OUTLET	Below the blades	Above the blades	Lower outlet during penetration Upper outlet during withdrawal
BLADE ROTATION NUMBER	$T = \Sigma M \cdot \left(\frac{R_p}{V_p} + \frac{R_w}{V_w}\right)$	$T = \Sigma M \cdot \left(\frac{R_w}{V_w}\right)$	$T = \Sigma M \cdot \left(\frac{R_p}{V_p} \cdot \frac{W_p}{W} + \frac{R_w}{V_w}\right)$

where:
T = blade rotation number (rev/m)
ΣM = total number of mixing blades
R_p = rotational speed during penetration (rev/min)
V_p = penetration velocity (m/min)
R_w = rotational speed during withdrawal (rev/min)
V_w = withdrawal velocity (m/min)
W_p = amount of binder injected during penetration (kg/m^3)
W = total amount of injected binder (kg/m^3)

In particular, to ensure homogeneity to the treated soil, on the base of field data, blade rotation of 360 has been recommended in Japan for loose sands and clays using wet method, and T = 430 in Poland for silty/sandy clay using wet method. For dry method, T = 274 or 284 for DJM and 200÷400 for the Nordic method (Topolnicki, 2013).

The deep mixing method is suitable for treating contaminated soils, stabilising many types of contaminants.

The maximum treatment depth is about 40 meters.

The wet DMM require large space for the treatment plant, conversely, dry DMM require little space for the light equipment. Moreover, the wet method treatment requires a slurry plant composed by storage silos, slurry mixing equipment, agitation tanks and slurry pumps; conversely, dry method require track-mounted binder delivery unit to storage the powder binder.

2.10.3 Limitations and drawbacks

Deep mixing method is not suitable for treating very stiff soils and soils in very dense conditions. Moreover, obstructions present in the soil (cobbles, boulders, dense sand layers, buried logs) can interfere with the penetration of the mixing apparatus. Conversely, soft in-situ soils may require a work platform to support the equipment.

Some types of organics can interfere with the chemical reactions of the binder, in particular colloidal organics; in this case, slag-cement blends can be used, producing stronger mixtures than pure cement.

DMM treatments are not suitable for areas with buried utilities or in close proximity to existing structures.

DMM is a source of vibration and noise, but lower compared to some other techniques.

The wet method produces spoils that can be used as filler material; if reuse is not possible, the spoils can be used for other projects or have to be disposed.

The treatment costs are generally higher than other techniques. Deep soil mixing is economically advantageous to treat large areas, in particular the dry method is cheaper but not always applicable due to the site conditions and the required strength properties.

2.10.4 Treatment parameters

The most important treatment parameters are reported below:

- air pressure;
- slurry pressure (only for the wet method);
- penetration rate and retrieval rate;
- rotation speed (penetration and retrieval);
- quantity of binder per meter of depth (penetration and retrieval).

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Moreover:

- grid of treatment;
- maximum depth of treatment.

2.11 Jet grouting

2.11.1 Principle

In jet grouting (JG) technique, high-pressure jets of grout/water/air break the soil structure and mix the native soil with the grout to form an improved material known as *soilcrete*.

The typical JG procedure, as summarised in Figure 2-40, is composed of a drilling phase and a subsequent grouting phase. The drilling is carried out with air, water, grout or foam. When the desired depth is obtained, the injection of fluids (grout, air, water) is performed moving upward, combining rotation and translation. In particular, the fluids are injected through small nozzles placed on the tool mounted at the end of the string, named *monitor*.

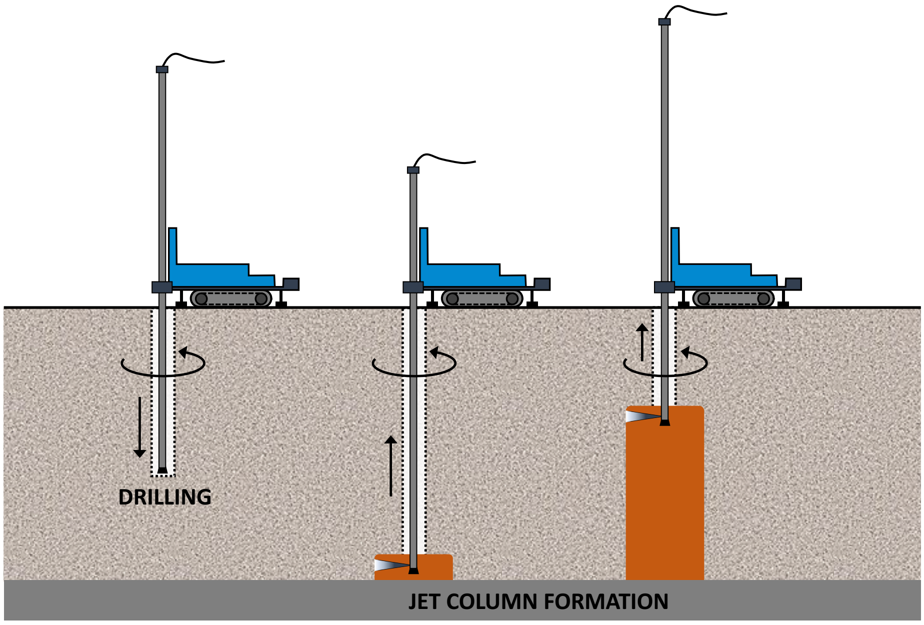

Figure 2-40: Jet grouting procedure.

The injected grout is composed of water (W) and cement (C) with W-C ratio by weight usually ranging between 0.6 and 1.3 (Croce et al., 2014).

Although the classical shape of the treated volume is cylindrical (column), different shapes can be obtained regulating the rotation and the translation of the monitor (e.g. panel, V-shape element and candy-shape element). The consolidated elements have good mechanical properties and reduced permeability.

On the base of the number of fluids injected into the soil, three types of jet grouting system are available: *single*, *double* and *triple fluid systems* (Figure 2-41).

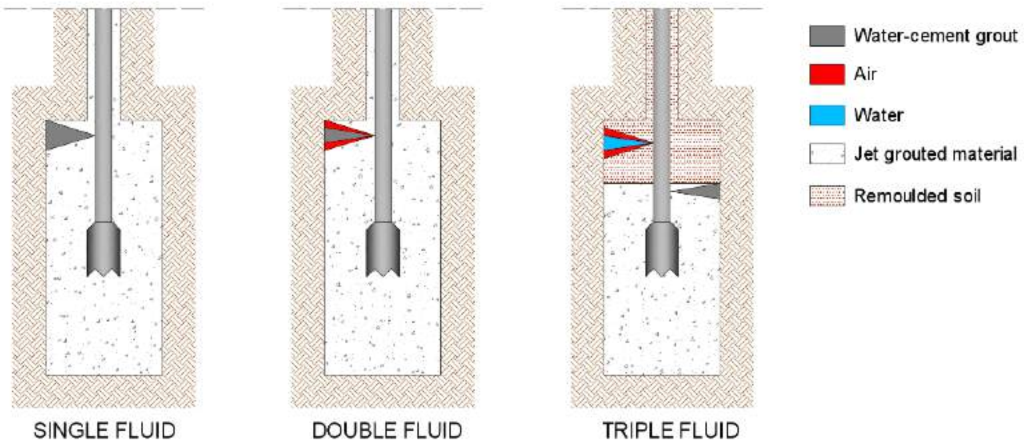

Figure 2-41: Jet grouting systems.

In particular, the main characteristics of the different injection systems are reported below and summarised in Table 2-16:

- *Single fluid system*: soil remoulding and cementation are caused by the W-C grout;
- *Double fluid system*: soil disaggregation and cementation are carried out by the W-C grout. The grout jet is enhanced in effectiveness by a coaxial air jet;
- *Triple fluid system*: disaggregation and cementation are caused by different fluids. Soil disaggregation is caused by a high-velocity water jet surrounded by a coaxial air jet (through nozzles on the upper part of the monitor). Conversely, soil cementation is produced by the W-C grout with lower velocity (through a separated lower nozzle).

Table 2-16. Main characteristics of the jet grouting systems.

SYSTEM	INJECTED FLUID	FUNCTION		
		Disaggregation	Cementation	Auxiliary Jet
SINGLE FLUID	1) W-C grout	✓	✓	X
DOUBLE FLUID	1) W-C grout	✓	✓	X
	2) Air	X	X	✓ coaxial to the W-C grout jet
TRIPLE FLUID	1) W-C grout	X	✓	X
	2) Air	X	X	✓ coaxial to the water jet
	3) Water	✓	X	X

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Typical applications of jet grouting are listed below:

- water barriers (horizontal/vertical barriers with permanent/provisional purposes);
- foundation systems (settlement control and increasing bearing capacity);
- retaining structures;
- tunnels (excavation support/waterproofing barrier);
- excavation support;
- stabilisation of the slopes;
- protection of waterfront structures;
- reduction of liquefaction susceptibility;
- contaminant barriers.

In particular, in liquefiable soils, it is possible to reduce or eliminate the liquefaction susceptibility by creating a "*cellular structure*" to stiffen the soil (case "b" in Figure 2-42) or by creating a cemented mass (case "a" in Figure 2-42).

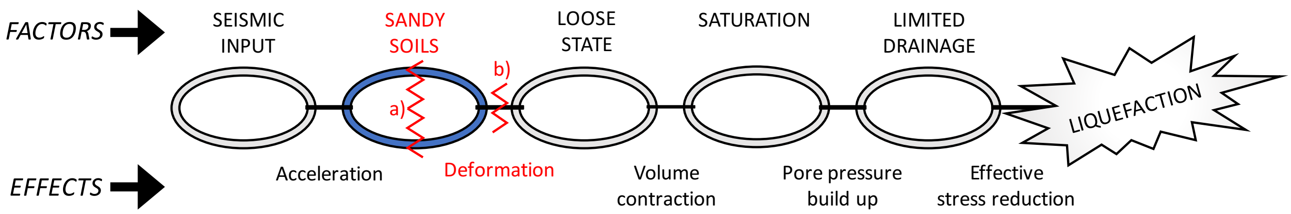

Figure 2-42: Jet grouting for liquefaction mitigation.

2.11.2 Applicability

Jet grouting technique can be used to treat a wide range of soil types, from gravel to clay, as reported in Figure 2-43. The most suitable soils are cohesionless soils or soft cohesive soils.

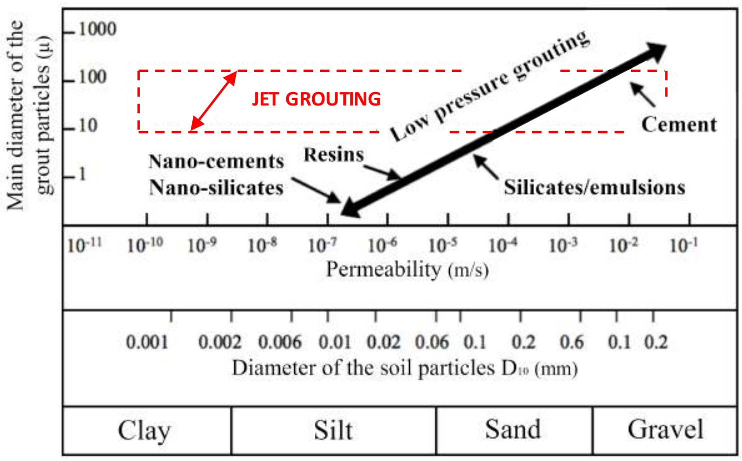

Figure 2-43: Soil suitable for different types of injection (modified from Flora & Lirer, 2011).

It is important to highlight that the soil type and the stratigraphy affect the quality of soilcrete and the erodibility (as summarised in Figure 2-44). In particular, the jet-soils mechanisms (erosion, cutting and seepage) depending on soil grading (Croce et al., 2014):

- *Gravel/ sandy gravels*: the predominant mechanism is erosion. The soilcrete obtained is quite homogeneous. Larger diameters can be achieved by increasing the specific energy (increasing the diameter of the nozzles and the flow rate). In clean gravel the grout seepage could contribute; larger diameters can be better achieved increasing the jet pressure or the flow rate.
- *Sands/ gravelly sands/ silty sands*: the relevant mechanism is erosion. The soilcrete obtained is very homogeneous. Larger diameters can be better achieved by increasing the energy of the jet (increasing the diameter of the nozzles and the flow rate).
- *Silt/ sandy silt/ clay*: the relevant mechanism is cutting. The homogeneity depends on the number of passes. Larger diameters can be better obtained by increasing the jetting time.

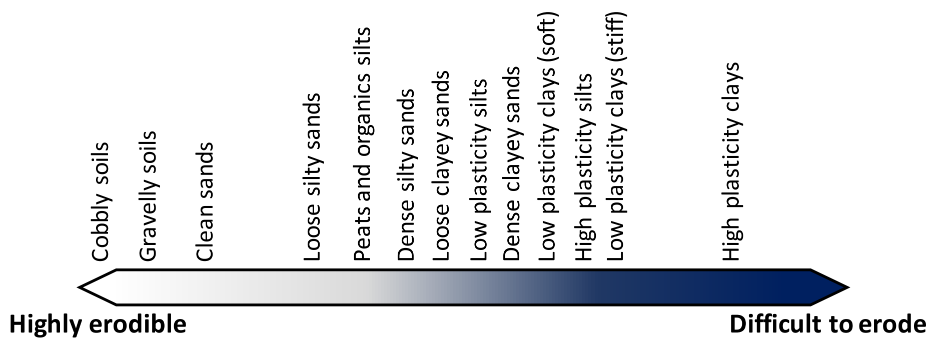

Figure 2-44: Soil erodibility scale (modified from Burke & Yoshida, 2013).

Jet grouting treatments require a plant composed by several tools, the most important are: the W-C grout manufacturing system, the pumps for grout and air, the air compressor, the drilling rig, the string rods, the monitor and the hydraulic circuits.

JG is very suitable for treating soils in difficult operating conditions (confined spaces or difficult to reach) and with light equipment; it is suited for treating areas with a high density of structures or utilities, soils not very homogeneous and where a significant strength is required.

As mentioned above, the most common jet grouted elements are the columns, with a diameter ranging from 0.8 to 5 meters, generally overlapped and constructed by means of a primary/secondary sequence. Table 2-17 reports the typical mean diameters for different types of soil and types of treatment.

Table 2-17: Typical value of the mean diameters (modified from AGI, 2012).

TREATMENT SYSTEM	Moderately stiff clay	Soft silt and clay	Silty sand	Sand and/or gravel
SINGLE FLUID	Not recommended	0.4÷0.8	0.6÷1.0	0.6÷1.2
DOUBLE FLUID	0.5÷1.0	0.6÷1.3	1.0÷2.0	1.2÷2.5
TRIPLE FLUID	0.8÷1.5	1.0÷1.8	1.2÷2.5	1.5÷3.0

The prediction of the diameter of the columns is an important aspect of the project. It depends on the hydrodynamic properties of the jet and on the soil resistance to erosion. The desired value of the diameter can be obtained selecting an appropriate jet grouting system and suitable treatment parameters. Several authors have proposed empirical relations useful to predict the mean diameter of the columns, as reported in Table 2-18.

Table 2-18: List of correlations available in literature (modified from Croce et al., 2014).

REFERENCE	FORM OF CORRELATION	SOIL CLASSIFICATION
Botto (1985); Bell (1993)	Chart	Soil type
Miki & Nakanishi (1984); Shibazaki (1996)	Chart	Coarse grained (N_{SPT})
Xanthakos et al. (1994)	Table	Fine to coarse grained
Kutzner (1996)	Table	Soil type
Tornaghi (1989)	Chart	Coarse grained (N_{SPT}); fine grained (s_u)
JJGA (2005)	Table	N_{SPT}
Tornaghi & Pettinaroli (2004); Flora & Lirer (2011)	Chart	Soil type
AGI (2012)	Table	Soil type
Modoni et al. (2006); Croce et al. (2011)	Chart	Shear strength parameters (φ',s_u)
Wang et al. (2012)	Equation	Soil type
Flora et al (2013)	Equation, charts	Coarse grained (N_{SPT}); fine grained (q_c)

Flora et al. (2013) propose to calculate the mean diameter of the column using the jet energy and the soil resistance based on in situ results, as reported in Equation 2-11 for fine-grained soils and in Equation 2-12 for coarse-grained soils.

$$D_m = D_{ref} \cdot \left(\frac{\alpha_E \cdot \Lambda^* \cdot E'_n}{7.5 \cdot 10}\right)^{\beta} \cdot \left(\frac{q_c}{1.5}\right)^{\delta}$$ **Equation 2-11**

$$D_m = D_{ref} \cdot \left(\frac{\alpha_E \cdot \Lambda^* \cdot E'_n}{7.5 \cdot 10}\right)^{\beta} \cdot \left(\frac{N_{SPT}}{10}\right)^{\delta}$$ **Equation 2-12**

where E'_n is in MJ/m and q_c in MPa, and:
D_m = Mean diameter of the column
E'_n = the specific kinetic energy at the nozzles

D_{ref} = reference diameter (calibrated on experimental data, some indications can be obtained using Table 2-19)
β, δ= coefficients calibrated on experimental data, some indications can be obtained using Table 2-19
Λ* = hydrodynamic coefficient related to the composition of the central jet of eroding fluid, function of the water-cement ratio (for W-C ratio = 1 for single and double fluid systems Λ*=7.5, and for triple fluid = 16)
α_E = coefficient related to the influence of the shrouding air jet on boundary dissipation (calibrated on experimental data Table 2-19)

Table 2-19: Values of the parameters to adopt in Equation 2-11 and Equation 2-12, calibrated on experimental data (modified from Croce et al., 2014).

SOIL		ASTM D2487 CLASSIFICATION	D_{ref} (m)	β	δ	α_E (single fluid)	α_E (double and triple fluid)
Coarse grained	Without fine	Gravels and sands with <5% fines (GW-GP-SW-SP)	1.00	0.2	-0.25	1	6
	With fine	Gravels and sands with >5% fines (GM-GC-SM-SC)	0.80				
Fine grained		Silts, clay and organic soils (CL-ML-OL-CH-MH-OH-Pt)	0.50				

2.11.3 Limitations and drawbacks

Jet grouting treatment is difficult in plastic soils.

One of the most important limitations of JG is related to the uncertainties on the dimensions and properties of the treated elements.

Defects in the treated soil can be caused by insufficient overlapping of the elements, by a vertical deviation of the elements or inhomogeneous soil conditions.

The properties of soilcrete are affected by the presence of organics, low pH groundwater or groundwater flows. Since the injected elements have not adequate resistance to horizontal actions, some reinforcements can be introduced to improve it.

The treatment can cause diffusion of contaminants, noise and vibration.

The treatment produces spoil from the injected grout and the eroded soils rising to the ground surface. Since the spoil is a waste, its production should be minimised. Moreover, the spoil production is useful to ensure effective treatment, without clogging.

2.11.4 Treatment parameters

The jet grouting treatment parameters can be grouped in *geometrical characteristics* of the mechanical device, *kinematic variables* for the movement of the string and *characteristics of the injected fluids*. The treatment parameters can also be subdivided into *fundamental* and *derived parameters*.

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In particular, the fundamental parameters are:

- *Geometrical*: number of nozzles, nozzle diameter;
- *Kinematic*: time interval per step, rotational velocity, lifting step;
- *Injected fluids*: W-C ratio by weight, fluid pressure, fluid flow rate.

The derived parameters are:

- average lifting speed of the monitor;
- monitor rotation for each lifting step;
- injected grout volume per treatment unit length;
- mass of injected cement per treatment unit length.

Although the treatment parameters are selected after performing preliminary field tests, typical values are reported in Table 2-20.

Table 2-20: List of correlations available in literature (modified from Croce et al., 2014).

TREATMENT PARAMETER	SYMBOL	UNIT	SYSTEM		
			SINGLE FLUID	DOUBLE FLUID	TRIPLE FLUID
Lifting step	Δs	mm	40÷50	40÷80	40÷100
Average lifting speed	v_r	Mm/s	4÷10	1÷8	0.5÷5
Rotational velocity	ω	rpm	5÷40	3÷30	1÷40
Nozzle diameter	D	mm	2÷8	2÷8	2÷8
Number of nozzles	M	-	1÷2	1÷2	1÷2
Grout pressure	p_g	MPa	30÷55	20÷40	2÷10
Air pressure	p_a	MPa	-	0.5÷2.0	0.5÷2.0
Water pressure	p_w	MPa	-	-	20÷55
Grout flow rate	Q_g	L/s	2÷10	2÷10	2÷5
Air flow rate	Q_a	L/s	-	200÷300	200÷300
Water flow rate	Q_w	L/s	-	-	0.5÷2.5
W-C ratio by weight	W/C	-	0.60÷1.25	0.60÷1.25	0.40÷1.00

3. DESIGN OF GROUND IMPROVEMENT FOR LIQUEFACTION MITIGATION

3.1 Objective

The main scope of ground improvement for liquefaction mitigation is to inhibit the onset or to limit the effects of the phenomenon. As shown in the previous chapter, there are different techniques to achieve these goals whose principles can be briefly summarised in reducing the deformation and contractive tendency of the soil induced by cyclic loading, in preventing the pore pressures build-up or in enhancing the exhaust capacity with drainage. Whatever the technique, design procedures should be formulated to encompass the fixed requirement with the most efficient, reliable and cost-effective way. Design of ground improvement for liquefaction mitigation, like for any other application, should be developed following a sequence of steps, going from site characterization to cost assessment, passing through the verification of serviceability and ultimate limit states. However, with respect to conventional geotechnical structures, it is commonly recognised that the technological aspects play a more relevant role. In common practice, the designer provides only simple indication on the ground improvement project, which is then specified at the construction stage, often following some sort of trial-and-error procedure. The reason for such simplistic procedure lies in the widespread belief that the effects of ground improvement cannot be forecasted at the design stage, and so, it may seem more realistic to rely only on a purely empirical approach. However, even if this lack of knowledge is still true for many techniques, the experience gained all over the world in more than 30 years of practice and the recent research activity on this topic have now provided more reliable design tools. Clearly, there are additional steps that must be added to the usual design process, strictly related to the quantification of the technological effects:

- the choice of the ground improvement methodology;
- the quantification of treatment parameters;
- the prediction of the volume and the mechanical properties of the treated soil;
- the analyses of possible undesired collateral effects on the surrounding constructions and on the environment.

Because technology plays a relevant role in the success of a ground improvement project, the designer should be aware of the uncertainty and risk connected with all the previously listed steps, not blindly leaving them to the specialist contractor only. The prediction of the performance of the improved soil, considering ultimate and serviceability limit states, must thus be seen as a whole with the definition of the set of operations necessary to obtain the desired result in terms of dimensions and properties. In this sense, a winning design strategy must conjugate theoretical and technological knowledge to conceive practically feasible solutions able to provide safety, functionality and economy.

Suggested design procedures and possible alternative approaches, methods of analysis and calculation examples are reported in this chapter for the most common applications, showing that the effects of technology on the mechanical performance can be rationally considered and accounted for.

3.2 Strategy for the assessment of liquefaction and application of ground improvement

Traditionally (e.g. EN1998) the assessment of liquefaction is carried out with a three-level strategy that implies the appraisal of susceptibility, triggering and effects. Once susceptibility of the subsoil is ensured, the assessment of triggering is performed in free field conditions checking that safety factors are larger than a minimum value (e.g. 1.25) at any depth. In opposite case, the cumulated effect at the ground level is quantified checking that the assumed indicator (e.g. LPI from Iwasaki et al., 1978) is lower than a prescribed value. Despite its limitation (e.g. Cubrinovski & van Ballegooy, 2017), this analysis can be considered satisfactory for a preliminary assessment of a large variety of situations. However, it must not be forgotten that this calculation is performed in one-dimensional conditions and without the presence of superstructures at the ground level.

When a superstructure is present at the ground level, the shear stress induced in the subsoil by the dead load determines the condition for the development of shear and coupled volumetric strains during the earthquake summed to the increase of pore pressures. This phenomenon occurs even for mean effective stresses not approaching zero value ($r_u>0$ where $r_u=\Delta u/\sigma'_{vo}$). The immediate consequences are settlements ranging from minor to intermediate values depending on the local conditions (shaking intensity, soil state, applied load etc.), eventually reaching the complete failure of the foundation.

In such a case, the assessment should be carried out with a different strategy compared with the previously recalled analysis, i.e. considering the performance of the foundation in terms of limit states. Methods considering the interaction between soil and superstructure have been developed in the last years (e.g Karamitros et al., 2013; Bray and Macedo, 2017; Bullock et al., 2018) to compute settlements or to analyse the ultimate limit load of a foundation as function of r_u.

The complete strategy for the assessment of liquefaction effects on a foundation is outlined in Table 3-1, together with the specific calculation to be performed and the uncertainty connected with the analysis.

This assessment is the preliminary step for the design of ground improvement mitigation. In fact, as recalled in the standards (prEN 1997-1:2004) *"The effectiveness of the ground improvement shall be checked against the acceptance criteria by determining the induced changes in the appropriate ground properties".*

The above sentence state that one (or more) soil property, originally weak with reference to the seismic performance of the foundation, should be identified and ground improvement should be able to modify it up to a level able to fulfil the design requirements (Limit States). The strategy traced in Table 3-1 is conjugated in the next paragraphs defining the calculation methods necessary for the different steps. From this analysis, the soil response against liquefaction and the modification given by ground improvement are characterised making reference to site and laboratory tests. Finally, typical results of some ground improvement methodologies are given referring to this schematization.

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Table 3-1: Strategies for the assessment of liquefaction.

CASE	Liquefaction check	Pore pressure ratio	Construction performance CHECK	CALCULATION	ISSUES
1	$FS(z)<FS_{min}$ at some z	$r_{u,ff}(z)=1$ at some z	-	Calculate integral ground response (LPI, LSN, …)	What is a reliable integral parameter?
2	$FS(z)\geq FS_{min}$ at any $z\leq z_{max}$	$r_{u,ff}<1$ at any $z\leq z_{max}$	$ULS(r_{u,max})$	• Calculate bearing capacity, $f(r_u)$	• Estimate of $r_{u,ff}$ • Estimate of $r_{u,str}$ • Rational bearing cap. formulas available • Lack of experimental verification
			$SLS(r_{u,max})$	• Calculate settlement w, $f(r_u)$	• Estimate of r_u • Difficulty in estimating undrained and drained components of w

3.3 Assessment of free field liquefaction with semi-empirical methods

For a given soil profile, the triggering of liquefaction at each depth can be evaluated by applying simplified methods. Traditionally, a stress based approach is adopted where assessment is performed introducing a safety factor (FSL) that compares the cyclic stress ratio τ/σ'_v induced by earthquake (CSR) with the cyclic stress ratio producing liquefaction (cyclic resistance ratio denoted with CRR) dependent on the soil (Equation 3-1).

$$FSL = CRR/CSR \qquad \text{Equation 3-1}$$

A simplified method to estimate the CSR profile was developed by Seed and Idriss (1971) based on the maximum ground surface acceleration (a_{max}) at the site (Equation 3-2).

$$CSR\ (z) = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{vo}(z)}{\sigma'_{vo}(z)}\right) \cdot r_d(z) \qquad \text{Equation 3-2}$$

where:
σ_{v0}, σ'_{v0} = vertical total and effective stress at depth z
a_{max}/g = maximum horizontal acceleration (as a fraction of gravity) at the ground surface
r_d = shear stress reduction factor that accounts for the dynamic response of the soil profile

The 0.65 factor given in Equation 3-2 was originally proposed to relate the acceleration time history from an irregular earthquake to the number of loading cycles from uniform cyclic loading. Although this value is somewhat arbitrary and unnecessary once MSF is introduced, 0.65 is still a standard coming from historical precedent.

The depth-dependent shear stress reduction coefficient, r_d, accounts for the non-rigid response of the soil deposit (characterized in the small strain regime by the shear wave velocity [V_s] profile at the site) as well as for the characteristics of the earthquake waves traveling through the soil profile. Seed & Idriss (1971) initially proposed a relationship between r_d and depth developed from a limited number of dynamic response analyses for a range of generic site conditions. Using additional site response analyses, Idriss (1999) modified this relation introducing the effect of magnitude. According to this formula, the shear stress reduction coefficient r_d is calculated using two functions of the depth (namely α(z) and β(z)), and of the earthquake magnitude (M):

$$r_d = \exp\left[\alpha(z) + \beta(z) \cdot M\right] \qquad \text{Equation 3-3}$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \qquad \text{Equation 3-4}$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \qquad \text{Equation 3-5}$$

The Idriss (1999) relationship is used to develop the triggering relationships of Idriss & Boulanger (2008) and Boulanger & Idriss (2014), Figure 3-1.

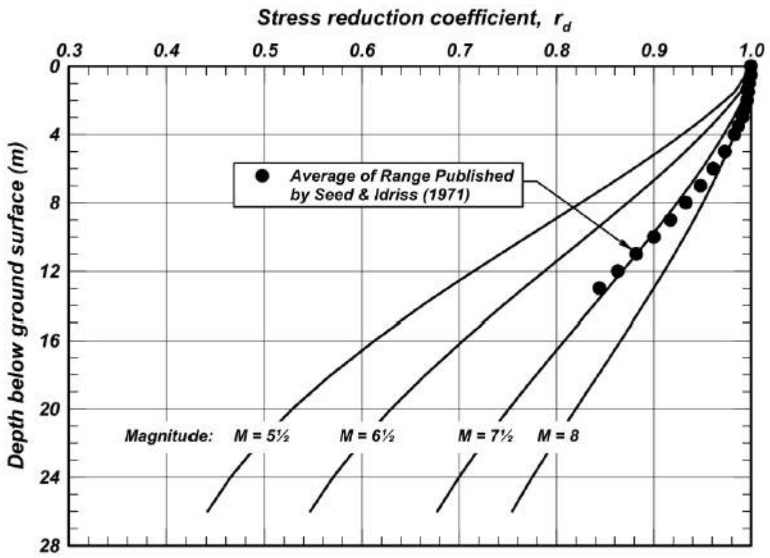

Figure 3-1: Shear stress reduction factor, r_d, relationships (Boulanger & Idriss, 2014).

For a given cyclic stress ratio, initiation of liquefaction depends on the number of loading cycles. In the semi-empirical assessment, this aspect is tackled associating seismic loading with a number of equivalent loading cycles. Earthquake magnitude is used as a proxy for the number of loading cycles because the duration of shaking and the associated number of loading cycles correlate with earthquake magnitude. The CSR is then adjusted using a magnitude scaling factor (MSF) and compute the equivalent CSR for a reference magnitude value M = 7.5 (Equation 3-6).

$$CSR_{M=7.5}\,(z) = CSR_{M=m}\,(z) \cdot \frac{1}{\mathrm{MSF}} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{vo}(z)}{\sigma'_{vo}(z)}\right) \cdot r_d(z) \cdot \frac{1}{MSF} \qquad \text{Equation 3-6}$$

The magnitude scaling factor (MSF) accounts for duration effects (i.e., number and relative amplitudes of loading cycles) on the triggering of liquefaction. Several formulations (Andrus & Stokoe, 1997; Idriss & Boulanger, 2008) have been proposed to evaluate the Magnitude Scaling Factor, after the first developed by Seed & Idriss (1982) (see Figure 3-2).

For instance, the MSF for sands used by Boulanger & Idriss (2014) was developed by Idriss (1999), who derived the following relationship (Equation 3-7).

$$MSF = 6.9 \cdot exp\left(\frac{-M}{4}\right) - 0.058 \leq 1.8 \qquad \text{Equation 3-7}$$

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An upper limit for the MSF is assigned to very-small-magnitude earthquakes for which a single peak stress can dominate the entire time series. The value of 1.8 is obtained by considering the time series of stress induced by a small magnitude earthquake to be dominated by single pulse of stress (i.e., ½ to 1 full cycle, depending on its symmetry), with all other stress cycles being sufficiently small to be neglected.

Therefore, CSR evaluation requires estimates of PGA, M_w, and r_d; since the required PGA is at the ground surface, it must account for the effects of the near-surface soil conditions on ground shaking.

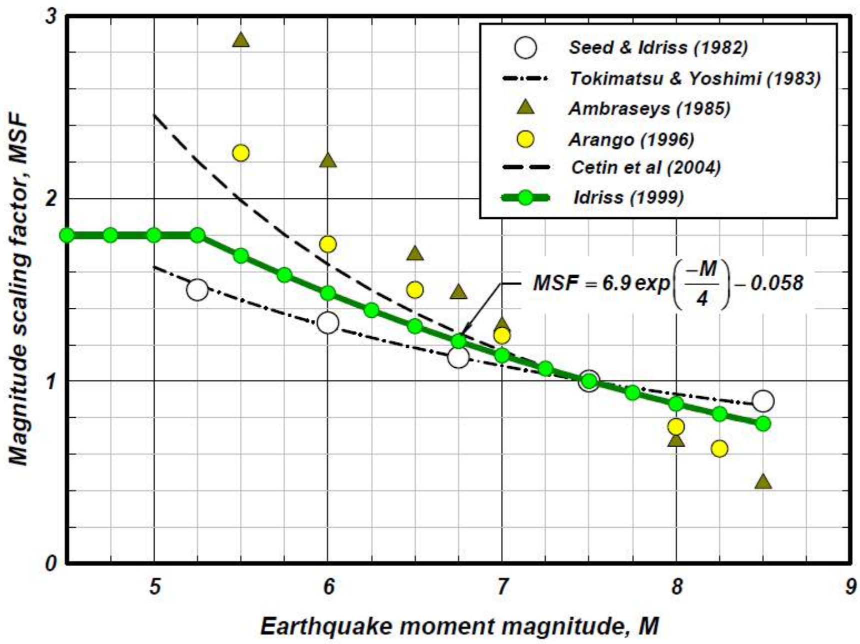

Figure 3-2: Magnitude scaling factor.

Several empirical procedures (Robertson, 1998; Boulanger & Idriss, 2014) were proposed to evaluate the CRR starting from geotechnical and geophysical in-situ tests (CPT, SPT and V_s profile). In particular, Boulanger and Idriss (2014) provide an empirical formulation of the Cyclic Resistance Ratio based on the survey of liquefaction and the results of the most common penetrometer tests (CPT and SPT), while Andrus and Stokoe (2000) propose a method to evaluate the CRR starting from the shear wave velocity (V_s) profiles.

With the above methods, the calculation of CRR requires geotechnical and geophysical profiles, with measurement of the SPT blow count, CPT tip resistance and sleeve friction, V_s as a function of depth and at multiple locations across the site. Correction factors need generally to be applied to refer to standard values and remove the effects of local experimental conditions; for instance, the measured SPT blow count need to be corrected to refer to a standard stress and energy resistance $(N_1)_{60}$ based on the test setup. Furthermore, SPT blow counts recorded in hollow stem auger borings below the water table are particularly susceptible to error due to soil disturbance and may result in abnormally low blow count values.

Measurement with the different methods rarely reach the same level of quality. For instance, SPT provides measurements at spaced intervals (often 1.5 m, in any case not less than the length of the split spoon

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sampler, 0.45 m) that limits the ability to use SPT measurements to identify thinner layers and obtain detailed variations of the soil profile. On the contrary, CPT provides continuous measurements along depth, that represent a very powerful mean to characterize thinner layers and detailed variations within strata. Pore-pressure data from piezocone penetration testing (CPTU) can provide additional information, both qualitative (e.g., whether soil is dilatant or not) and quantitative (e.g., the steady-state porewater pressure).

Depending on the method of measurement, V_s may be used to identify thin layers and variations within strata, even if it has not the detail and the resolution of the CPT.

Liquefaction triggering analyses require the most rigorous soil type characterization. Since CPT and V_s methods do not provide a direct view of the soil type, additional boring and sampling, or sampling using a special tool adapted for use with CPT rigs, should accompany the in-situ test to identify the soil type. When using liquefaction triggering methods that require V_s (e.g., Andrus and Stokoe, 2000; Cetin and Seed, 2004), values should be measured directly and not estimated from correlations with SPT or CPT tests.

3.3.1 CPT-Based liquefaction triggering

One of the most popular CPT-based procedure to evaluate the Safety Factor against liquefaction at each depth of a soil profile, summarized in Figure 3-3, is provided by Boulanger and Idriss (2014). The authors calculate the Cyclic Resistance Ratio (CRR) from the measured CPT tip resistance, q_c, the CPT sleeve friction, f_s, and the effective vertical stress, σ'_v, in the soil. These are used to estimate an overburden correction factor, CN, and correct the tip resistance to account for the overburden stress, q_{c1}. The normalized overburden stress, q_{c1N}, is q_{c1} divided by the atmospheric pressure (pa≈100 kPa). During the iteration (usually about 3 cycles are sufficient), q_{c1} is always based on the measured tip resistance, q_c, while CN is based on the iteratively updated value for qc_{1N}. Another correction, seen as an additional tip resistance Δq_{c1N}, is applied to consider the possible presence of finer material (FC). For flat ground or uniform surcharge, no correction is applied for the effects of an initial static shear stress ratio (K_α=1).

The empirical correlations defined by Robertson (2015) is used to characterize the soil behaviour type (SBT) and evaluate the fraction of fines, FC.

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$$q_{c1} = C_N * qc$$
$$q_{c1N} = q_{c1}/Pa$$
$$C_N = \left(\frac{Pa}{\sigma'_v}\right)^m \leq 1.7$$
$$m = 1.338 - 0.249 * (q_{c1N})^{0.264}$$
Pa = atmospheric pressure having same units as qc

Fine Content evaluation (FC)

$$\boldsymbol{Fc = 80 * (Ic + C_{FC}) - 137}$$
Where: Ic is evaluated according to Robertson, 1998
$$\boldsymbol{C_{FC} = -0.29, 0, +0.29}$$
(where CFC is a fitting parameter that can be adjusted based on site-specific data when available)

$$\boldsymbol{\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) exp\left(1.63 - \frac{9.7}{Fc+2} - \left(\frac{15.7}{Fc+2}\right)^2\right)}$$
$$\boldsymbol{(q_{c1N})cs = q_{c1N} + \Delta q_{c1N}}$$

$$\boldsymbol{CRR = exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.8\right)}$$

$$CSR = 0.65 * \left(\frac{a_{max}}{g}\right) * \left(\frac{\sigma_v}{\sigma'_v}\right) * rd * \left(\frac{1}{MSF}\right) * \left(\frac{1}{K\sigma}\right)$$

$$ln(rd) = \alpha + \beta M$$
$$\boldsymbol{\alpha = -1.012 - 1.126 sen\left(\frac{z}{11.73} + 5.133\right)}$$
$$\boldsymbol{\beta = 0.106 - 0.118 sen\left(\frac{z}{11.28} + 5.142\right)}$$

$$K\sigma = 1 - C\sigma \ln\left(\frac{\sigma'_v}{pa}\right) \leq 1.1$$
$$C\sigma = \frac{1}{37.3 - 8.27 * qc1N_{cs}^{0.264}} \leq 0.3$$

$$MSF = 6.9 \exp\left(-\frac{M}{4}\right) - 0.058$$
Idriss and Boulanger (2008)

$$FSL = \frac{CRR}{CSR}$$

Figure 3-3: Flowchart of the Boulanger and Idriss (2014) CPT-based procedure.

3.3.2 SPT-Based liquefaction triggering

Boulanger & Idriss (2014) also propose a SPT-based procedure to evaluate the CRR (Figure 3-4) starting from the number of blows $(N_1)_{60}$, normalized with respect to the atmospheric pressure Pa and increased to account for the fine content. In this case, the soil behaviour type index I_c can be evaluated with numerous empirical correlations between in-situ tests and geotechnical parameters.

$$(N_1)_{60cs} = CN \cdot CE \cdot CB \cdot CR \cdot CS \cdot N + \Delta(N_1)_{60} \qquad \text{Equation 3-8}$$

where:

N is the recorded blow count

CN is the correction factor to adjust the blow count to a reference vertical effective stress of 100 kPa

CE is a correction factor for the kinetic energy of the hammer (i.e. hammer weight and height of fall)

CB is a correction factor for the borehole diameter

CR is a rod length correction factor

CS is a correction factor for the configuration of the SPT sampler

$\Delta(N_1)_{60}$ is an artificial resistance added to consider the influence of fines content where it is present

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$$(N_1)_{60} = C_N * N_{60}$$
$$C_N = \left(\frac{Pa}{\sigma'_v}\right)^m \leq 1.7$$
$$m = 0.784 - 0.0768 * \sqrt{(N_1)_{60cs}}$$

Fine Content evaluation (FC)

$$Fc = 80 * (Ic + C_{FC}) - 137$$
$$C_{FC} = -0.29, 0, +0.29$$

CFC is a fitting parameter, while Ic is evaluated according to Mayne (2006)

$$Ic = -0.7174 * \ln\left(\frac{Vs^2}{9.81 * z}\right) + 6.3211$$
$$Vs = 100.59 * N_{SPT}^{0.302}$$

Modified after Palmer and Stuart (1957)

$$\Delta(N_1)_{60} = exp\left(1.63 + \frac{9.7}{FC + 0.01} - \left(\frac{15.7}{FC + 0.01}\right)^2\right)$$
$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

$$CRR = exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right)$$

$$CSR = 0.65 * \left(\frac{a_{max}}{g}\right) * \left(\frac{\sigma_v}{\sigma'_v}\right) * rd * \left(\frac{1}{MSF}\right) * \left(\frac{1}{K\sigma}\right)$$

$$ln(rd) = \alpha + \beta M$$
$$\alpha = -1.012 - 1.126 sen\left(\frac{z}{11.73} + 5.133\right)$$
$$\beta = 0.106 - 0.118 sen\left(\frac{z}{11.28} + 5.142\right)$$

$$K\sigma = 1 - C\sigma \ln\left(\frac{\sigma'_v}{pa}\right) \leq 1.1$$
$$C\sigma = \frac{1}{18.9 - 2.55 * (N_1)_{60cs}^{0.5}} \leq 0.3$$

$$MSF = 6.9 \exp\left(-\frac{M}{4}\right) - 0.058$$

Idriss and Boulanger (2008)

$$FSL = \frac{CRR}{CSR}$$

Figure 3-4: Flowchart of the Boulanger & Idriss (2014) SPT-based procedure for liquefaction triggering analysis.

3.3.3 Vs-based liquefaction triggering

Shear wave velocity (V_s) is another soil property used to characterize the response against liquefaction. V_s refers to the propagation speed of shear waves through the subsoil. It depends significantly on the soil density, the directions of wave propagation and particle motion, and the effective stresses in those two directions. V_s, by convention, refers to very small strain amplitudes being related to the shear modulus of the soil at small strain (G_o) and the mass density of the soil (ρ) by the equation:

$$V_s = \sqrt{\frac{G_o}{\rho}} \qquad \textbf{Equation 3-9}$$

Compared with the previous examples, V_s measurement is generally less invasive. The latter capability can be beneficial if soil profiles contain inclusions (i.e., gravel or cobble inclusions) that can make testing difficult or even prohibit SPTs and CPTs. There are many V_s measurement techniques, including downhole measurements (ASTM International, 2014a), cross-hole measurements (ASTM International, 2014b), suspension logging (Nigbor & Imai, 1994), and non-invasive methods (Stokoe & Santamarina, 2000). Because non-invasive V_s tests do not provide soil samples, however, some drilling and sampling may still be required as part of a subsurface investigation.

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Andrus & Stokoe (2000) define the following relation to calculate CRR from the shear-wave velocity, V_s:

$$CRR = \left[0.022\left(\frac{V_{s1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right)\right] \qquad \text{Equation 3-10}$$

in which:

V_{s1} is the stress-corrected shear wave velocity; V^*_{s1} is the limiting upper value of V_{s1} for cyclic liquefaction occurrence, which varies between 200-215 m/s depending on the fines content of the soil.

$$V_{S1} = Vs\left(\frac{pa}{\sigma'_v}\right)^{0.25}$$

$$V_{S1,csa1} = \frac{V_{S1}}{Ka1}$$

Pa = atmospheric pressure (kPa); σ'v effective vertical stress (kPa).

ka1 is the correction factor accounting for the age of the deposit

Time (years)	Soil aging factor (Ka1)
1	1.09
10	1.01
100	0.94
1 000	0.88
10 000	0.83
100 000	0.78

$$CRR = 0.022 * \left(\frac{V_{S1,csa1}}{100}\right)^2 + 2.8 * \left(\frac{1}{V_{S1}^* - V_{S1,csa1}} - \frac{1}{V_{S1}^*}\right)$$

V_{S1}^* is the limiting upper value of $V_{S1,csa1}$ for cyclic liquefaction occurrence, which varies between 200-215 m/s depending on the fines content of the soil.

$$CSR = 0.65 * \left(\frac{a_{max}}{g}\right) * \left(\frac{\sigma_v}{\sigma'_v}\right) * rd * \left(\frac{1}{MSF}\right) * \left(\frac{1}{K\sigma}\right)$$

$$rd = 1 - 0.00765z \qquad if\ z < 9.2m$$

$$rd = 1.174 - 0.0267z \qquad if\ z \geq 9.2m$$

Liao and Whitman (1986)

$$K\sigma = 1 - C\sigma \ln\left(\frac{\sigma'_v}{pa}\right) \leq 1.1 \qquad C\sigma = \frac{1}{18.9 - 3.1 * \left(\frac{Vs_1}{100}\right)^{1.976}} \leq 0.3$$

$$MSF = \left(\frac{Mw}{7.5}\right)^{-2.56}$$

Andrus and Stokoe (1997)

$$FSL = 1.4 * \frac{CRR}{CSR}$$

Juang et al. (2005)

Figure 3-5: Flowchart of the Andrus & Stokoe (2000) procedure for liquefaction triggering evaluation.

The procedure proposed by Mayne (2006) can be then applied to evaluate the soil behaviour type index, I_c:

$$I_c = -0.7174 \cdot ln\left[\frac{V_s^2}{(9.81 \cdot z)}\right] + 6.3211 \qquad \text{Equation 3-11}$$

The fine content FC can be taken into account applying the following correlation (Robertson and Fear, 1995):

$$FC(\%) = 42.4179 \cdot I_c - 54.8574 \qquad \text{Equation 3-12}$$

About the Factor of Safety, Juang et al. (2005) found that the traditional FSL is conservative for calculating CRR, resulting in lower factors of safety and over-prediction of liquefaction occurrence. To account for this, they introduce a multiplication factor of 1.4 to obtain a more realistic estimate of the factor of safety.

After an 11-years period of V_s site data collection and the development of probabilistic correlations for seismic liquefaction occurrence, new correlations for probabilistic/deterministic assessment of liquefaction potential from shear wave velocity were proposed by Kayen et al. (2013).

Data coming from 301 liquefaction field case histories in China, Taiwan, Japan, Greece and the United States were merged to previously published case histories to build a global catalogue of 422 case histories of V_s liquefaction performance. Then, after Bayesian regression and structural reliability methods a probabilistic treatment of the Vs catalogue for performance-based engineering applications was developed.

3.3.4 Liquefaction triggering based on laboratory tests

An alternative strategy, totally different from the previous ones, is to estimate the soil cyclic resistance with laboratory tests. In laboratory testing, liquefaction is usually studied through undrained cyclic triaxial or simple shear tests. Even though cyclic simple shear tests apply more realistic stress paths, cyclic triaxial tests are more popular and widely used to assess soil liquefaction potential. The results are usually interpreted in the CSR vs. N plane, being CSR the *Cyclic Stress Ratio* and N the applied number of constant amplitude stress cycles. N_{liq} is the value of N needed to reach liquefaction for a given value of CSR. For $N=N_{liq}$, the applied cyclic stress ratio represents the *Cyclic Resistance Ratio* CRR. The locus (N_{liq}:CRR) identifies the *Cyclic Resistance Curve*. Conventionally, it is assumed that liquefaction is triggered at 5% double strain amplitude (strain criterion) or at r_u=0.90, being $r_u=\Delta u/\sigma'_{v0}$ (stress criterion), where Δu is the excess of pore air pressure for the specimen with positive suction measurement, otherwise it is the excess of pore water pressure (Wang et al., 2016; Mele et al., 2018). In triaxial tests, CSR is defined as:

$$\mathrm{CSR} = \frac{q_d}{2 \cdot \sigma'_c} \qquad \text{Equation 3-13}$$

where q_d is the cyclic deviatoric stress and σ'_c is the confining effective stress.

The CRR(N) relationship can be simply expressed with a power function (Idriss & Boulanger, 2008) as:

$$\mathrm{CRR} = \mathrm{a} \cdot N^{-b} \qquad \text{Equation 3-14}$$

where the coefficient a and the exponent b depend on soil physical and mechanical properties (via q_{c1Ncs}) and on σ'_v:

$$\mathrm{a,b} = \mathrm{f}\left(q_{c1Ncs,}\ \sigma'_v\right) \qquad \text{Equation 3-15}$$

Typical N_{liq}-CRR curves are reported in Figure 3-6. The results refer to soil having different relative density, where this latter property has been expressed in terms of normalized CPT tip resistance $q_{c1N.}$ The dependency of the curve on this factor has been defined according to Boulanger & Idriss (2014) that give a different steepness for the different curves. Additionally, the relation between loading cycles and earthquake magnitude provided by Idriss, 1999 is also reported in the plot. The values of CRR corresponding to N=15 (M=7.5) are the reference values typically expressed in triggering estimates from in situ tests.

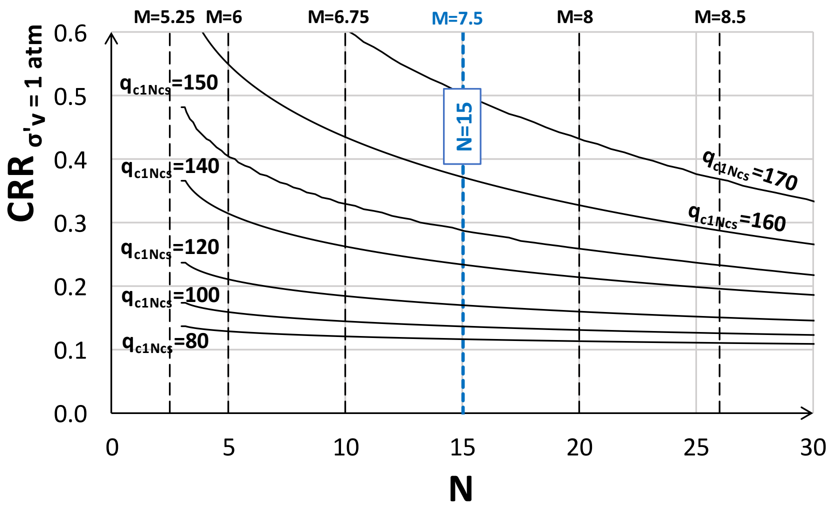

Figure 3-6: Typical N_{liq}-CRR plot (Boulanger & Idriss, 2014).

Another advantage of the laboratory tests consists in the possibility of monitoring the pore pressure development, and the consequent effective stress reduction, along with loading cycles. The seismically induced pore pressure increments Δu, even if not causing liquefaction, reduce soil stiffness and shear strength. These reductions induce settlements and reduce the bearing capacity and the safety margin for existing structures (e.g., Cascone & Bouckovalas, 1998; Karamitros et al. 2013), possibly triggering unforeseen limit states. Quantification of this additional risk requires the estimate of Δu_{ff} in free field conditions, and the calculation of the corresponding values underneath the structure of interest. The latter issue has been effectively addressed by Karamitros et al. (2013), while the way of quantifying Δu_{ff} without carrying out complex coupled analyses is still an open issue. Few indications exist in literature, all attempting to correlate Δu_{ff} to $FS_{liq,ff}$. Iwasaki et al. (1984) proposed the empirical correlation:

$$r_{u,ff} = FS_{liq,ff}^{-1/b} \qquad \text{Equation 3-16}$$

where $r_{u,ff}$ is the pore pressure ratio (defined as the ratio between Δu_{ff} and the initial effective overburden stress, σ'_{v0}) and b is one of the two parameters needed to define the cyclic resistance curve of a soil, as explained in the following.

Equation 3-14 is of limited practical interest if values of b are not given as a function of soil intrinsic and state properties. This issue will be dealt with in more detail in Marcuson et al. (1990) have collected laboratory data on gravels and sands and have produced a qualitative chart to estimate $r_{u,ff}(FS_{liq,ff})$ (Figure 3-7) depending on soil grading. Again, this chart is of limited interest, as it indicates wide ranges of $r_{u,ff}$ for a given value of $FS_{liq,ff}$.

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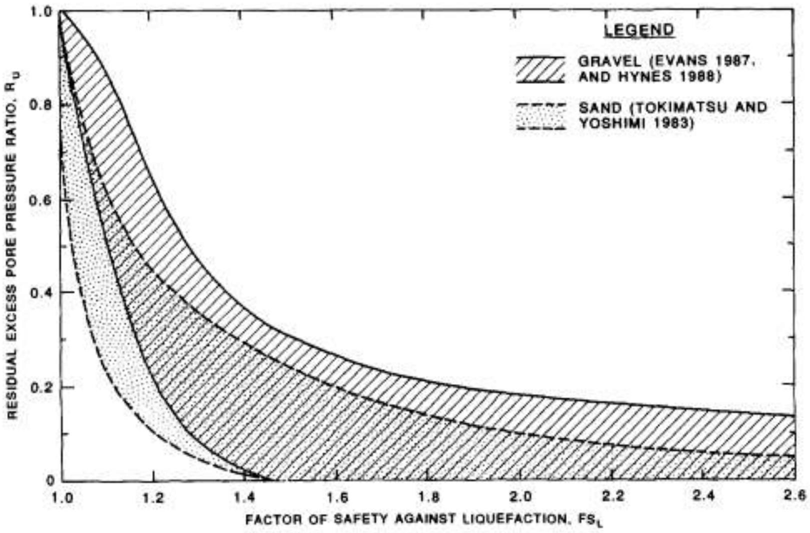

Figure 3-7: Experimental relationships between r_{uff} and $FS_{liq,ff}$ (after Marcuson et al., 1990).

Now there is the need to link r_u to the loading history activated by the earthquake. In principle, this is not an easy task, as the irregular seismic shaking leads to an irregular pore pressure build-up history. However, a simplification can be introduced considering a regular Δu build-up history (Figure 3-8), similarly to what has been proposed by Seed & Idriss (1971) to calculate CRR considering instead of the irregular shear stress history caused by the earthquake an equivalent constant amplitude cyclic shear stress (Equation 3-14).

The analytical relationship $r_{u,ff}(N/N_L)$ proposed by Booker et al. (1976) can then be adopted to relate $r_{u,ff}$ to the number of lading cycles, where N_L is a reference parameter:

$$r_{u,ff} = \frac{2}{\pi} \, arcsin \left(\frac{N}{N_L}\right)^{1/2\beta} \qquad with \; r_{u,ff} \leq 1$$ **Equation 3-17**

in which the parameter β depends on soil physical and mechanical properties.

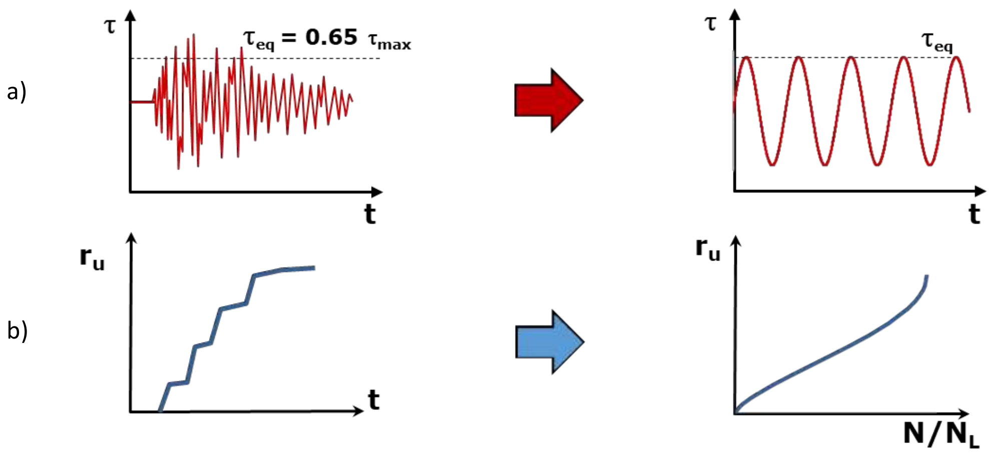

Figure 3-8: Conceptual conversion from an irregular to a regular loading history of (a) shear stress (Seed & Idriss, 1971) and (b) pore pressure ratio (Chiaradonna & Flora, 2019).

Examples of r_u development curves for different N_L values are reported in Figure 3-9.

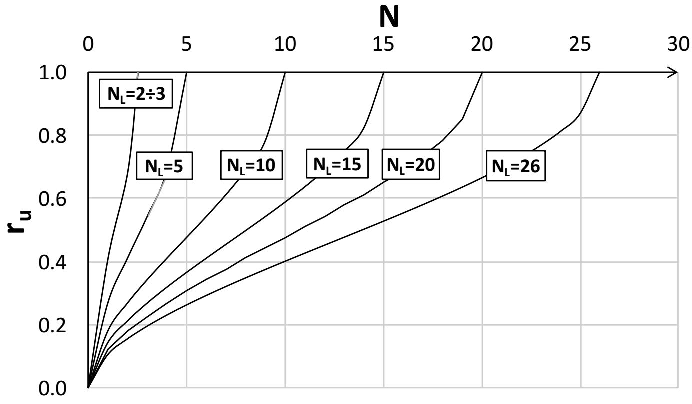

Figure 3-9: Typical N-r_u plot (Booker, 1976).

Combining the above relations, Chiaradonna & Flora (2019) propose the following charts to compute $r_{u,ff}$ as function of the safety factor FS_{ff}.

It must necessary be recalled that the reliability of prediction relies on the quality of retrieved samples. As well known, retrieving undisturbed samples from cohesionless soil is problematic (e.g. Yoshimi et al. 1994) and this is the reason why in situ tests are normally chosen to estimate the response of these soils. Nowadays, innovative sampling techniques, like the gel pusher sampler are being proposed to by-pass this problem in an affordable manner. The gel-push sampling methodology was developed by Kiso-Jiban Consultants (Japan) for a cost-effective retrieval of undisturbed specimens of silty and clean sands, comparatively with the very expensive ground freezing technique. It is believed that the main source of disturbance associated with conventional downhole tools is due to friction which is mobilised on the sides of the soil sample as it enters the core-liner barrel. Gel-push sampling removes this friction by coating the outer surface of the soil (as it enters the sampler) with a low-friction polymer gel. The technique has been tested by several researchers in Japan, Taiwan, Poland, Bangladesh and New Zealand (e.g. Lee et al., 2012; Taylor et al., 2012; Jamiolkowski, 2014). The procedures for sampling are still evolving and it is expected that as more experience is gained, refinements will be made to the way that sampling is carried out which will improve the performance of the gel-push samplers. However, trials carried out in New Zealand by Stringer et al. (2016) have shown that there will be a range of soils for which these samplers can recover high quality soil specimens. In the trials, sampling was successful in silts and silty sands with low values of cone penetration resistance (<5MPa). On the contrary, sampling of micaceous silts was unsuccessful due to the large amount of swelling which occurred after the soil was captured in the core-liner barrel.

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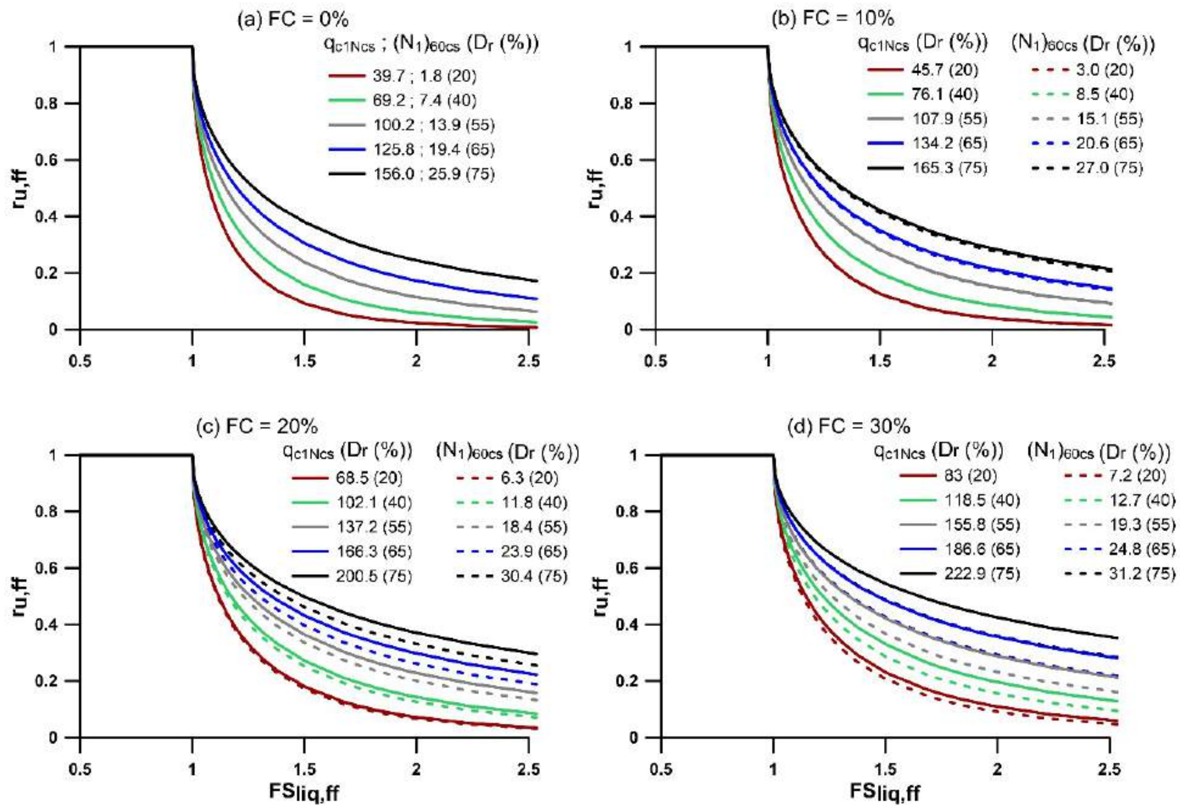

Figure 3-10: Charts with the proposed relationship between the free field pore pressure ratio, $r_{u,ff}$ and the free field liquefaction safety factor, $FS_{liq,ff}$ for different fine contents: (a) FC=0%, (b) FC=10%, (c) FC=20% and (d) FC=30%.

3.4 Assessment of liquefaction for foundations

The assessment of foundation performance is traditionally achieved comparing the design condition with two different situations, the extreme Ultimate Limit State characterised by the failure load, and the Serviceability Limit State generally characterised with a limit movement (settlement, titling or distortion). The shear strength degradation of the foundation soil induced by the pore pressure build-up may result in post-shaking static bearing-capacity failure and in excessive settlements accumulation, even when liquefaction has not completely developed. The situation may be more complex considering other factors influencing the foundation response. For instance, there is ample field evidence that a sufficiently thick and shear resistant non-liquefiable soil crust, between the foundation and the liquefiable soil, may effectively mitigate the detrimental effects of liquefaction and lead to adequate foundation safety and satisfactory performance (Ishihara et al., 1993; Acacio et al., 2001).

The first shocking evidence of liquefaction-induced seismic settlement accumulation and bearing-capacity failure of shallow foundations dates back to 1964 in Niigata (Japan) where approximately 340 RC buildings settled or tilted with movements settlements reaching 3.8 m (Yoshimi & Tokimatsu, 1977). In that area the depth of liquefaction exceeded 10 m.

Other remarkable examples were observed during the 1990 Luzon earthquake, in the city of Dagupan, where buildings settled of 0.50 m on average and more than half tilted (Ishihara et al., 1993; Tokimatsu et al., 1994) due to liquefaction occurring below a 2.0- to 8.0-m-thick crust, (Acacio et al., 2001) or during the Kocaeli earthquake of August 17, 1999, which caused significant damage to the Izmit Bay area in Northwestern Turkey, caused many buildings in Adapazari City to collapse, settle, or tilt, because of subsoil liquefaction. Here notwithstanding a large variation in local soil profiles (Bray et al., 2004), a general relation between settlements and buildings storeys was noted (Sancio et al., 2002; Yoshida et al., 2001).

The liquefaction performance of shallow foundations has also been modelled in centrifuge and large-scale shaking table experiments (Liu and Dobry, 1997; Kawasaki et al., 1998; Adalier et al., 2003; Coelho et al., 2004; Dashti et al., 2010). For instance, results from experiments determined that settlements primarily accumulated during shaking and just a small portion developed during the post-shaking period due to pore-pressure dissipation. Furthermore, it was revealed that the presence of the foundation significantly affected excess pore pressure buildup, relative to the free field, thus complicating the overall interpretation of foundation performance.

Plenty simulations with advanced numerical codes have been performed in recent years (e.g. Karamitros et al., 2013; Bray & Macedo, 2017; Bullock et al., 2018) to quantify the role of the different parameters.

3.4.1 Effective stress analysis

The increased power of numerical computation enables to perform very accurate and complete analyses of the seismic response of a building considering the coupling among subsoil, foundation and building. An example of effective stress analyses performed in this project is shown in Figure 3-11. The subsoil investigated with a CPTU tests is modelled as multi-layered deposit with an alternance of sandy and clayey strata. In the present calculation carried out with a commercial finite difference code (Itasca, 2016) the clayey and sandy layers are simulated respectively with a Mohr Coulomb hysteretic model and with a non-linear model (PM4 sand, Boulanger & Ziotopoulou, 2012). In particular, the last model considers the distortional-volumetric coupling of sands in relation to the initial relative density of the material and stress, strain and excess pore pressures are determined by the coupled simulation of seismic wave propagation and seepage.

The model is subjected to a given acceleration time history at the bottom. As shown by the sequence of images, the motion is amplified at the top of the deeper clay layer and excess pore pressures continuously grow in the above sand until a value of r_u (ratio between excess pore pressure and initial vertical effective stress) equal to 1 is attained. Afterwards, the propagation of shaking to the ground level is inhibited by the liquefied sand that substantially behaves as a seismic isolator. Compared with the simplified methods, the effective stress analysis simulates along the entire time history the interdependency of the different sandy and clayey layers. Effects at the ground level are thus given by the combination of shaking and pore pressures development.

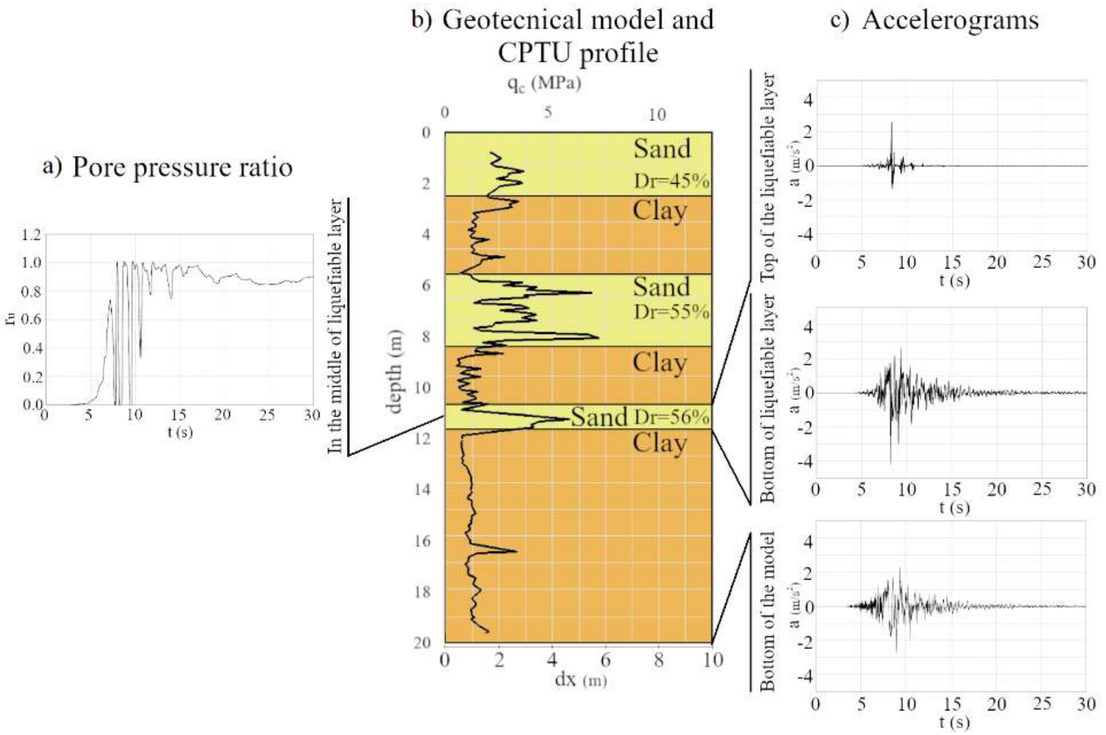

Figure 3-11: Effective stress analyses for liquefaction assessment: (a) excess pore pressure ratio in the sandy layer, (b) geotechnical model and CPTU profile, (c) acceleration time histories at different depths.

The calculation examples refer to the earthquake that struck Emilia Romagna (Northern Italy) in May 2012 with an intense seismic activity having two major sequences occurred respectively on May 20th (main event Mw = 5.9 - hypocentral depth 6.3 km) and on May 29th (main event Mw = 5.8 - hypocentral depth 10.2 km). Widespread liquefaction was observed in areas located near old abandoned watercourses, especially in the municipalities of Sant'Agostino and Mirabello, located along the old riverbed of the Reno River. The village of San Carlo, Municipality of Sant'Agostino, is the most emblematic area for the greatest concentration of liquefaction evidence (Fioravante et al., 2013). The subsoil of San Carlo is the product of a relatively recent geologic history, characterized by an intensive depositional sequence of the Reno river (Figure 3-12.b) and a very shallow water table. Its urban area is mainly built near the paleo-channel and paleo-levees of the Reno River (Figure 3-12.a) and consequently the subsoil can be categorized in three main units. Starting from the top, fluvial channel deposits few meters deep are located above a stratum of fine-grained materials (swamps) and Pleistocene alluvial plain (Figure 3-12.b). Manmade silty sand layers built to protect the area against flooding (paleo-levees) are positioned along the old riverbed.

The seismic input at San Carlo has been computed for the seismic event of May 20th 2012 following the procedure suggested by Sinatra & Foti (2015) illustrated in Figure 3-13. The acceleration time history recorded at the ground level in Mirandola (Luzi et al., 2016) has been firstly deconvoluted to the bedrock, then transferred to San Carlo adopting the attenuation law given by Bindi et al. (2011), finally convoluted to the base of the numerical model. Deconvolution and convolution have been accomplished with one-dimensional equivalent linear calculation performed with EERA (Bardet et al., 2000).

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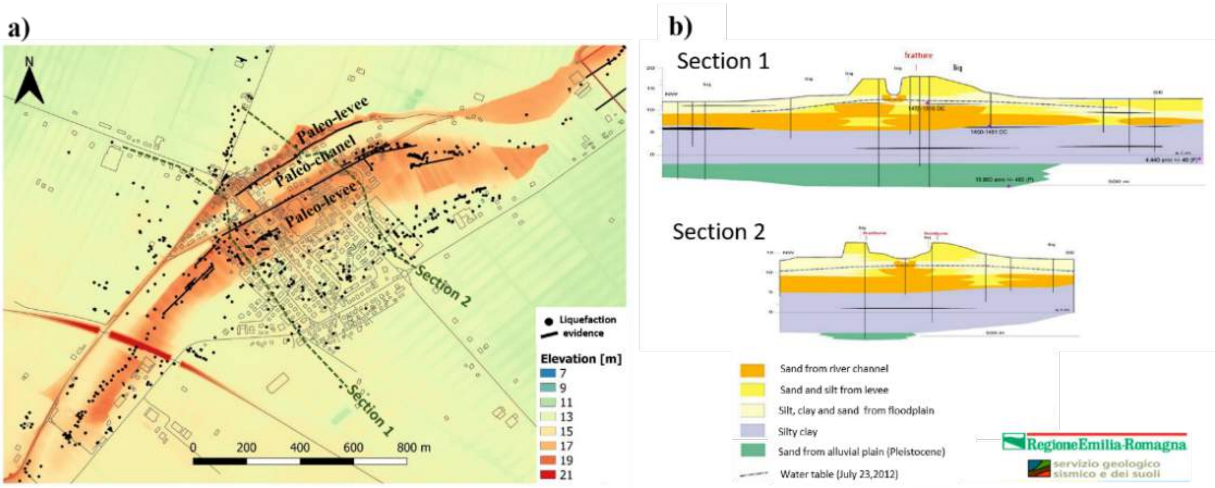

Figure 3-12: Digital Elevation Model (a) and representative geological profiles (b) of San Carlo (Emilia Romagna Region).

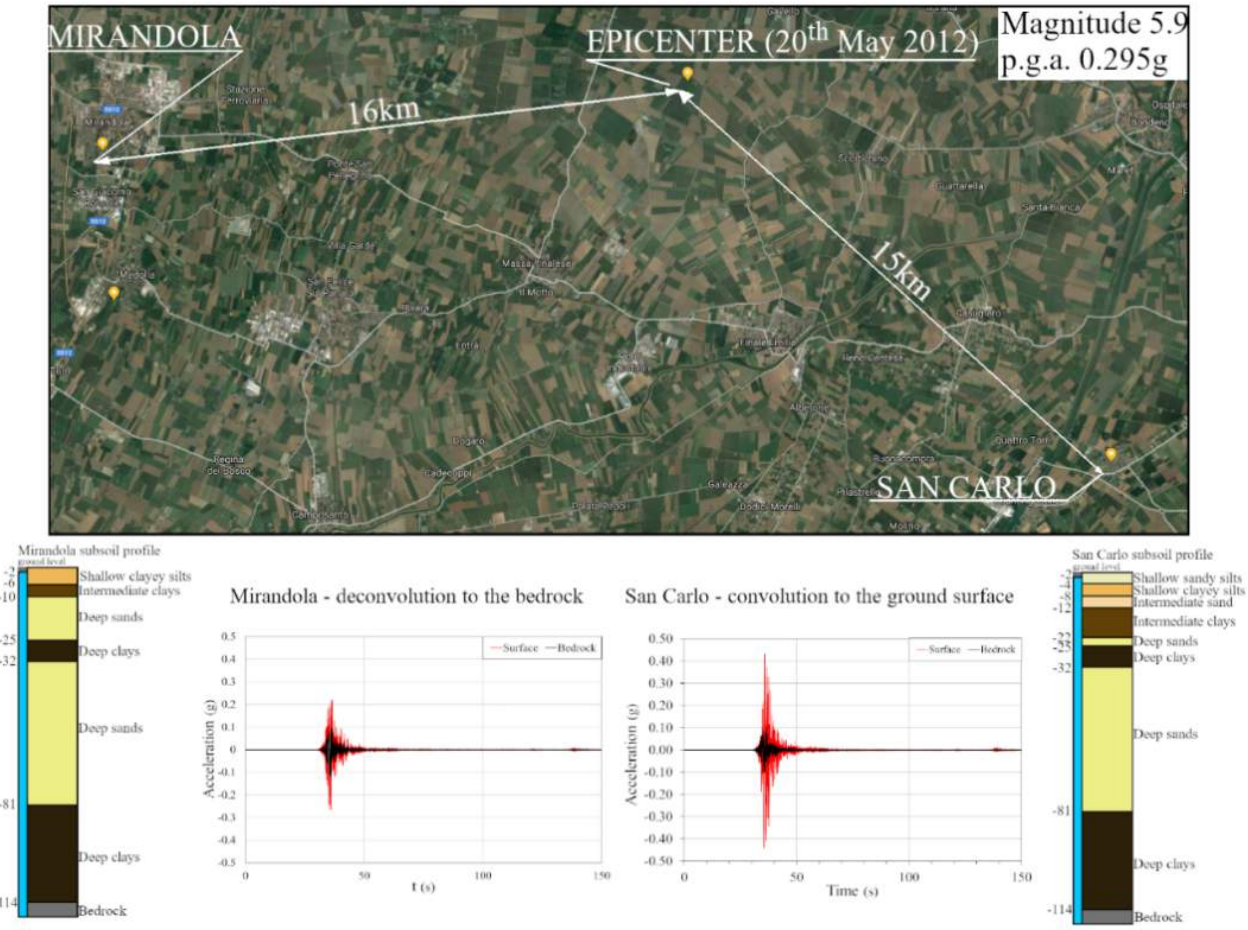

Figure 3-13: Calculation of the seismic input given at San Carlo by May 20th 2012 earthquake.

The previously shown numerical model has been validated (Figure 3-14) against the case of a building located in the village exploiting a set of available information on the mechanical properties of subsoil and building (dimensions, load distribution, foundation type) and the reconnaissance of damage available from reconstruction reports. This building was selected as it suffered relatively high displacements that reached 35 and 5 cm at respectively the left and right corners of the foundation.

The constitutive models and parameters for the different material are summarized in Table 3-2. In particular, the properties of the lower clay and upper silt have been fixed looking at tests performed by Sinatra & Foti (2015). The PM4 sand model has been calibrated fitting with the numerical model the liquefaction curve (CSR versus number of cycles triggering liquefaction) proposed by Idriss &Boulanger (2008) where the CSR value corresponding to N=10 cycles is put equal to that corresponding to the normalized cone resistance (q_{c1N}) found in the area. The numerical analysis reproduces rather closely the settlements recorded at the different locations.

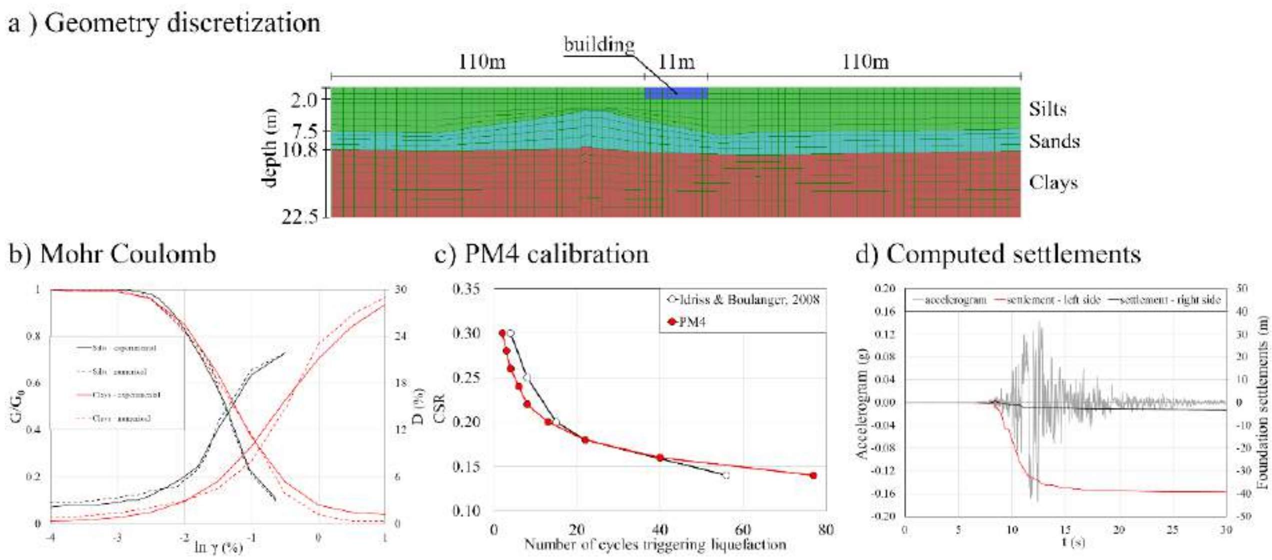

Figure 3-14: Validation of the numerical model on a selected case study in San Carlo (Emilia Romagna, Italy).

Table 3-2: Constitutive parameter adopted in the numerical analysis.

Stratum	Physical		Mohr Coulomb					Hysteretic	
	γ(kN/m^3)	k (m/s)	υ	G_0(MPa)	K(MPa)	c (kPa)	ϕ (°)	L_1	L_2
Silty crust	17.0	2.89E-7	0.3	17.99	18.83	10	20	-3.00	0.50
Clay	18.3	3.99E-9	0.3	73.20	85.40	10	22	-2.20	0.30

	γ(kN/m^3)	k (m/s)	PM4 sand						
			Dr	G_0(MPa)	h_{p0}	ϕ (°)	n_d	n_b	A_{d0}
Sand	18.0	2.89 E-7	0.4	41.63	1.5	33	0.2	0.3	0.2

3.4.2 Simplified methods

3.4.2.1 Ultimate Limit State

Failure of shallow foundation-soil systems during an earthquake may be dictated by the increase in magnitude and orientation of the loads coming from the superstructure coupled with the dynamic shear stresses activated in the soil. Together with these mechanisms applicable to any soil type (cohesive or cohesionless), a shear strength degradation may occur in saturated cohesionless (sand, sand and gravel, non-plastic or low plasticity silt) under the water table. Consequences develop rapidly and may lead to total loss of bearing capacity. In addition, unlike soil and superstructure inertia effects, liquefaction effects do not vanish at the end of shaking but persist for as long as it takes to earthquake-induced excess pore pressures to dissipate.

Cascone & Bouckovalas (1998) suggest a simplified three-step procedure to compute the ultimate limit load of the foundation in this condition:

1) compute dynamic shear stresses or strains in the foundation soil;
2) compute the corresponding excess pore pressures, either from experimental data or from appropriate empirical relationships;
3) compute the "equivalent static" bearing capacity of the foundation considering the reduction in effective stresses and shear strength induced by the build-up of excess pore water pressures.

The authors propose a modification of the empirical relationships for seismic pore pressures in sands proposed initially by Seed et al. (1976) and in simpler form, by Seed & Booker (1977):

$$r_u = 0.65 \cdot arcsin\left[N^{\frac{1}{2}\alpha} \cdot sin\left(\frac{\pi}{2} \cdot U_1\right)\right]^{\frac{1}{2}\beta} \quad \text{Equation 3-18}$$

where:

$$U_1 = B\left(\frac{\tau_c}{\sigma'_0}\right)^{\beta} \cdot D_r^{\delta} \quad \text{Equation 3-19}$$

τ_c is the cyclic stress amplitude, σ'_0 is the effective confining stress prior to the earthquake, N the number of equivalent uniform loading cycles, D_r the soil relative density, the parameters B (=2.70), α (=0.70), β (=2.80) and δ (=-4.00) are estimated from statistical analysis of experimental data from shaking table tests on sand presented by De Alba et al. (1976).

The developed pore pressure is then responsible for the soil strength:

$$\tau_f = \sigma'_{vf} \cdot tan\varphi' = (\sigma'_{vo} - u) \cdot tan\varphi' = \sigma'_{vo}(1 - r_u) \cdot tan\varphi' = \sigma'_{vo} \cdot tan\varphi^{*\prime} \quad \text{Equation 3-20}$$

where:

$$tan\varphi^{*\prime} = (1 - r_u) \cdot tan\varphi' \quad \text{Equation 3-21}$$

Practically, the increase of pore pressure is equivalent to a degradation of the friction angle (Figure 3-15). This friction angle can be used to compute the ultimate load of the foundation. For a strip footing overlying

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a layer of clay having unit weight γ_1, undrained shear strength c and thickness H superposed to a sand deposit having unit weight γ_2 the following limit load can be computed as the minimum between:

$$q_c = 5.14\, c - \gamma_1 \cdot D \qquad \text{Equation 3-22}$$

and

$$q_s = \left[\frac{2cH}{B} - \gamma_1 H\right] + \left[\frac{1}{2}\gamma'_2 B N^*_\gamma + \gamma'_1 (H+D) N^*_q + \gamma'_w (H+D)\right] \qquad \text{Equation 3-23}$$

where N*γ and N*q are bearing capacity coefficients computed as a function of the degraded friction angle.

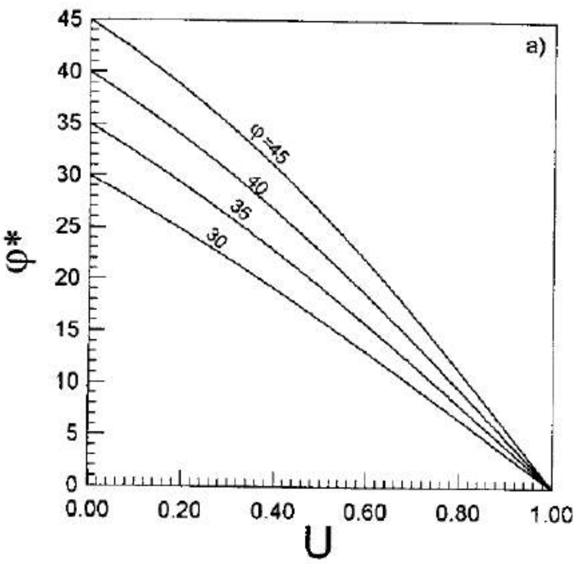

Figure 3-15: Soil shear strength degradation due to the increase of pore pressure (Cascone & Bouckovalas, 1998).

The methodology for Ultimate limit state assessment of foundations and the input for mitigation can be computed with the procedure depicted in Figure 3-16. The key point of this assessment is represented by the computation of the pore pressure ration below the footing that affect the ultimate load of the foundation via a reduction of the friction angle like that given in Equation 3-21 and shown in Figure 3-15.

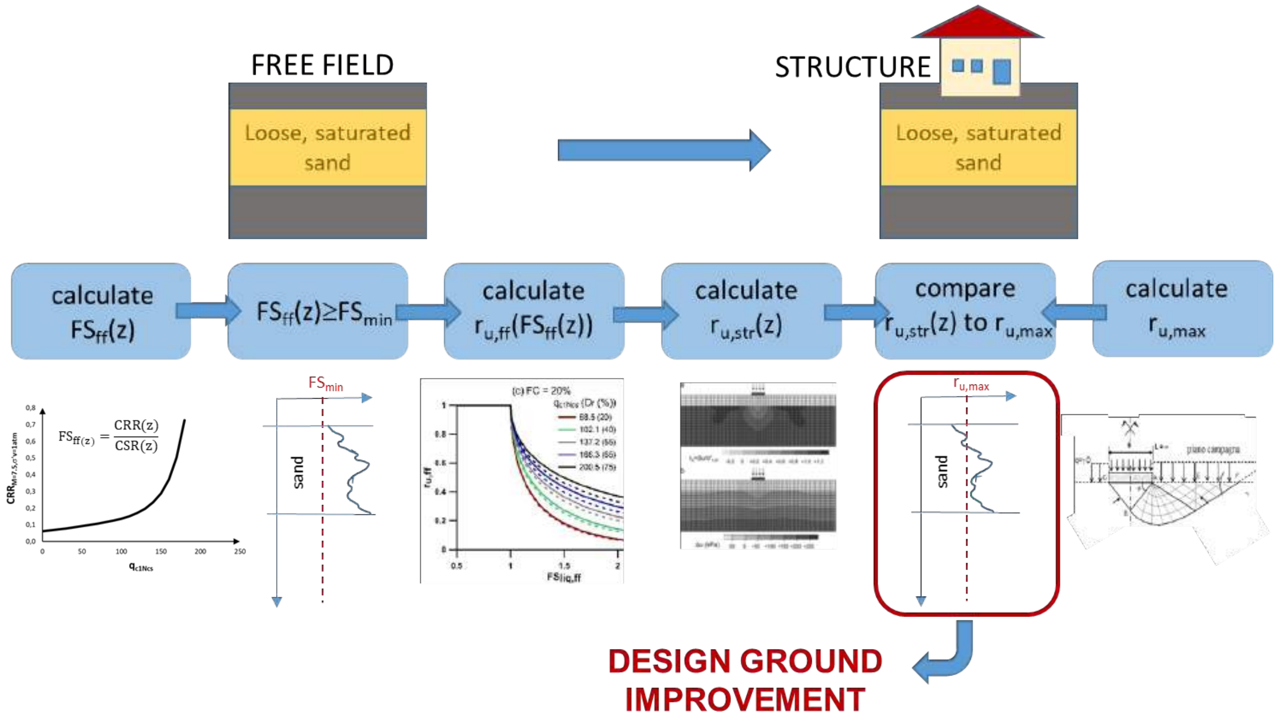

Figure 3-16: Ultimate Limit State assessment of foundation on liquefiable layers.

The whole computation procedure proposed by Karamitros et al. (2013) is described in Figure 3-17. Here the pore pressure ratio used to compute the friction angle degradation ($r_{u,str}$) is the mean between the one computed in free field conditions ($r_{u,ff}$ see for instance one given in Figure 3-10) and the one computed in a representative point C below the foundation $r_{u,foot}$. The latter is simply approximated as the ratio between the original effective stress at the characteristic point C (in the conservative assumption that liquefaction is attained in free field) and the vertical effective stress after the footing is placed multiplied times a dynamic correction factor α. Karamitros et al. find that this factor depends on the ratio between dynamic settlement and width of the footing, but a value equal to 0.8 can be conservatively assigned for ultimate limit states where settlements are generally large. Once the pore pressure ratio $r_{u,str}$ is obtained, the degraded friction angle and the ultimate limit load can be computed. In this calculation ground improvement can thus added considering its effect on the reduction of the pore pressure ratio.

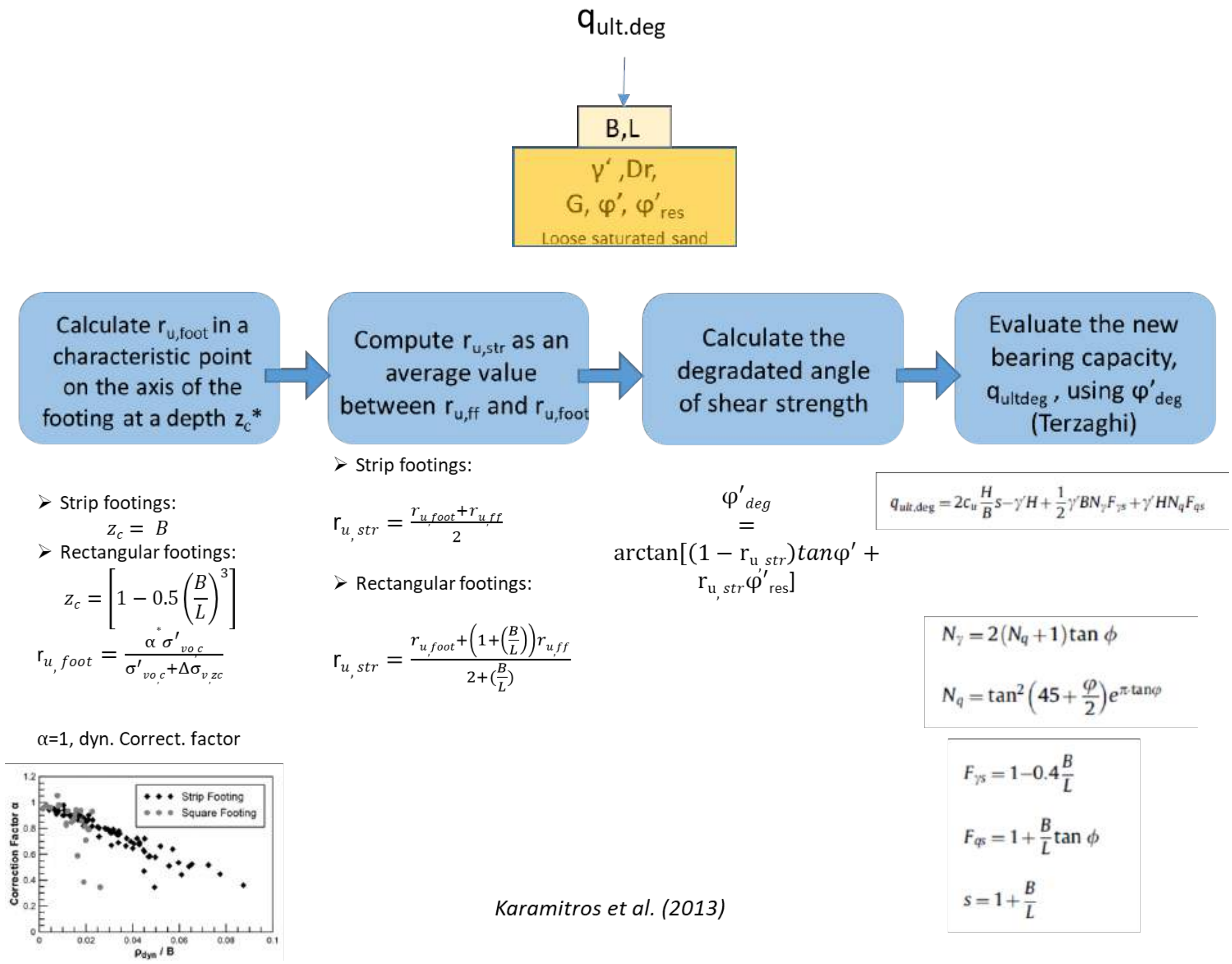

Figure 3-17: Calculation of the Ultimate Limit Load of foundations on liquefiable layers.

3.4.2.2 Serviceability Limit State

The assessment of serviceability limit states requires that the absolute and differential displacement of a foundation keep below the corresponding admissible values. Referring to Figure 3-18, the quantities to be considered are:

- the overall settlement w;
- the differential settlement δ_{AB} between two points A and B;
- the tilt α;
- the angular distortion or relative rotation β_{AB} between two points A and B, equal to the differential settlement divided by the distance L_{AB} between the two points ($\beta_{AB} = \delta_{AB}/L_{AB}$);
- the relative deflection δ_{AB} of wall and panels between two points A and B;
- the deflection ratio equal to the relative deflection divided by the distance between the two points ($\eta_{AB} = \delta_{AB}/L_{AB}$).

Data on allowable values for the above quantities, collected by Poulos et al. (2001) are listed in Table 3-3. The use of a single quantity, such as angular distortion or deflection ratio, to assess building damage neglects several important factors related to the building performance (flexural and shear stiffness, geometrical configuration), to the subsoil (coarse-grained or fine-grained soils, and related differences in the rate of occurrence of settlement) and to the nature of ground movement profile (e.g. sagging or hogging).

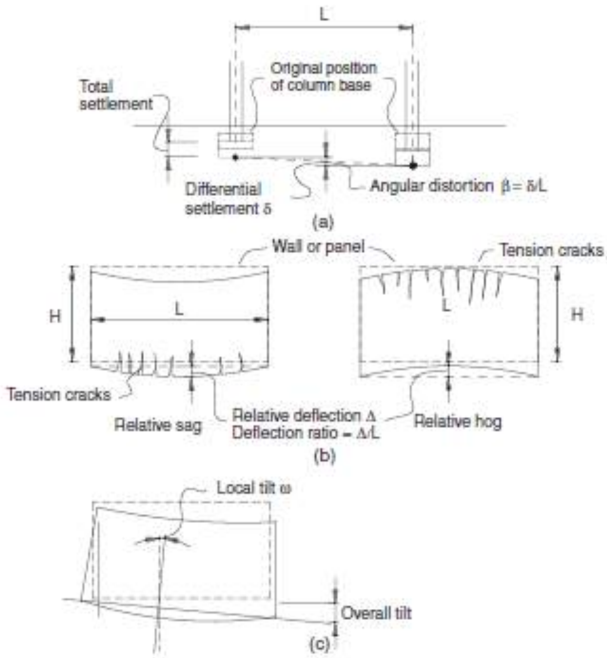

Figure 3-18: Deformation of foundation and relevant quantities (Burland & Wroth, 1974).

Table 3-3: Allowable settlement and distortion for structures of different typology.

Type of structure	*Type of damage/ concern*	*Quantity to be considered*	*Limiting value*
Framed building and reinforced load bearing walls	Structural damage	Angular distortion	1/150–1/250
	Cracking in walls and partitions	Angular distortion	1/500 (1/100–1/1400 for end bays)
	Visual appearance	Tilt	1/300
	Connection to services	Total settlement	50–75 mm (sand) 75–135 mm (clay)
Tall buildings	Operation of lifts and elevators	Tilt after lift installation	1/1200–1/2000
Structures with unreinforced load bearing walls	Cracking by sagging	Deflection ratio	1/2500 (L/H = 1) 1/1250 (L/H = 5)
	Cracking by hogging	Deflection ratio	1/5000 (L/H = 1) 1/2500 (L/H = 5)
Bridges – general	Ride quality	Total settlement	100 mm
	Structural distress	Total settlement	63 mm
	Function	Horizontal movement	38 mm
Bridges – multiple span	Structural damage	Angular distortion	1/250
Bridges – single span	Structural damage	Angular distortion	1/200

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Boscardin & Cording (1989) pointed out the importance of horizontal strain, and derived the relation shown in Figure 3-19 between the degree of damage, the horizontal strain and the angular distortion. It may be seen that the larger the horizontal strain, the less is the tolerable distortion before some form of damage occurs. Such consideration may be important when assessing potential damage arising from different sources.

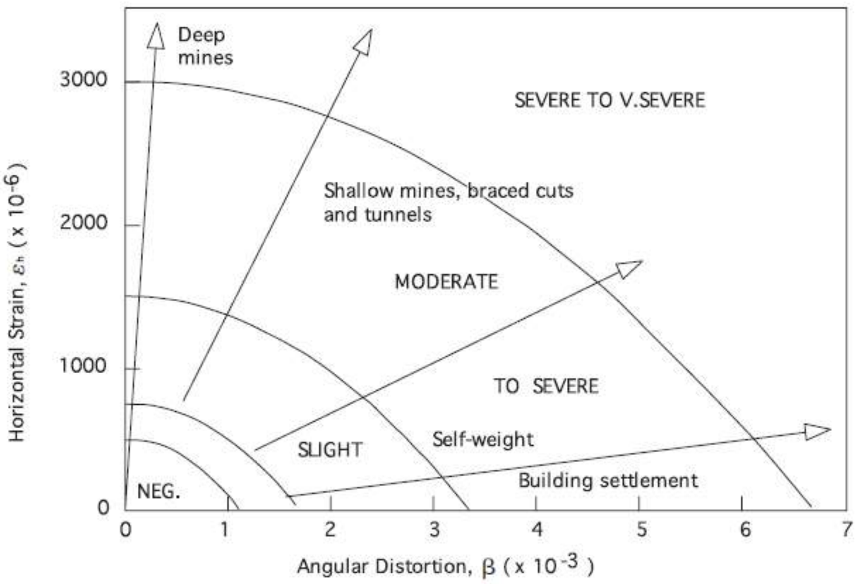

Figure 3-19: Damage level as function of horizontal strain and angular distortion (after Boscardin & Cording, 1989).

The influence of foundation stiffness has been herein investigated with a number of numerical analyses simulating the case of a shallow foundation (having width B=10m) transferring a uniform load of 50 kPa to a layered subsoil formed by an upper cohesive crust 4 m thick and a liquefiable layer 6 m thick. Earthquakes of different amplitudes have been applied scaling with three values (respectively 0.7, 1.0 and 1.6) the acceleration time history of the May 22nd earthquake in Mirandola (Emilia Romagna - Italy). The numerical analyses have been performed with a numerical code (FLAC – ITASCA, 2016) adopting a hysteretic Mohr-Coulomb model for the upper crust a non-linear model (PM4 sand, Boulanger & Ziotopoulou, 2012) for the liquefiable layer. Different flexural stiffnesses have been assigned to the foundation slab. As can be seen from Figure 3-20, absolute settlements and angular distortion are mutually related, but the connection varies with the flexural stiffness of the foundation. It is interesting to compare these results with experimental data collected by Grant et al. (1974). Results shows that the numerical relation found for nil flexural stiffness of the foundation slab coincides with the upper bound of the experimental values.

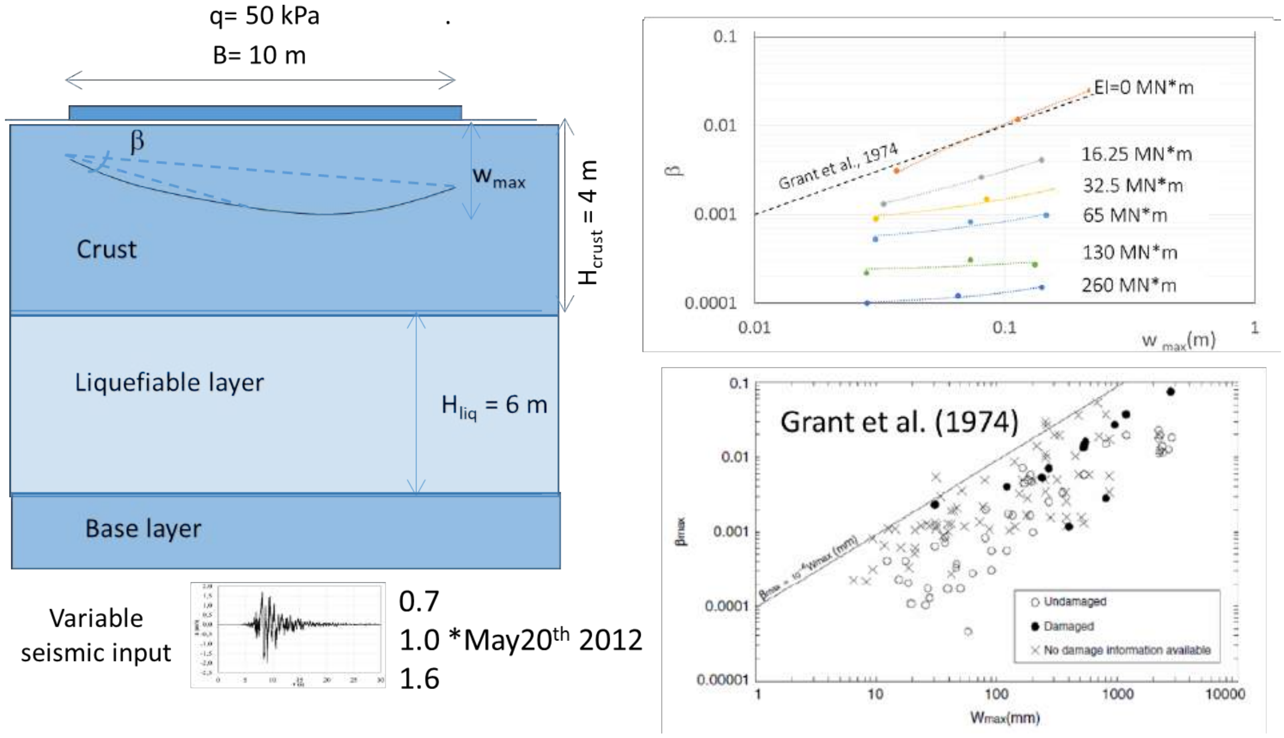

Figure 3-20: Example of parametric numerical calculation showing the effect of foundation stiffness on the relation between angular distortion and absolute settlement.

The above relation highlights the importance of quantifying the absolute settlement to quantify the Serviceability performance of the foundation. In the following, three up to date methods have been reported that provide simplified equations aimed at computing the settlements of foundation induced by liquefaction. In all cases, the proposed formulas fit the results of a large number of numerical simulations.

3.4.2.3 Karamitros et al. formula (2013)

Karamitros et al. (2013) gives a simplified analytical formula for the computation of the seismic settlements of strip and rectangle footings resting on a clay crust overlying a liquefiable layer. Such settlement is associated to a ''sliding-block'' type of punching failure through the clay crust and within the liquefied sand layer. In particular, liquefaction-induced settlements are correlated to the seismic excitation characteristics and the post-shaking degraded static factor of safety, while the effect of shear-induced dilation of the liquefied subsoil is also taken into account.

The proposed expression for the dynamic settlement ρ_{dyn} (i.e. the settlement during shaking) is shown in Equation 3-24, being *c* a foundation aspect ratio correction (3.25, where *c'*=0.003), a_{max} the peak bedrock acceleration, T the representative period of the motion, N the number of cycles of the excitation, Z_{liq} the thick liquefiable sand layer, B the structure width and FS_{deg} the degraded static factor of safety of the foundation.

$$\rho_{dyn} = c a_{max} T^2 N \left(\frac{Z_{liq}}{B}\right)^{1,5} \cdot \left(\frac{1}{FS_{deg}}\right)^3 \qquad \text{Equation 3-24}$$

$$c = c'\left(1 + 1{,}65 \cdot \frac{L}{B}\right) \leq 11{,}65\, c' \quad \text{Equation 3-25}$$

$$\mathrm{a_{max} T^2 N} = \int_{\mathrm{t=0}}^{\mathrm{t}} |\mathrm{v(t)}|\mathrm{dt} \quad \text{Equation 3-26}$$

FS_{deg} in Equation 3-24 can be calculated as the degraded bearing capacity ($q_{ult,deg}$) divided by the bearing pressure (q) (Equation 3-27). The foundation bearing capacity failure mechanism is simulated by the Meyerhof & Hanna (1978) model for a crust on a weak layer using the degraded friction angle in Equation 3-21 where r_u is the average excess pore pressure ratio of the liquefied sand and ϕ_0 is the initial friction angle. Superficial crust is beneficial and there is an upper bound beyond where failure occurs entirely within the crust and does not get affected by the liquefiable layer.

$$FS_{deg} = \frac{q_{ult,deg}}{q}$$	Equation 3-27

Such methodology was evaluated against results from a large number of relevant centrifuge and large-scale experiments, as well as against observations of the performance of shallow foundations in the City of Adapazari, during the 1999 Kocaeli Earthquake. Even if good agreement was found among analytical predictions and liquefaction-induced settlements, in future applications the parameters of the numerical analyses should be respected.

3.4.2.4 Bray and Macedo (2017)

Combining in-situ observation, experimental tests and numerical analyses, Bray & Macedo (2017) propose a method to evaluate the shear-induced building settlement (Ds) due to liquefaction below the building. The simplified procedure for estimating liquefaction-induced building settlement involves these steps:

- Perform a liquefaction triggering assessment and calculate the safety factor against liquefaction (FSL) for each potentially liquefiable soil layer preferably using a CPT-based method (e.g., Boulanger & Idriss, 2016).
- Calculate the post-liquefaction bearing capacity factor of safety (FS) using the simplified two-layer solution of Meyerhof & Hanna (1978), where the average shear strength of the non-liquefied crust layer represents the top layer and the post-liquefaction residual shear strength of the liquefied soil layer represents the bottom layer. If the post-liquefaction bearing capacity FS is less 1.0 for light or low buildings or less than 1.5 for heavy or tall buildings, large movements are possible, and the potential seismic building performance can generally be judged to be unsatisfactory.
- Estimate the likelihood of sediment ejecta developing at the site by using ground failure indices such as LSN, LPI, or the Ishihara (1985) ground failure design chart. If the amount of sediment ejecta is significant, estimate the amount of building settlement as a direct result of loss of ground due to the formation of sediment ejecta (De). This can best be done using relevant case histories to estimate the amount of ejecta and then assuming that the ejecta has been removed below the building foundation.

- Estimate the amount of volumetric-induced building settlement (Dv) preferably using a CPT-based method (e.g., Zhang et al., 2002).
- Estimate the shear-induced building settlement (Ds) due to liquefaction below the building using Equation 3-29:

$$ln(Ds) = c1 + 4.59 \cdot ln(Q) - 0.42 \cdot ln(Q)^2 + c2 \cdot LBS + 0.58 \cdot ln(tanh(HL)) - 0.02 \cdot B + 0.84 \cdot ln(CAV_{dp}) + 0.41 \cdot ln(S_a) + \varepsilon \quad \text{Equation 3-28}$$

$$LBS = \int W * \frac{\varepsilon_{shear}}{z} dz \quad \text{Equation 3-29}$$

where Ds is in mm, LBS is calculated with 3.29, c1=-8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2=0.014 otherwise. Q is in units of kPa, HL is in m, B is in m, CAV_{dp} is in g-s, and Sa1 is in g; ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. CAV_{dp} is the standardised Cumulate Absolute Velocity as defined in Campbell and Bozorgnia (2012) where N is the number of discrete 1 second time intervals, x is PGAi-0.025 (PGAi is the value of the peak ground acceleration (g) in time interval i, inclusive of the first and last values) and H(x) is 0 if x<0 or 1 otherwise. LBS (Equation 3-29) is an index of equivalent liquefaction-induced shear strain on the free-field (ε_{shear}), defined as the integration along the soil column of the strain estimated by means of the CPT-based procedure proposed in Zhang et al. (2004), weighted by the depth in order to provide more importance to the soil close to the foundation). ε_{shear} is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL). z(m) is the depth measured from the ground surface > 0 and W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W= 1.0 otherwise. Finally, the total liquefaction-induced building settlement (Dt) can be estimated from Equation 3-30, as:

$$Dt = De + Dv + Ds \quad \text{Equation 3-30}$$

3.4.2.5 Bullock et al. formulation (2018)

The authors present a comprehensive predictive relation for the settlement of shallow-founded structures on liquefiable ground during earthquakes. The relation is derived interpolating with a non-linear regression and latent variable analysis the results of an extensive fully coupled three-dimensional numerical parametric study of soil–structure systems, validated with centrifuge experiments and adjusted with case history observations to capture all mechanisms of settlement below the foundation, including volumetric and deviatoric strains as well as ejecta. The resulting probabilistic building settlement model incorporates the influence of the soil profile, the presence and properties of the structure and the characteristics of the ground motion, thus providing engineers with a comprehensive procedure for predicting liquefaction-induced settlement of a mat-founded building. The formula is written as:

$$ln(\bar{S})_{num} = f_{so} + f_{fnd} + f_{st} + s_o ln(CAV) \quad \text{Equation 3-31}$$

where $\ln(\bar{S})_{num}$ is the natural logarithm of the median predicted numerical foundation settlement (mm) and f_{so}, f_{fnd} and f_{st} are functions that capture effects due to the characteristics of the soil profile, foundation and the structure, respectively.

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The first term is computed as:

$$f_{so} = \left[\sum_i H \cdot \left(H_{S,i} - 1 + \varepsilon\right) \cdot f_{S,i} \cdot f_{H,i}\right] + \left[c_0 + c_1 \cdot \ln\left(CAV\right) \cdot F_{LPC}\right]$$	Equation 3-32
$$f_{S,i(SPT)} = \begin{cases} a_0, \quad N_{1,60,i} < 12.6 \\ a_0 + a_{1,SPT} \cdot \left(N_{1,60,i} - 12.6\right), \quad 12.6 \leq N_{1,60,i} < 17.2 \\ a_0 + 4.6 \cdot a_{1,SPT}, \quad 17.2 \leq N_{1,60,i} \end{cases}$$ $$f_{S,i(SPT)} = \begin{cases} a_0, \quad q_{c1N,i} < 112.4 \\ a_0 + a_{1,CPT} \cdot \left(q_{c1N,i} - 112.4\right), \quad 112.4 \leq q_{c1N,i} < 140.2 \\ a_0 + 27.8 \cdot a_{1,CPT}, \quad 140.2 \leq q_{c1N,i} \end{cases}$$	Equation 3-33
$$f_{H,i} = b_0 \cdot H_{S,i} \cdot exp\left[b_1\left(max\left(D_{S,i}\right)^2 - 4\right)\right]$$	Equation 3-34

H(-) is the Heaviside step function; ε is an infinitesimal positive quantity to make H(-) equal to 1 for an argument of zero; FLPC is a flag that is equal to 1 if a low-permeability layer is present above the uppermost susceptible layer; $N_{1,60}$ is the corrected standard penetration test (SPT) blow count in the i^{th} layer; $q_{c1N,i}$ is the corrected normalized cone penetration test (CPT) tip resistance in the i^{th} layer; $H_{S,i}$ is the thickness of the i^{th} susceptible layer; and $D_{S,i}$ is the depth from the bottom of the foundation to the centre of the i_{th} susceptible layer. The term related to the presence of a low-permeability cap indicates that its influence is dependent on motion intensity.

The second term is computed as:

$$f_{fnd} = f_q + f_{B,L}$$ $$f_q = \{d_0 + d_1 \cdot ln[\min\left(CAV, 1000\right)]\} \cdot \ln\left(q\right) \cdot exp\{d_2 \cdot min[0, B - \max\left(D_{S,1}, 2\right)]\}$$ $$f_{B,L} = \{e_0 + e_1 \cdot ln[\max\left(CAV, 1500\right)]\} \cdot [\ln(B)]^2 + e_2 \cdot \left(\frac{L}{B}\right) + e_3 \cdot D_f$$	Equation 3-35

where q is the bearing pressure of the foundation (in kPa), B is the width of a rectangular foundation (m); L=B is its unitless length-to-width ratio; and D_f is the depth from the surface to the bottom of the foundation (m). $D_{S,1}$ is the depth to the centre of the uppermost susceptible layer with $N_{1,60}$ less than 17.2 blows (q_{c1N} less than 140.2). The exponential decay term included in f_q reduces the influence of q for profiles where there are no loose susceptible layers within the foundation's depth of influence. This decay term, determined to maximise model performance with respect to the numerical database, engages for layer depths greater than B (taken here as the depth of influence), rather than 1.2 B (per Tokimatsu et al., 2017) or 1.5B (per Boussinesq's solution), which were based on the size of stress bulbs beneath a square footing. Using a threshold of B rather than either of these values offered improved model R2 and reduced bias for models with deep layers and multiple layers. This slight difference may be the result of a highly non-linear and elasto-plastic soil response considered in this numerical study, which contradicts simplifying assumptions used by previous researchers in their formulation of the foundation's zone of influence.

The form and intensity threshold of equation (Equation 3-35) captured the effects of the foundation dimensions well (as demonstrated later in this paper). The orders of scaling (natural logarithm squared for B and linear for L=B and D_f) were determined by inspection to minimize residuals.

The third term is:

$$f_{st} = \{f_0 + f_1 \cdot ln[\min(CAV, 1000)]\} \cdot h_{eff}^2 + f_2 \cdot min\left[\left(\frac{M_{st}}{10^6}\right), 1\right] \quad \text{Equation 3-36}$$

where h_{eff} is the effective height of the structure (m) and M_{st} is the inertial mass of the structure (kg). The orders of the terms in this equation reflect the expectation that the building's effective moment of inertia should affect its ratcheting behaviour, which in turn influences settlement. An upper and a lower bound are given to CAV in equations (Equation 3-35, Equation 3-36) are included to allow the functional form to capture the trends in the numerical model. Table 3-4 provides the coefficients for this relation as determined by a non-linear regression analysis.

Table 3-4: Coefficients and uncertainty parameters for the model in Equation 3-32 to Equation 3-36.

Model parameter	Value	Model parameter	Value
a_0	1·000	e_0	−0·8727
$a_{1,SPT}$	−0·2174	e_1	0·1137
$a_{1,CPT}$	−0·0360	e_2	−0·0947
b_0	0·3026	e_3	−0·2148
b_1	−0·0205	f_0	−0·0137
c_0	1·3558	f_1	0·0021
c_1	−0·1340	f_2	0·1703
d_0	−1·3446	s_0	0·4973
d_1	0·2303	σ_{num}	0·3314
d_2	0·4189		

The numerical prediction ($\ln S_{num}$) is used as a floor because this is expected to be an accurate estimate of settlement due to shear-type (deviatoric) deformations. A correction is the applied to fit with observation:

$$\ln(\bar{S})_{adj} = \ln(\bar{S})_{num} + k_0 + k_1 \cdot \min\left(H_{S,1}, 12\right)^2 + k_2 \cdot \min(q, q_c) + \\ + k_3 \cdot \max(q - q_c, 0) \geq \ln(\bar{S})_{num} \quad \text{Equation 3-37}$$

with its coefficients provided in Table 3-5. The model coefficient of the term for large q (k_3) is negative and counteracts the effects of increased $H_{S,1}$ for large values of q.

Table 3-5: Coefficients for the adjustment of the predictive formula as in Equation 3-37.

Model parameter	Value	Model parameter	Value
k_0	−1·5440	k_3	−0·0218
k_1	0·0250	q_c (assumed in kPa)	61
k_2	0·0295	σ_{adj}	0·6746

3.5 Mitigation with ground improvement

The scope of ground improvement is to prevent damages or, in a performance-based approach, to reduce risk at tolerable level. The design methodology changes significantly depending on the type if structure/infrastructure to be protected. In fact, while containment of large deformation induced by fully developed liquefaction (r_u=1) is the goal of mitigating liquefaction on horizontal lifelines (pipelines, sewers, cables), the increase of bearing capacity and the reduction of settlements below prescribed thresholds is the goal when protecting buildings and this target must be achieved even when liquefaction is not fully developed (r_u<1). In the first case, assessment may thus consist in checking that a sufficiently high safety factor exists in free field conditions, while in the second case a more complex analysis involving the soil-structure interaction must be performed. The general characters of liquefaction resistance can be summarily seen in Figure 3-21 that makes reference to the classical stress-based assessment criterion. In this example, the plots are shown with reference to the CPT resistance (see Figure 3-3), showing the dependency of the normalised resistance on the effective overburden pressure and soil density (Figure 3-21.a and c), but similar plots can be traced for other indicators of the soil response (N_{SPT} or V_s).

The assessment can be thus formulated comparing capacity with demand, looking simultaneously at the different plots, each expressing a specific information. In this logic, mitigation may be characterised with a modification of the soil resistance or with a reduction of demand.

Making reference to the first plot (Figure 3-21.a), assessment is performed comparing the cyclic stress ratio $CSR_{M=7.5}\ (\sigma'_{v0} = 101.3\ kPa)$ (Equation 3-6) with the cyclic resistance ratio $CRR_{M=7.5}\ (z)$, the former dependent on the seismic excitation, the latter on the soil state (stress level and density according to Figure 3-21.c). If CSR is larger than CRR (e.g. Figure 3-22), the likelihood of liquefaction increases and, according to the depth and extension of the liquefiable layer, risk at the ground level can become meaningful (see section 3.3). Ground improvement is thus aimed at increasing the soil resistance and its mechanical characterisation could be expressed with an upward shift of the CRR curve.

Alternatively, the soil resistance can be expressed with the plot of Figure 3-21.b reporting the cyclic stress resistance as a function of the number of loading cycles (Boulanger and Idriss, 2014) the latter function of the earthquake magnitude (Idriss, 1999). In this formulation, the seismic demand is represented with a dot whose coordinates are CSR (function of the seismic intensity PGA) and the earthquake magnitude, while the liquefaction resistance is characterised by a curve in the CRR-N plane. The relative position between dot and curve determines the likelihood of liquefaction (e.g. Figure 3-22). In this representation, ground improvement is characterised by an upward shift of the capacity curve.

The fourth plot (Figure 3-21.d) contains information about the pore pressure build-up ratio that, as seen in the previous sections, is strictly related to the soil deformation. It may happen that the outcome of the liquefaction assessment is positive (FS>1), but the r_u value is intolerable for the safety of the considered structure or may lead to excessive deformation, like in the example of Figure 3-19. In this case an increase of the soil liquefaction resistance with ground improvement would also lead to reduce the pore pressure build-up, like in the example of Figure 3-24.

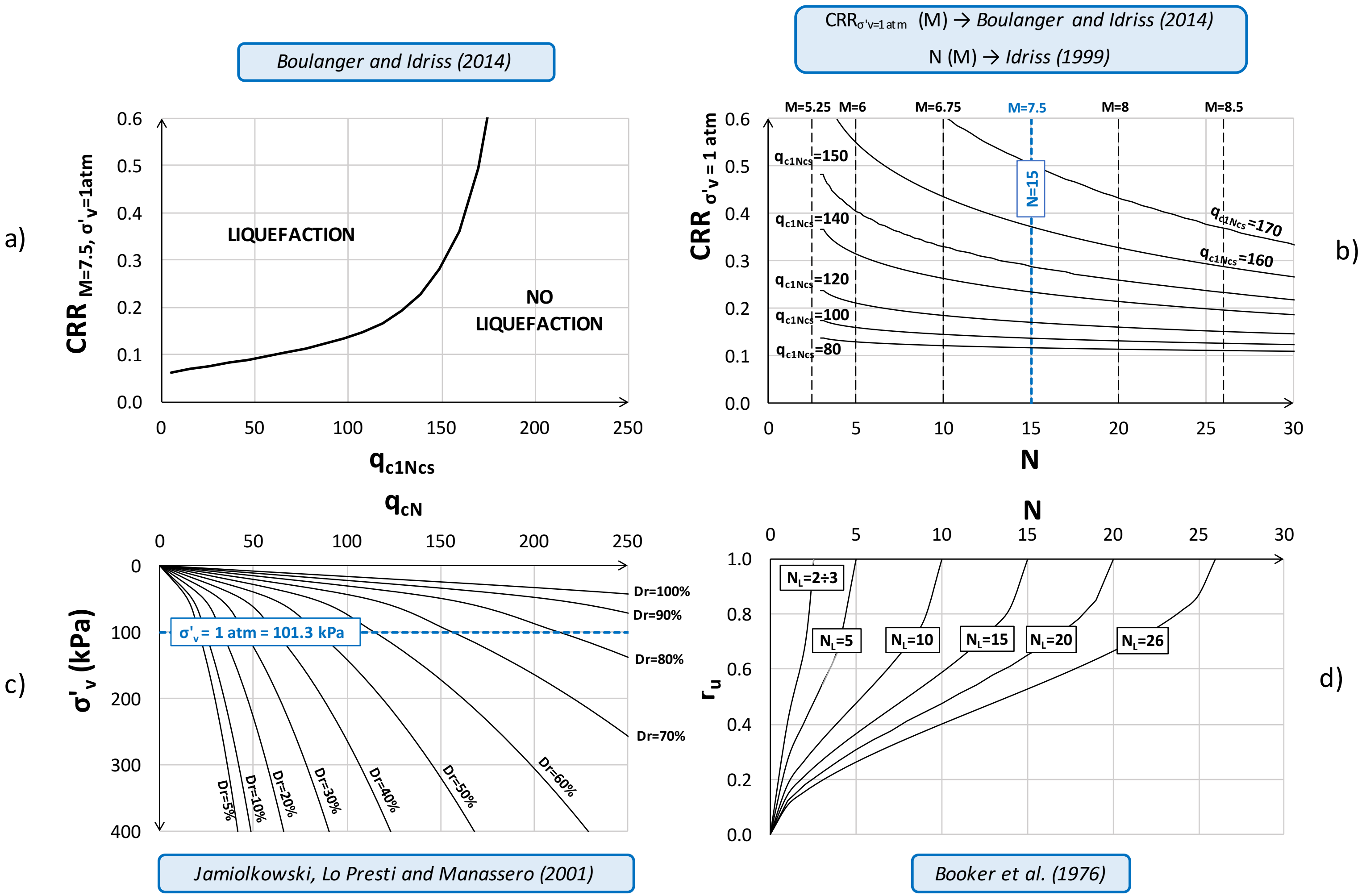

Figure 3-21: Cyclic Resistance Ratio versus the normalized CPT resistance (a. Boulanger and Idriss, 2014) and number of loading cycles/seismic magnitude (b. Boulanger and Idriss, 2014; Idriss, 1999), normalized CPT resistance versus effective stress and soil density (c. Jamiolkowski et al., 2007), Pore pressure ratio versus number of loading cycles (d. Booker, 1976).

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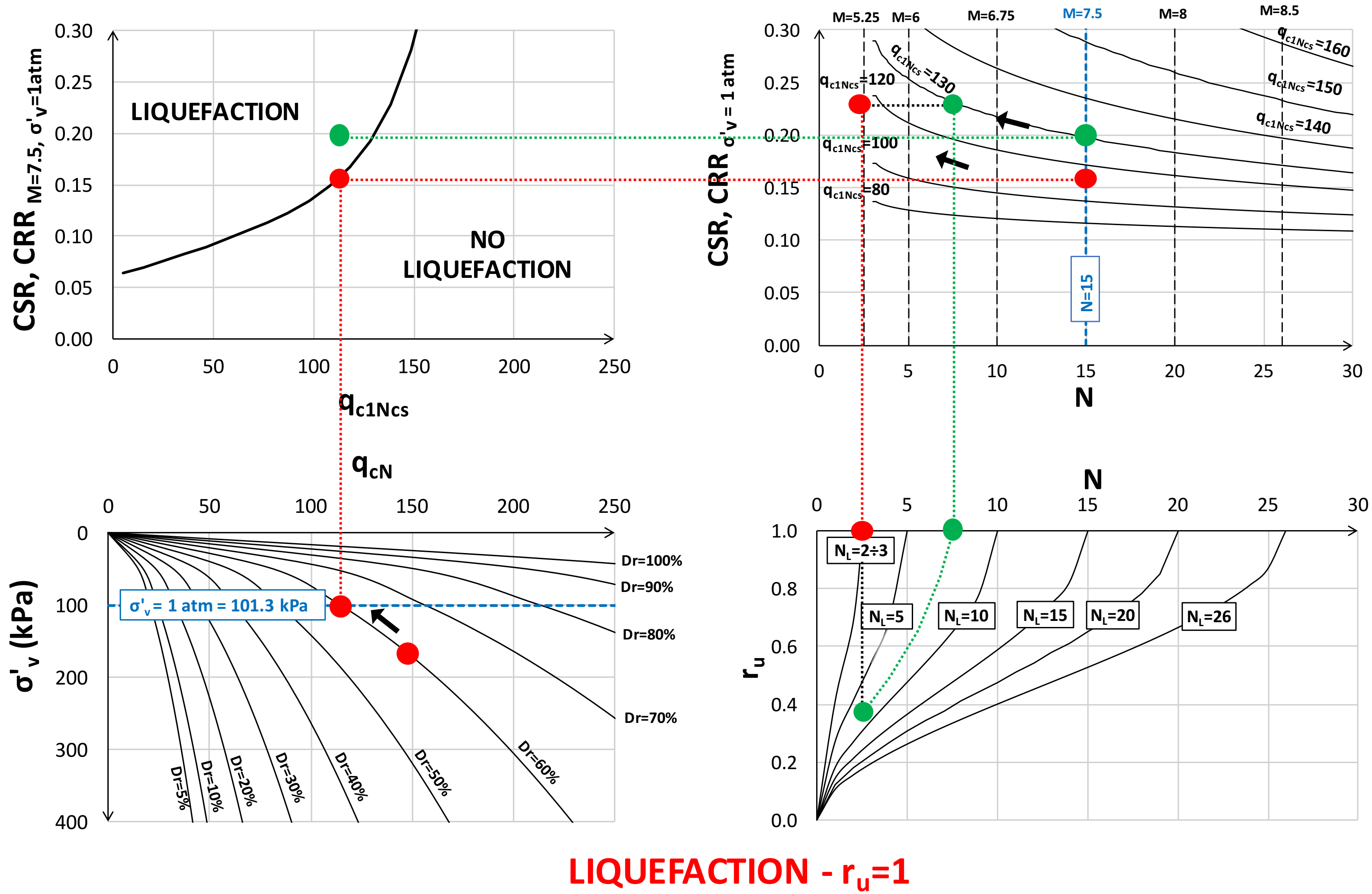

Figure 3-22: Example of liquefaction assessment with negative results (FS<1).

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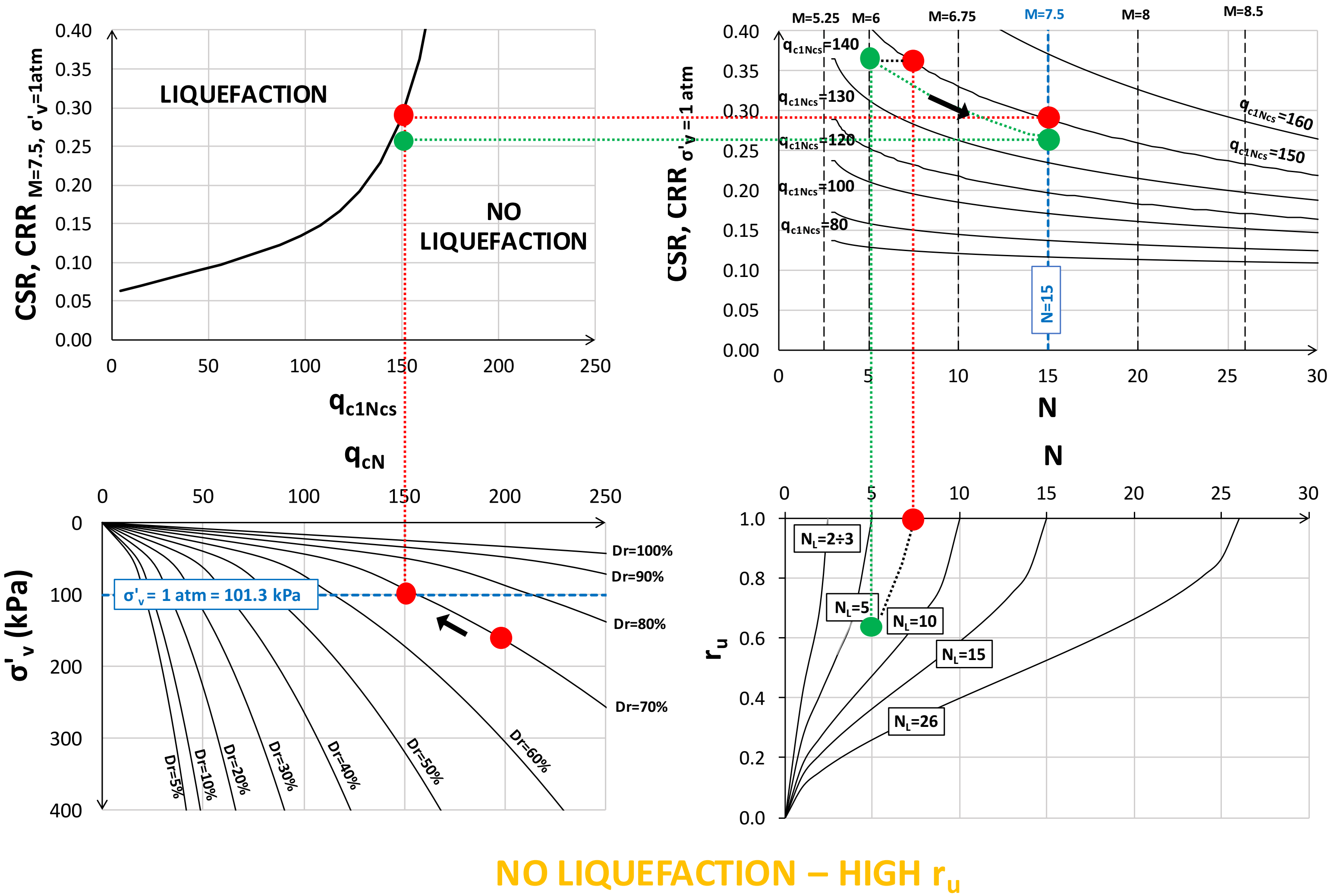

Figure 3-23: Example of liquefaction assessment FS<1 but with high pore pressure ratio r_u.

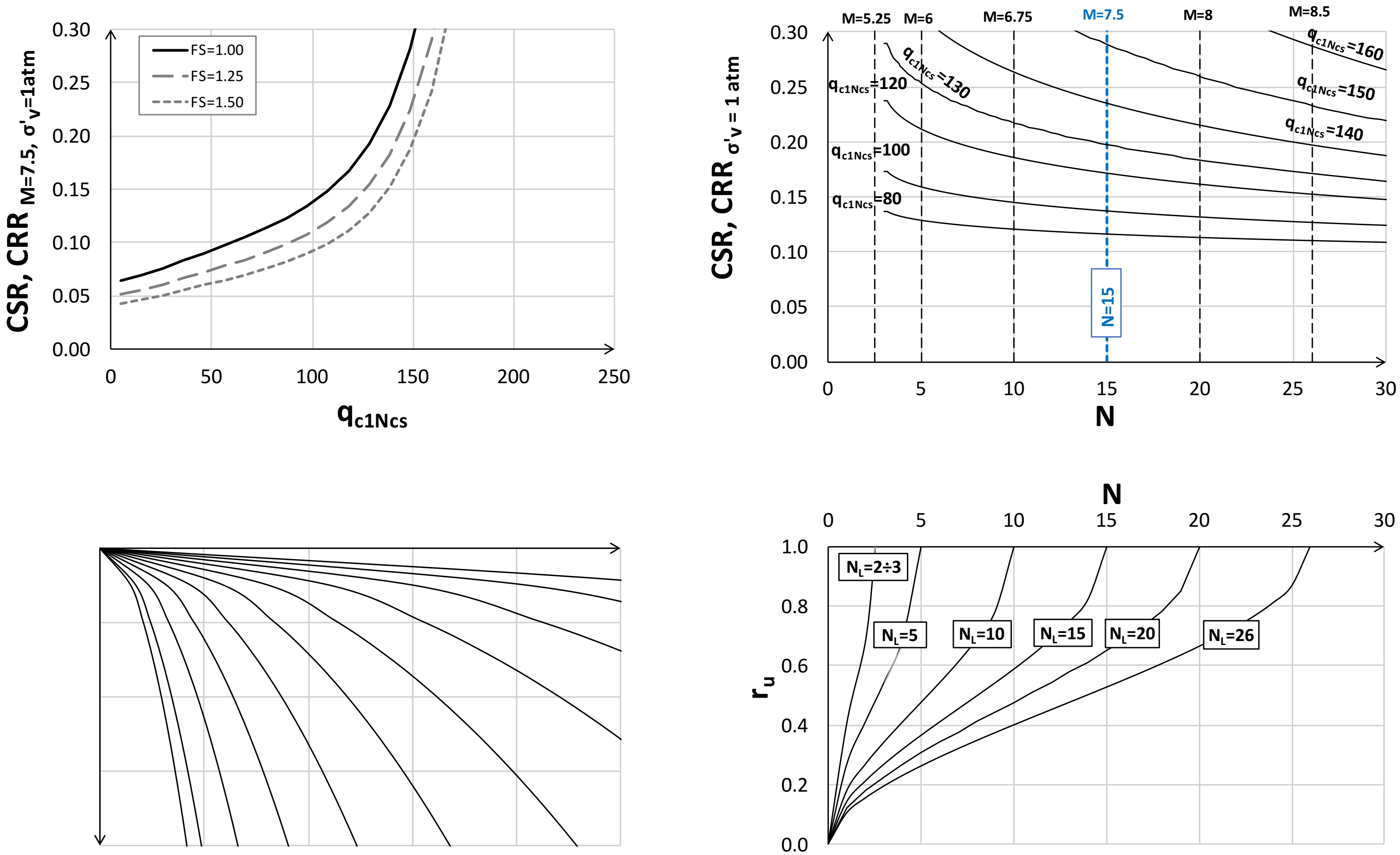

Figure 3-24: Example of liquefaction assessment with positive results (FS<1, low r_u).

The factor of safety can be increased (and the pore pressure increments decreased) by either increasing the soil capacity CRR, decreasing the demand CSR or by some combination of both such that the ratio has a net positive increase. Then to determine the effect that ground improvement techniques will have on the FS it is useful to examine them in terms of how they either increase capacity or decrease demand. The liquefaction potential of sandy soils is again evaluated with the "simplified" procedure that quantifies the FS against liquefaction by defining the capacity of the soil as the cyclic resistance ratio (CRR) and the demand imposed as the cyclic stress ratio (CSR).

Figure 3-25 shows ground improvement approaches for increasing soil capacity and decreasing demand. Increasing soil density (C1) or increasing the lateral effective confining stress (C2) results in an increase in penetration resistance. For these scenarios the boundary between "liquefaction" and "no liquefaction" is assumed to be unaffected. In contrast, preventing contraction of the soil skeleton (C3) or enhancing a rapid dissipation of excess pore water pressure (C4) produces a shift of the boundary between the zones of "liquefaction" and "no Liquefaction," thereby effectively reducing the zone of "Liquefaction". Finally, reduction of demand by shear stress redistribution (D1) results in a decrease of the $CSR_{M=7.5}$ moving the point of interest down into the zone of "no liquefaction".

The following section reports a selection of experimental and analytical studies from the present LIQUEFACT project providing the increase of liquefaction resistance for the most typical ground improvement techniques.

INCREASE CAPACITY	DECREASE DEMAND
C1) Increase soil density C2) Provide a mechanism for rapid dissipation of excess pore water pressure C3) Provide a mechanism to reduce excess pore water pressure	D1) Soil reinforcement/Shear stress redistribution

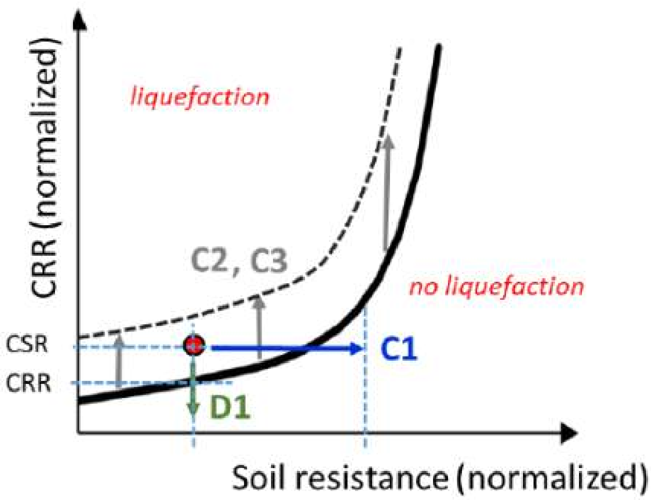

Figure 3-25: Approaches for increasing soil capacity and decrease demand (from Deliverable 4.5 of the present project).

3.5.1 Densification

As shown in Chapter 2, a variety of ground improvement techniques can be grouped as densification, all having the purpose of increasing soil density and reducing the contractive tendency upon shearing. The design requirement can be established with the procedure described in Figure 3-26 and Figure 3-27, being the goal of compaction fixed based on the following two conditions:

- Achievement of a minimum safety factor FS=CRR/CSR;
- Achievement of a maximum r_u.

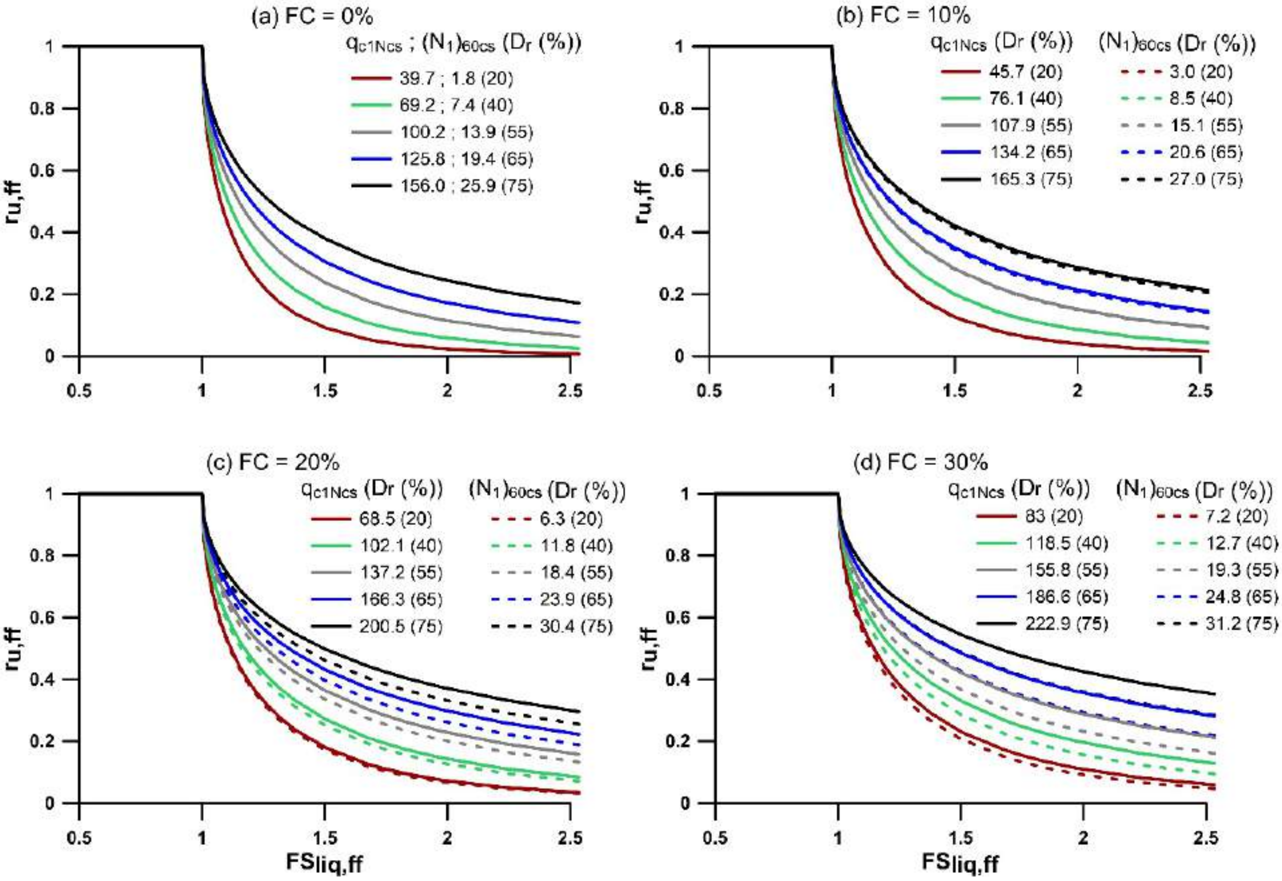

Figure 3-26: Design requirement for the densification techniques.

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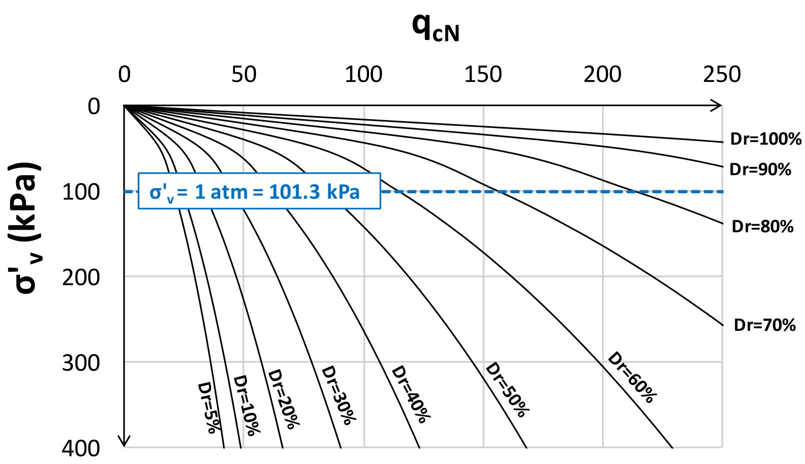

Figure 3-27: CPT tip resistance soil versus density and effective overburden pressure.

The plots of Figure 3-26 drawn for different fine content, allow to fix the required values of FS and r_{uff} and identify the necessary value of q_{1Ncs}. This value is then representative of a profile of CPT resistance (Figure 3-27) with depth or, equivalently of a relative density that becomes the benchmark for ground improvement.

The paramount parameters for densification are the power of the equipment, dependent on the adopted technology, and the grid of settlements that must be fixed in order to achieve the minimum goal deriving from the above calculation. The example of Figure 3-28 shows the results of a field trial where two different equipment of different power were adopted to compact a reclaimed land. The upper plot shows the profile of N_{SPT} values versus depth computed in the centre of a triplet of vibro-compaction columns. As can be seen, the number of SPT blows reduces with the spacing between treatment. In the lower plot, the increased number of blows is computed as a function of the energy released per unit soil volume, and two different trends are seen for adopted equipment being the upper one corresponding to the more powerful equipment.

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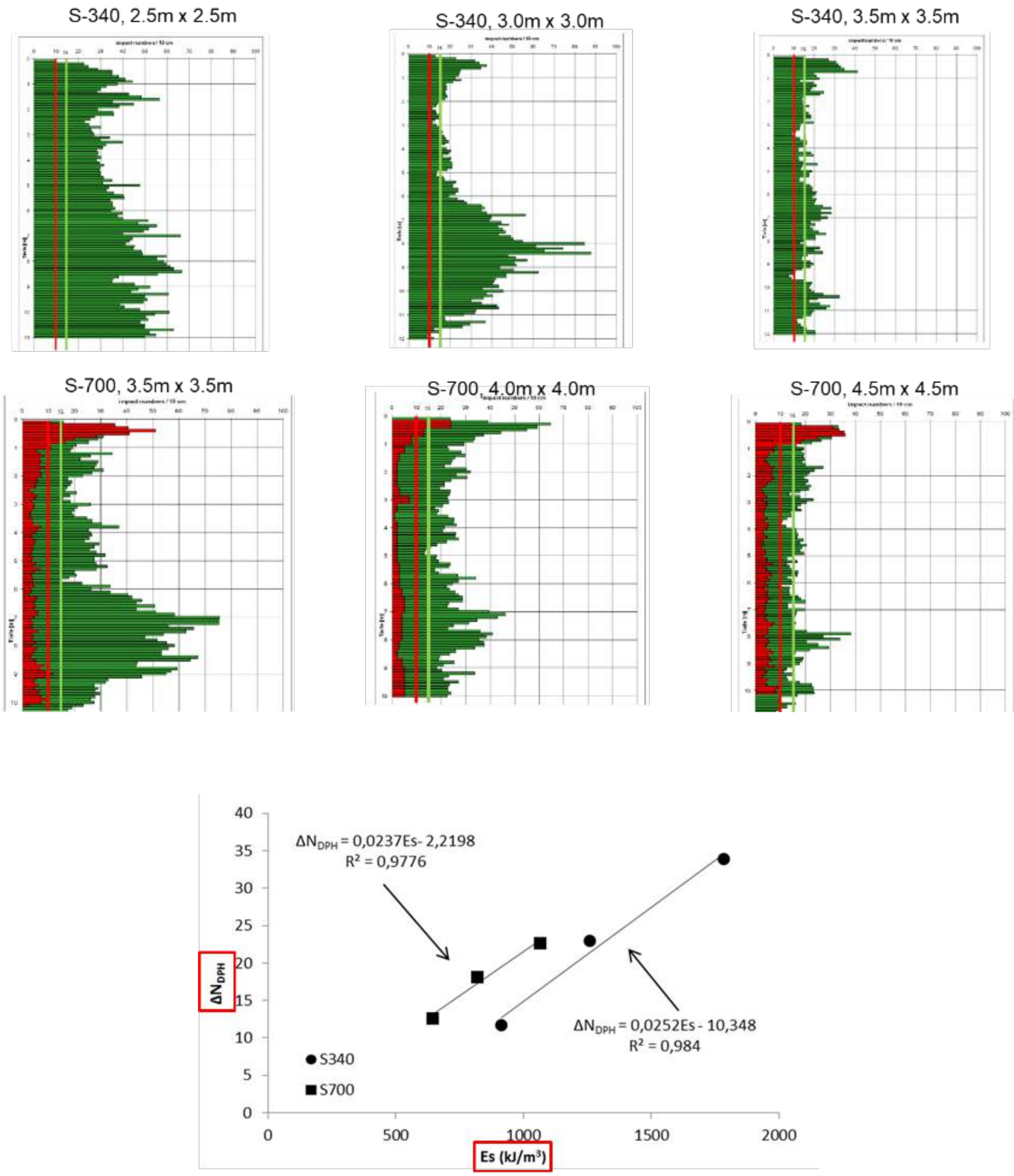

$$E_s = \frac{P \cdot t}{v}$$

Figure 3-28: Example of field trial for the vibro compaction of a reclaimed land.

3.5.2 Low pressure grouting

Cyclic undrained triaxial tests carried out in the present project have shown that nanosilicate grout is able to increase the liquefaction resistance and that the paramount factor to characterize ground improvement is the solid fraction of nanosilicate present in the grout (Salvatore et al., 2019). The nanosilica gel coats the sand grains and fills the interparticle pores preventing volume contraction and bonding of the grain contacts, this latter effect activating with a slower rate. From the applicative viewpoint, for a given soil state, solid fraction must be assigned considering its effect on the CSR-N_{liq} curve, with the principle that looser soils require higher silica fractions. Figure 3-29 shows the cyclic resistance ratios of soils having different density plotted versus number of cycles causing r_u=0.9. The results generally show an improvement of the liquefaction performance for the treated material with respect to the natural sand, but a distinction must be made between looser and relatively denser soil (Figure 3-29.a and b): the relatively denser sand (D_r=0.52÷0.61) initially possesses a fairly good liquefaction resistance (the cyclic stress ratio corresponding to 15 cycles is CSR15=0.22) that increases significantly with 5% nanosilica grout (CSR15=0.30); the looser sand (D_r=0.22÷0.3) possesses a much lower original liquefaction resistance (CSR15=0.12), but the increase obtained with 5% nanosilicate is limited (CSR15=0.15). This amount of nanosilicate (5%) seems to offer a good resistance to the volume contraction of the relatively denser soil but is not able to counteract with the same effectiveness the larger contraction tendency of the looser soil. For this reason, the experiments on the looser material were repeated increasing the solid silica fraction to 10%. This time the increase of liquefaction resistance become immediately evident (CSR15=0.45). From the above experiments it can be argued that the amount of solid silica present in the suspension influences the stiffness and strength of the gel. Considering the mechanisms activated during cyclic loading and the function of the gel, i.e. oppose to the soil pores contraction and bond the grain contacts, the most appropriate dosage should be fixed looking at the design requirements, given by the combination of seismic input and original soil properties. This analysis leads to find an optimal dosage of the product, identifying the dilution level that balances mechanical efficiency and cost effectiveness.

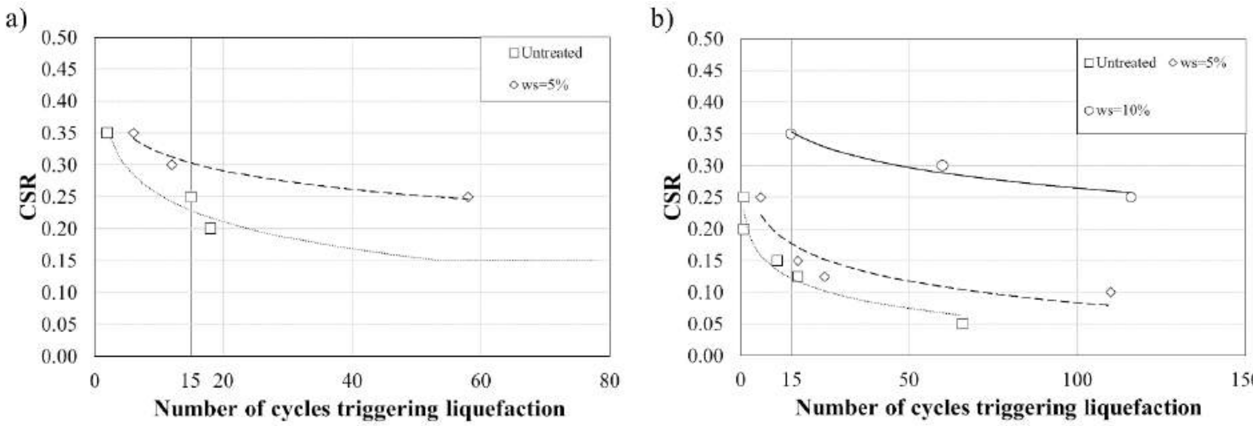

Figure 3-29: Results of the liquefaction triaxial tests reported for relatively dense (a) and relatively loose specimens (b) treated with grout having a variable silica content.

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The second relevant issue for design concerns the execution of treatments. In real scale application, a grid of injection points must be set with an appropriate span to permeate the desired volume with grout. The distance between injection points must be fixed considering the capability of the grout to seep through the soil pores, itself governed by the time change of grout viscosity. The tests performed by Salvatore et al. (2019) highlight the predominant role of the activator, the NaCl solution added to the grout before injection, on the decay of the soil-grout permeability (Figure 3-30). A high fraction may speed up the increase of grout viscosity and solidification at a level that hampers the diffusion of grout, while a low concentration may lead the injected grout to excessively dilute with groundwater, mostly if high hydraulic gradients are present. The most appropriate choice should be set including the change of viscosity into diffusion models.

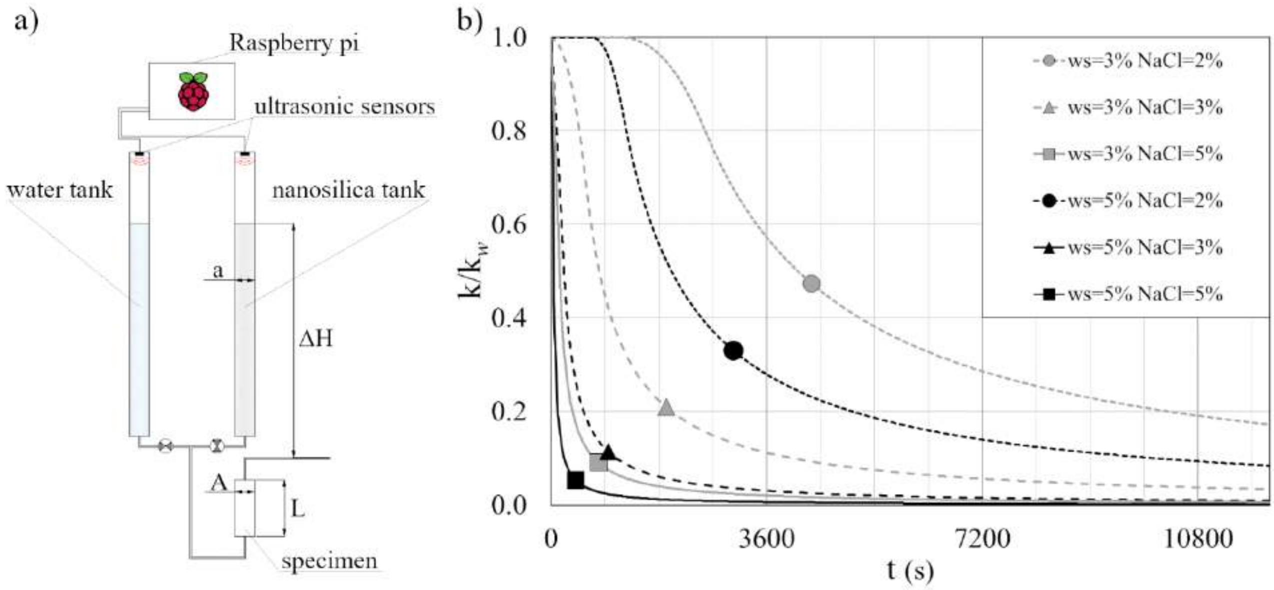

Figure 3-30: Seepage tests of nanosilicate grout: testing equipment (a); time variation of the soil-grout permeability (b).

The effective achievement of results can be assessed with Quality Control, i.e. the assessment of ground improvement efficacy. Sonic technique implemented in the laboratory with bender element tests has proven its efficacy, showing that shear wave velocities of grouted soil increase at variable rates with time. The increase is relatively fast in the beginning (0-9 days) but continues also for longer periods (up to 24 days in the present analysis). The sonic technique represents a suitable tool to assess the effectiveness of ground improvement on site.

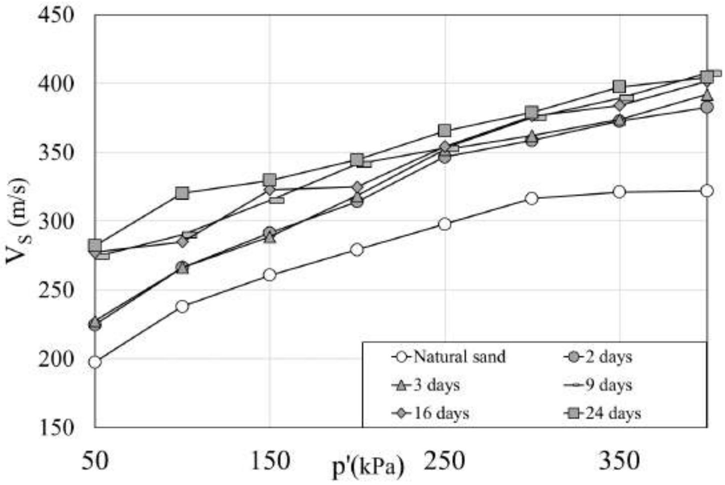

Figure 3-31: Shear wave velocity versus mean effective stress on a natural sand (D_r≈30%) and on a sample at similar relative density treated with nanosilicate grouting (w_s=5%) at different curing time.

3.5.3 Drainage

Drainage aims to reduce the build-up of pore water pressure. This mitigation action is typically obtained by the insertion of vertical drains (called for this specific application “earthquake drains”) and is one of the most popular and efficient ways to protect existing structures (e.g. Harada et al., 2006). Generally, the use of vertical drains poses no technological problems, being made with current tools like gravel columns, small steel cylindric drains and simple tape drains. The insertion of drains into the liquefiable soil modifies the hydraulic boundary conditions. The drains can be considered as surfaces having constantly zero excess pore pressure and, if properly spaced, accelerate the consolidation process during seismic shaking, with a beneficial reduction of soil susceptibility to liquefaction. Some design methods based on the solution of radial consolidation are already available in literature to assign drains spacing (Bouckovalas et al., 2009; Seed and Booker, 1976). However, since the current technology considers only vertical drains, the application of this technique to mitigate liquefaction risk for existing buildings implies that they cannot be placed below the structures. In such a way, drainage is enhanced around the structure to protect, but not in the volume of soil underneath it, on which the structure is directly resting. Because of these geometrical constraints, the result is a reduced effectiveness of the technology in the built environment. A possible solution to this technological limitation may be obtained by adopting directional drilling (Allouche et al., 2000) to place horizontal or sub-horizontal drains, made of micro-perforated pipes, directly underneath existing structures. This is a very promising evolution, whose use should not pose critical installation problems, at least as the horizontal drains are shallow (not deeper than 10 m), and their diameter is not very large (not more than 30 cm). Obviously, it is possible to deploy horizontal drains in multiple rows in either a square grid or in a staggered layout, as done for vertical drains.

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3.5.3.1 Vertical drains

As previously mentioned, the process of consolidation for vertical drains with pore pressure build-up induced by earthquake in liquefiable sand is due to Seed & Booker (1976) and after modified by Bouckovalas et al. (2009). They solved the problem of the radial consolidation with an additional excess pore water pressure build-up term u_g that is the pore pressure increment generated during shaking and ∂N is the number of cycles taking place in the time interval, ∂t (Equation 3-38). Within this framework, that is by far the most popular one adopted to study the dynamic consolidation of soils, different choices can be done to quantify the cyclic pore pressure build up ($\partial u_g/\partial N$). The latter term was defined as a function of the ratio N_{eq}/N_l between the seismic action quantified through the number of equivalent cycles, N_{eq} (Green & Terri, 2005; Seed & Idriss, 1971), and the number of cycles required to cause liquefaction, N_L(Kramer and Wang, 2015) (Equation 3-39).

$$\frac{k}{\gamma_w m_v}\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\left(\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}\right) \qquad \text{Equation 3-38}$$

This term is a function of the excess pore pressure build-up curve (Bouckovalas et al., 2009), (Equation 3-39).

$$\frac{\partial u_g}{\partial N}=\frac{\sigma_0^{'}}{\pi A N_{Ll}}\frac{1}{\left(\frac{t}{t_d}\frac{N_{eq}}{N_L}\right)^{1-\frac{1}{2A}}cos\left(\frac{\pi}{2}r_u\right)}\ ;\ \frac{\partial N}{\partial t}=\frac{N_{eq}}{t_d} \qquad \text{Equation 3-39}$$

where σ_0' is the initial vertical effective stress, t is the time variable, t_d is the significant duration of seismic shaking (Trifunac & Brady, 1975), r_u is the excess pore pressure ratio ($u_g/\sigma_0^{'}$) and A is a parameter that affect the shape of build-up curve (Seed et al., 1975).

In dimensionless terms the general equation becomes:

$$T_{ad}\left(\frac{\partial^2 r_u}{\partial\left(\frac{r}{d}\right)^2}+\frac{1}{\frac{r}{a}}\frac{\partial^2 r_u}{\partial\left(\frac{r}{d}\right)^2}\right)=\frac{\partial r_u}{\partial\left(\frac{t}{t_d}\right)}-\frac{N_{eq}}{\pi A N_L}\frac{1}{\left(\frac{N_{eq}}{N_L}\frac{t}{t_d}\right)^{1-\frac{1}{2A}}cos\left(\frac{\pi}{2}r_u\right)} \qquad \text{Equation 3-40}$$

The time factor T_{ad} (Equation 3-41) is a function of significant earthquake duration t_d, permeability coefficient k, diameter of drain d, volumetric compressibility $m_{v,3}$ and water unit weight γ_w. the pore pressure build-up term is a function of the equivalent cyclic number of the earthquake (N_{eq}) and of the cyclic number that leads to liquefaction (N_L).

$$T_{ad}=\frac{t_d k}{d^2\, m_{v,3}\,\gamma_w} \qquad \text{Equation 3-41}$$

The numerical solution of the problem for an infinite disposition of vertical drains, in parametric way, leads to define design charts to determine the spacing between vertical drains (Figure 3-32).

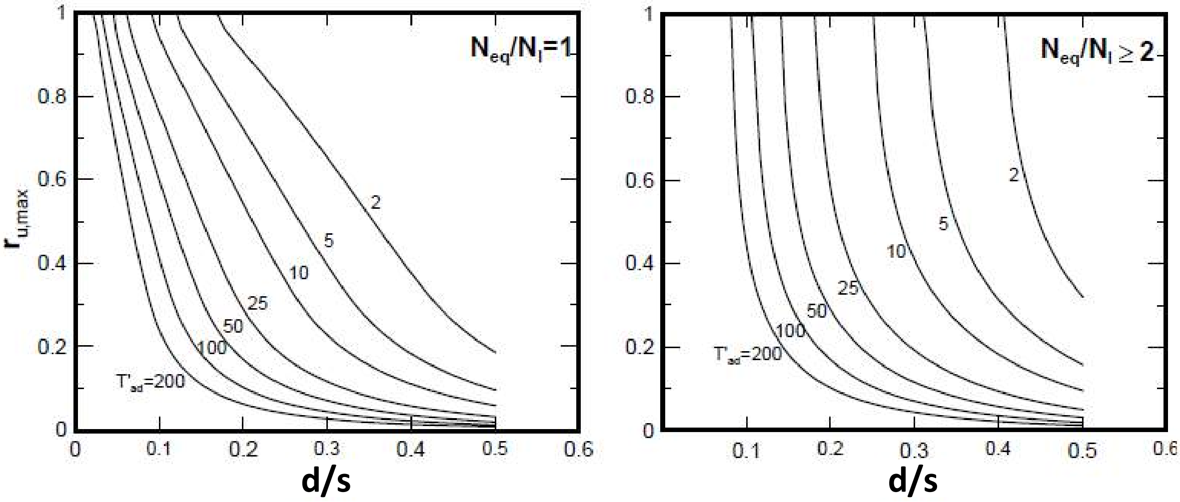

Figure 3-32: Design charts for vertical drains in liquefiable soil.

The application of the design approach for vertical drains requires different steps (Figure 3-33). The first step is the calculation of the ratio between the number of equivalent cycles N_{eq} and the number of cycles that leads to liquefaction N_L to determine the design chart. The procedure suggested by Biondi et al. (2012) can be applied to calculate N_{eq}, whereas N_L is the number of cycles to liquefaction taken at the same level of CSR due to the earthquake.
The curve in the chart is identified by the value of T_{ad}, and the choice of the design maximum value of excess pore pressure ratio $r_{u,max}$ permits to determine the ratio between the diameter of the vertical drain and the spacing between them (d/s).

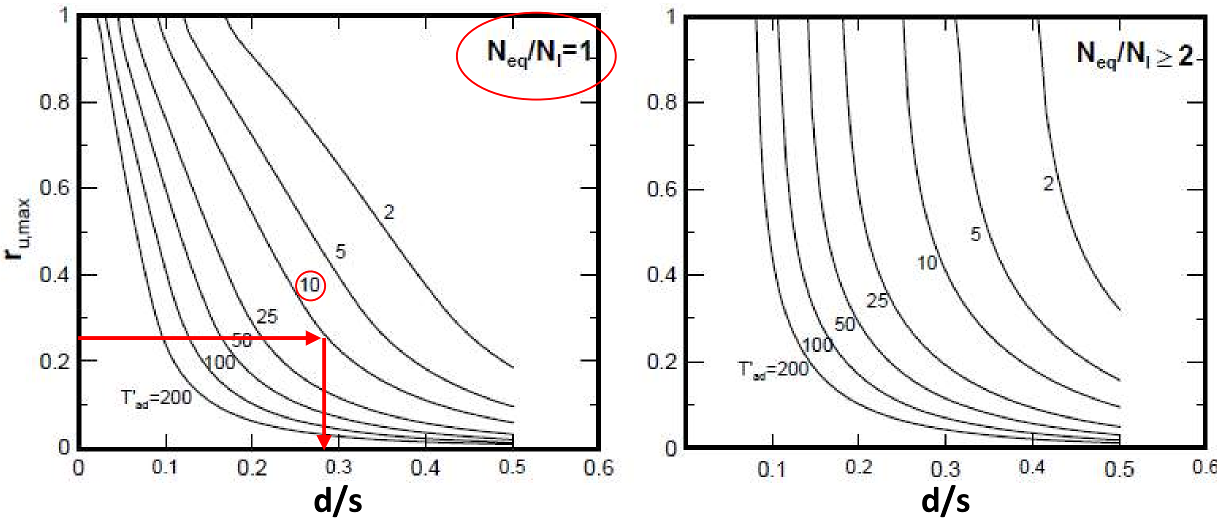

Figure 3-33: Procedure to apply for design charts.

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3.5.3.2 Horizontal drains

As was done for vertical drains, the development of excess pore water pressure during earthquake can be studied with an uncoupled approach, by introducing a build-up function for excess pore water pressure. Once the accumulation term is added, the consolidation equation can be solved. In bi-dimensional conditions and in the hypothesis of Terzaghi-Rendulic (Rendulic, 1936), it can be written as:

$$\frac{k}{\gamma_w m_{v,3}}\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)=\left(\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}\right) \quad \text{Equation 3-42}$$

In dimensionless term the equation becomes:

$$T_{ad}\left(\frac{\partial^2 r_u}{\partial\left(\frac{x}{d}\right)^2}+\frac{\partial^2 r_u}{\partial\left(\frac{y}{d}\right)^2}\right)=\frac{\partial r_u}{\partial\left(\frac{t}{t_d}\right)}-\frac{N_{eq}}{\pi A N_l}\frac{1}{\left(\frac{t}{t_d}\frac{N_{eq}}{N_L}\right)^{1-\frac{1}{2A}} cos\left(\frac{\pi}{2}r_u\right)} \quad \text{Equation 3-43}$$

The geometric layout analysed in this study is presented in Figure 3-34; a drainage system made of three rows of drains in a staggered disposition (α=60°) is assumed. The shallowest row is located at depth H' from the ground surface. Two symmetrical vertical planes (as shown in Figure 3-34) constitute the vertical impervious boundaries of the domain, except for three segments representing the drains characterized by zero excess pore pressure condition. The lower boundary was modelled as impervious at a distance equal to 2s/d from the last row of drains, thus minimizing the effect within the domain of interest. Indeed, the solution is given in a smaller volume in which the effect of drainage is significant, whose extent is up to 0.5 s/d underneath the last row of drains.

The upper boundary hydraulic condition can be either pervious or not. In fact, earthquake can induce liquefaction at depth up to about 20 m, thus it is also possible that a less permeable layer (made of silt or clay) can overly liquefiable one. This affects the hydraulic boundary condition and, consequently, the pore pressure build-up in the shallowest part of liquefiable soil. The importance of the upper part of soil profile is related to the presence of structures above, which may suffer larger vertical displacements, and subsequent damages, in case of an excessive pore water pressure build-up. Therefore, two limit conditions were considered for the upper boundary, a pervious boundary (BC1) and an impervious one (BC2), which bound all the intermediate cases.

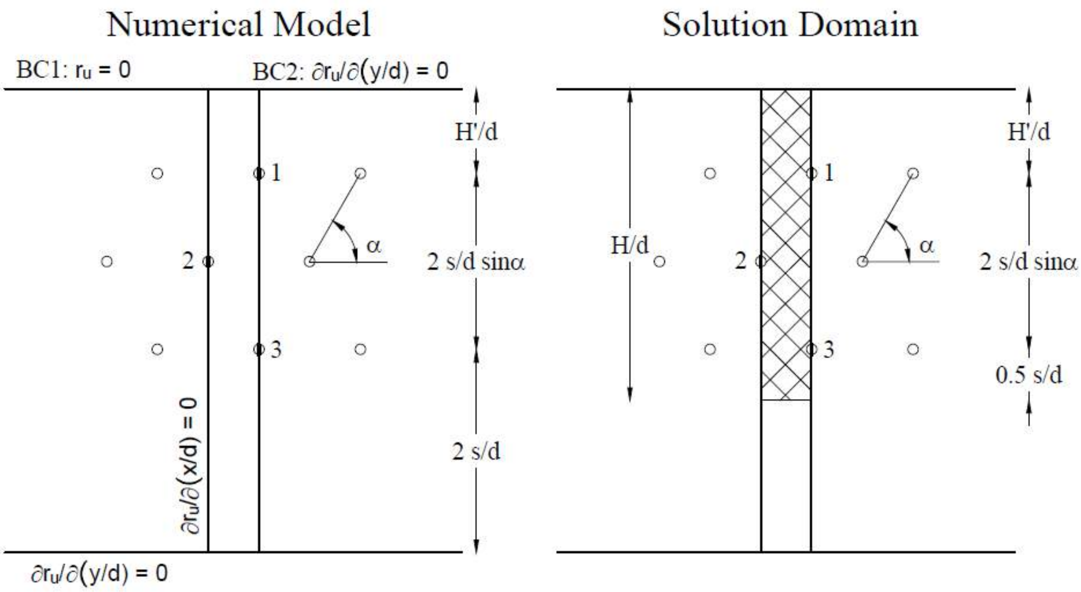

Figure 3-34: Numerical and solution domains of the geometrical system made of three rows of drains in a staggered disposition.

The charts represent the excess pore pressure ratio, r_u, in the solution domain for different sets of parameters. For each instant t, the mean and maximum excess pore pressure ratios in the solution domain were evaluated. Because of the seepage induced by the hydraulic gradients around drains, the worst conditions are not necessarily attained at the end of the shaking, being possible to observe them during the shaking. Therefore, the maximum values in time of the mean and maximum excess pore pressure ratios calculated over the whole domain, $r_{u,mean}$ and $r_{u,max}$, were considered in the design charts. Each one of them is related to specific values of the ratios H'/d and N_{eq}/N_L; each curve refers to a value of T_{ad} and represents the excess pore pressure ratio (as explained before) for different values of s/d.

For the horizontal drains system considered before the upper boundary was represented by a pervious surface, this condition allows the vertical drainage through this surface with a consequent reduction of the excess pore water pressure. In common practice it is possible the presence of a low permeability soil upon the liquefiable one. In this case the drainage through the upper surface is not possible and a larger excess pore water pressure is achieved.

Parametric analyses have been performed and their results summarized in design charts from Figure 3-35 to Figure 3-38. The depth of the first row of horizontal drains represents a new variable that leads to a great number of design charts. Each chart is identified by the dimensionless depth of first row (H'/d) and the ratio N_{eq}/N_L.

CSR is a function of the depth and by consequence, also N_L is a function of depth; thus, a representative depth has to be assumed to assign N_{eq}/N_L. In a design procedure, the depth would be set at the middle of the layer that has to be treated.

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Below are shown the design chart for different values of these variables and for the two different upper boundary conditions (BC1 and BC2).

The application of the design approach for horizontal drains has only one difference compared to the vertical one: the individuation of the design chart requires also the choice of the depth of the first row of drains.

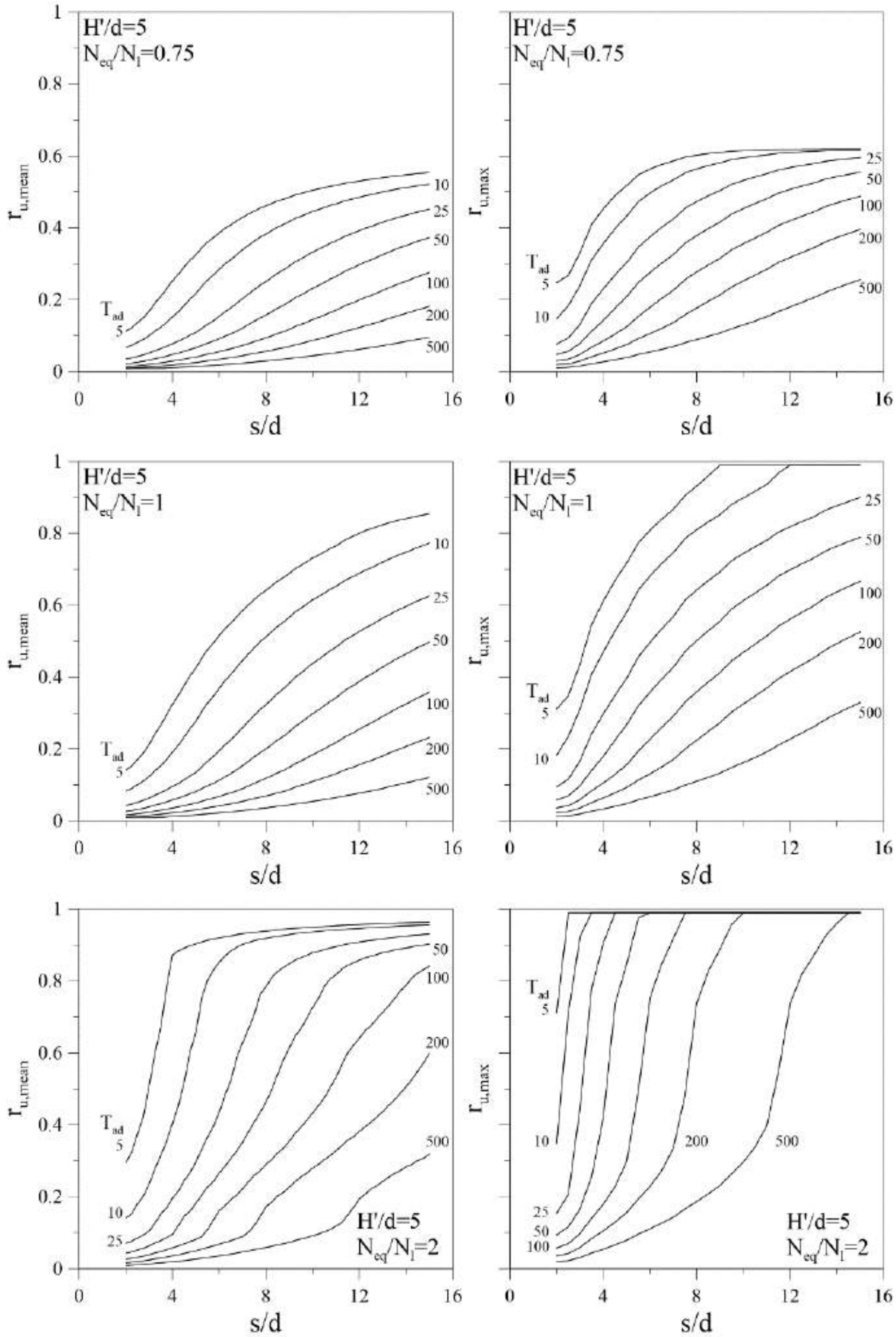

Figure 3-35: r_u charts for H'/d=5 BC1. (Fasano et al, 2019).

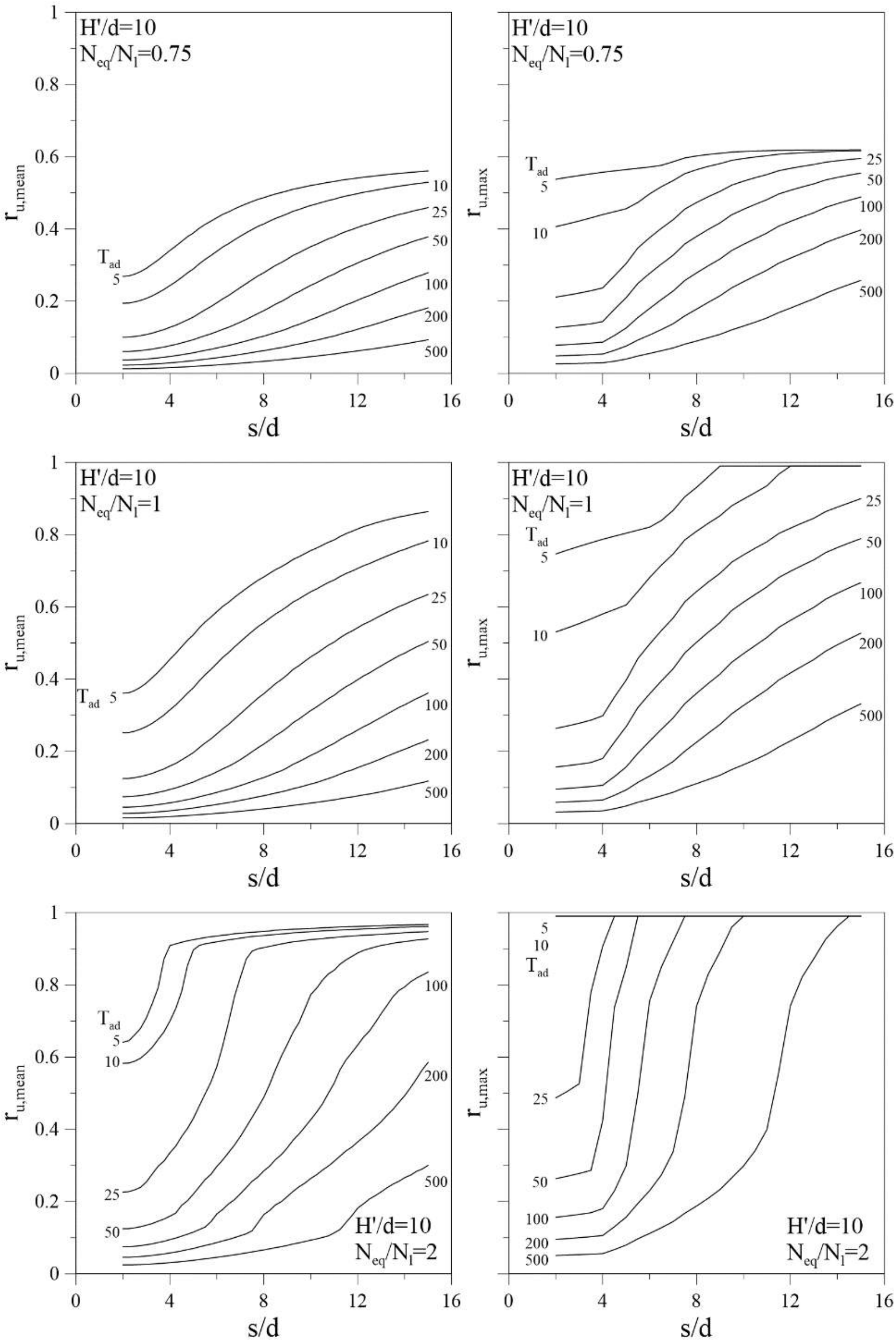

Figure 3-36: r_u charts for H'/d=10 BC1. (Fasano et al, 2019).

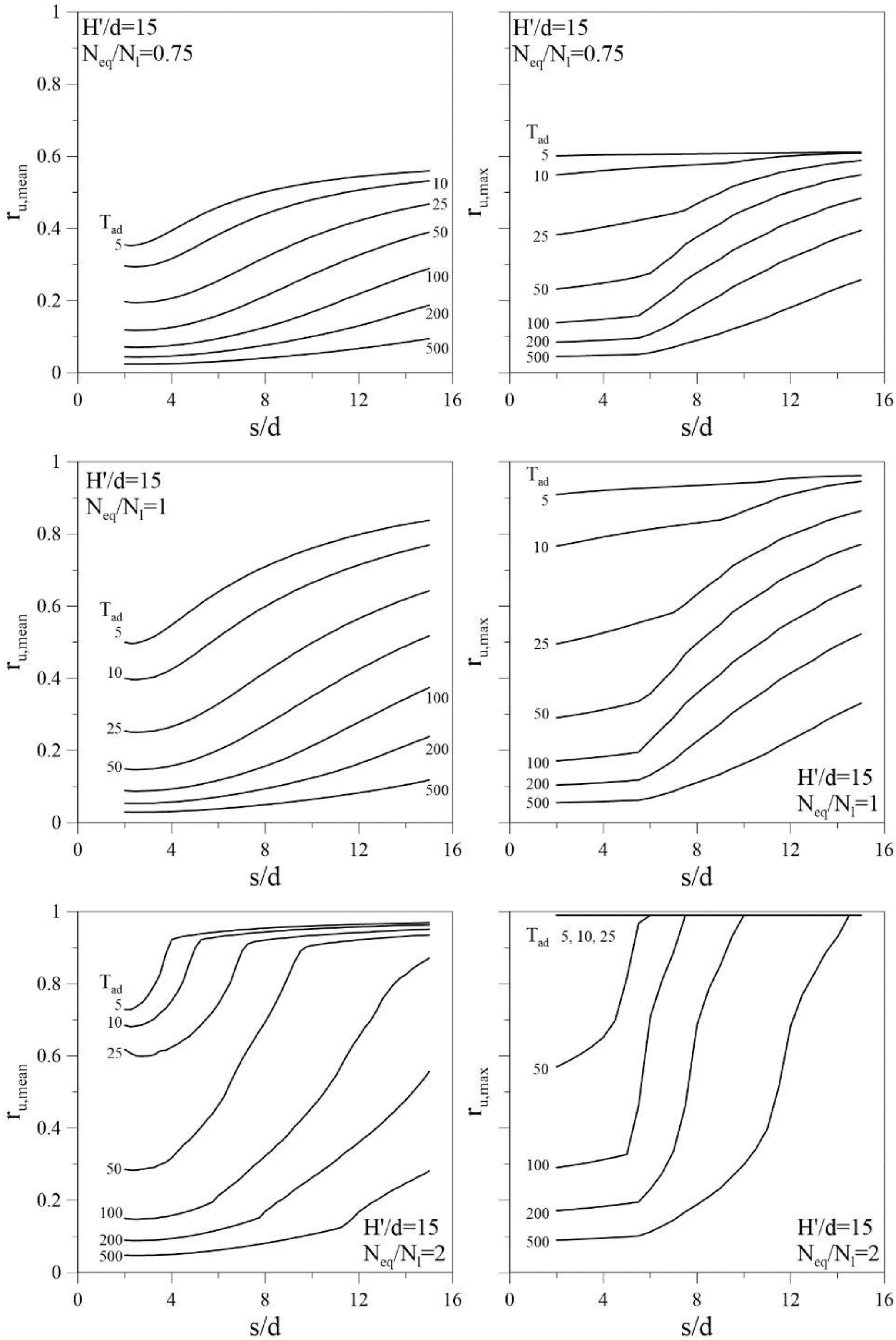

Figure 3-37: r_u charts for H'/d=15 BC1. (Fasano et al, 2019).

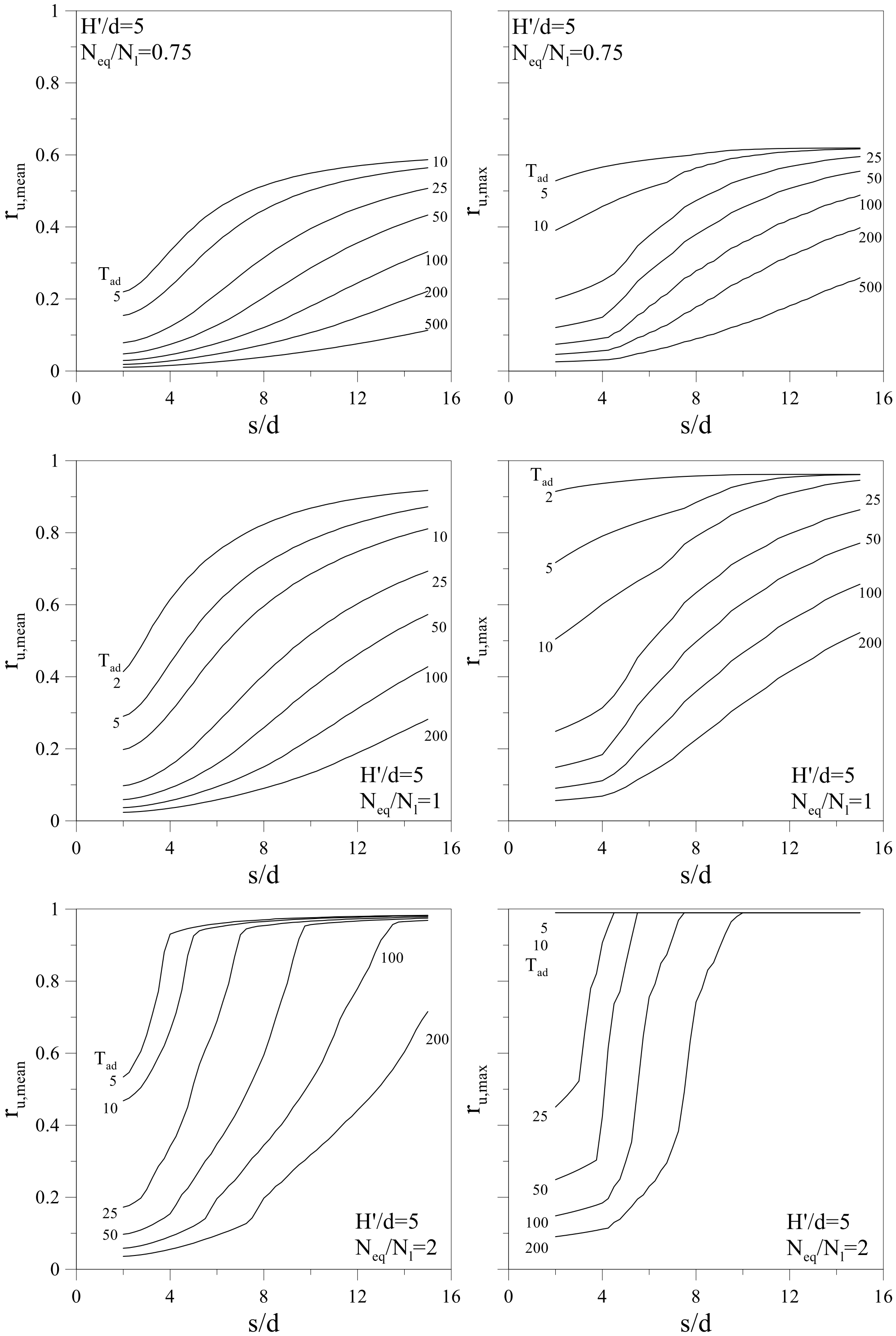

Figure 3-38: r_u charts for H'/d=5 BC1. (De Sarno et al., 2019).

3.5.4 Induced partial saturation

As well-known partial saturation increases the liquefaction resistance because of the very low volumetric stiffness of the gaseous phase, as shown by several researchers (e.g. Chaney, 1978; Yoshimi et al., 1989; Ishihara et al., 2002; Yegian, 2007; Tsukamoto et al., 2014; Wang et al., 2016; Mele et al., 2018). In unsaturated soils it is still possible to define an effective stress. Among the several proposals, the most used one to this aim is that proposed by Bishop &Blight (1963):

$$\sigma'_{un} = (\sigma - u_a) + \chi \cdot (u_a - u_w) \qquad \text{Equation 3-44}$$

where:
σ is the total stress and u_a, u_w and χ are respectively the pore air pressure, the pore water pressure and the material parameter accounting for the effect of the degree of saturation. The term (σ-u_a) is called “net stress”, while (u_a-u_w) is the “matric suction” (s). The parameter χ is assumed equal to the degree of saturation Sr (≤100%), according to Wheeler et al. (2003) and Gallipoli et al. (2003).

The design goal of IPS aims to calculate the degree of saturation (Sr) to be applied in-situ. To pursue this goal, Mele & Flora (2019) provided two possible design approaches based on an energetic model (Mele et al., 2018. Mele et al. (2018) showed that during undrained cyclic triaxial loading on several kinds of sand in same state conditions, regardless of the applied CSR, ε_v increases until to reach, for σ'_{un}=σ, the same final value ($\varepsilon_{v,fin}$). $\varepsilon_{v,fin}$ depends only on the initial state, defined by the degree of saturation (S_{r0}), the void ratio (e_0) and the effective stress (σ'_{un}). The value of $\varepsilon_{v,fin}$ can be easily calculated by the following equation (Okamura & Soga, 2006; Mele et al., 2018):

$$\varepsilon_{v,fin} = \frac{e_0}{1+e_0} \cdot (1 - S_{r0}) \cdot \left(1 - \frac{u_{a,0}}{\sigma}\right) \qquad \text{Equation 3-45}$$

where e_0, S_{r0} and $u_{a,0}$ are the void ratio, degree of saturation and pore air pressure, respectively, at the beginning of the cyclic phase.

Equation 3-45 is based on the hypothesis of process isothermal and considering air as an ideal gas. Moreover, a unique curve can be found in a non-dimensional plane ($\sigma'_{un}/\sigma'_{un,0}$:$\varepsilon_v/\varepsilon_{v,fin}$, Figure 3-39), having the expression:

$$\frac{\sigma'_{un}}{\sigma'_{un,0}} = 1 - \left(\frac{\varepsilon_v}{\varepsilon_{v,fin}}\right)^{1.7} \qquad \text{Equation 3-46}$$

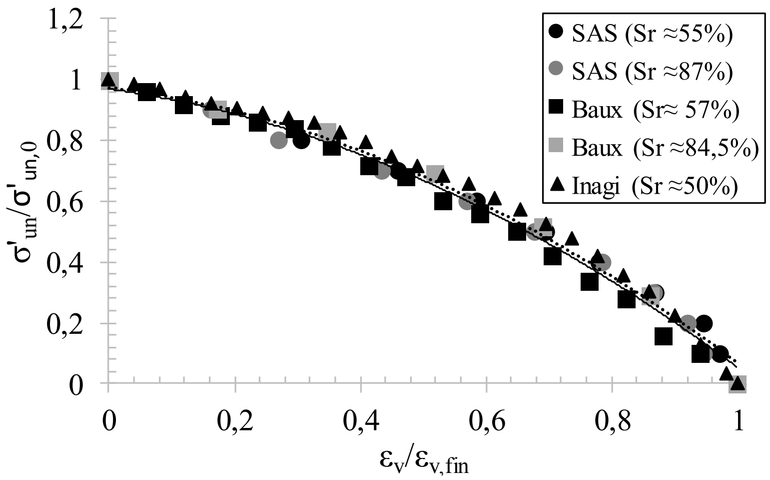

Figure 3-39: Dimensionless effective stress vs. dimensionless volumetric strain for some of the tests reported by Mele et al. (2018).

The main principle on which the energetic model is based is that the total specific energy of deformation E_{tot} needed to reach liquefaction can be seen as the sum of two components:

$$E_{tot,liq} = E_{v,liq} + E_{s,liq} \qquad \text{Equation 3-47}$$

where $E_{v,liq}$ is the volumetric specific energy and $E_{s,liq}$ is the deviatoric specific energy to reach liquefaction.

The volumetric specific energy can be seen as the sum of three components (Mele et al., 2018):

$$E_{v,liq} = E_{v,sk,liq} + E_{w,liq} + E_{air,liq} \qquad \text{Equation 3-48}$$

$E_{v,sk,liq}$, $E_{w,liq}$ and $E_{air,liq}$ represent the specific work done respectively to cause the deformation of the soil skeleton, the flow of water and the flow of air into the pores network. They can be expressed as:

$$E_{v,sk,liq} = \int_0^{\varepsilon_{v,liq}} [(\sigma - u_a) + sS_r] \cdot d\varepsilon_v \qquad \text{Equation 3-49}$$

$$E_{w,liq} = -\int_{Sr0}^{Sr,liq} \frac{e(S_r)}{1+e(S_r)} s(S_r) \cdot dS_r \qquad \text{Equation 3-50}$$

$$E_{air,liq} = \frac{e_0}{1+e_0}(1 - S_{r,0}) u_{a,liq} d(\ln \rho_{a,liq}) \qquad \text{Equation 3-51}$$

$E_{v,sk,liq}$ depends on the stress state (σ'_{un}) and on the initial void ratio e_0 ($E_{v,sk,liq} = f(\sigma'(S_r), e_0)$), while it depends neither on CSR nor on N_{liq}. Obviously, $E_{v,sk,liq}$=0 for undrained tests on saturated soils. The integral of eq.6 represents the area of the average curve σ'_{un}-ε_v for a specific soil state, which can be achieved by Equation

3-46 known $\sigma'_{un,0}$ and $\varepsilon_{v,fin}$. The integration extremes for the volumetric strains have to be assigned to calculate $E_{v,liq}$. These are 0 and $\varepsilon_{v,liq}$, respectively corresponding to the effective stresses (Bishop's notation) $\sigma'_{un,0}$ and $\sigma'_{un,liq}$. The latter is the value of the effective stress at liquefaction and is not nil because of the conventional definition of liquefaction (ε_{DA}=5%). It can be calculated as a function of the initial degree of saturation S_{r0} using the following equation:

$$\frac{\sigma'_{un,liq}}{\sigma'_{un,0}} = -2 \cdot 10^{-4} \cdot S_{r0}^2 + 2 \cdot 10^{-2} \cdot\ S_{r0} + 0.1$$ Equation 3-52

which is the best fitting curve of the experimental data presented by Mele et al. (2018) (Figure 3-40).

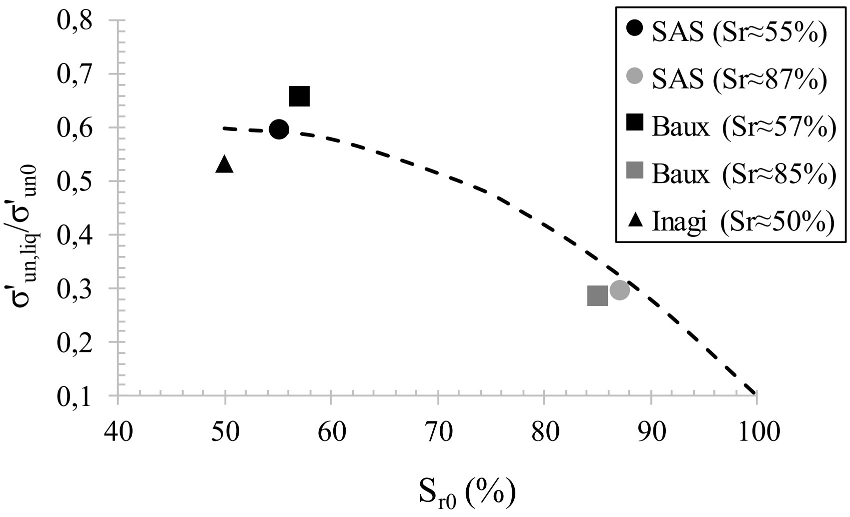

Figure 3-40: Experimental values of $\sigma'_{un,liq}/\sigma'_{un,0}$ and S_{r0} (Mele et al, 2018), along with a best fitting curve (Equation 3-52).

$E_{w,liq}$ is the specific volumetric energy of water and it is due to the change of water content. $E_{air,liq}$ describes the effect of pressure variation in the gas phase. Once the volumetric components have been defined, it is necessary to quantify the specific deviatoric energy of the soil skeleton spent to liquefaction, $E_{s,liq}$, connected to distorsional strains ε_s. From a physical point of view, $E_{s,liq}$ is the sum of the areas of all the cycles in the ε_s:q plane (D_{cyc} in Figure 3-41 for a single cycle) up to liquefaction (defined in terms of strains). Formally, for each cycle the energy is defined as:

$$E_{s,sk,liq} = \sum_{N_{cyc}=1}^{N_{cyc}=N_{liq}} \int\int_{D_{cyc}} dq \cdot d\varepsilon_s$$ Equation 3-53

$E_{s,liq}$ depends on soil properties, soil state and cyclic stress amplitude CSR.

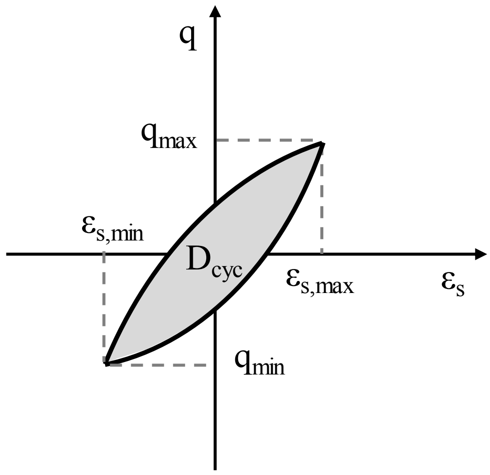

Figure 3-41: Definition of the specific deviatoric energy Es,sk for a single cycle in the q:εs plane (Mele et al., 2018).

With the main aim to predict liquefaction resistance of unsaturated sandy soils, two simple approaches (Mele & Flora, 2019) have been discussed.

Approach 1

This approach is based on the concept of specific volumetric energy to liquefaction, $E_{v,liq}$ as a synthetic state variable ruling the increment of liquefaction resistance of unsaturated sands. Mele and Flora (2019) showed the relationship between ΔCRR^{ctx} (or ΔCRR^{css}) defined as $\Delta CRR^{ctx}=CRR_{un}^{ctx}-CRR_{s}^{ctx}$ (or $\Delta CRR^{css}=CRR_{un}^{css}-CRR_{s}^{css}$) for N_{liq} = 15 and $E_{v,liq}$ for the three tested sands, where CRR_{un}^{CSS} and CRR_{s}^{CSS} have been obtained by Castro's correlation (1975):

$$CRR^{css} = c_r \cdot CRR^{ctx}$$
$$c_r = \frac{2\cdot(1+2K_0)}{3\sqrt{3}} \quad \text{Equation 3-54}$$

where k_0 is the coefficient of earth pressure at rest, evaluated as K_0=1-sinφ_p, where φ_p is the peak friction angle.

Based on the experimental results reported in Figure 3-42, the relationships between $E_{v,liq}$ and $\Delta CRR_{,Nliq}^{ctx}$ and $\Delta CRR_{,Nliq}^{css}$ (for N_{liq}=15) can be expressed as:

$$\Delta CRR_{N_{liq}}^{ctx} = -105.7\cdot\left(\frac{E_{v.liq}}{p_a}\right)^2 + 10.16\cdot\frac{E_{v,liq}}{p_a} \quad \text{Equation 3-55}$$

$$\Delta CRR_{N_{liq}}^{css} = -89.6\cdot\left(\frac{E_{v.liq}}{p_a}\right)^2 + 7.81\cdot\frac{E_{v,liq}}{p_a} \quad \text{Equation 3-56}$$

where p_a is the atmospheric pressure.

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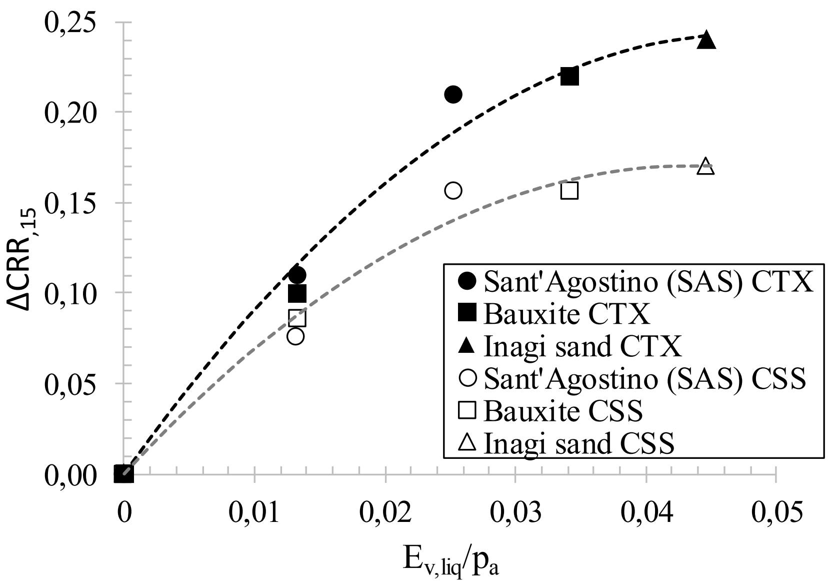

Figure 3-42: Ratio between unsaturated and saturated liquefaction resistance at Ncyc=15 (ΔCRR,15ctx and ΔCRR,15css) versus $E_{v,liq}/p_a$ (Mele & Flora, 2019).

Approach 2

In this approach the deviatoric component of energy is also evaluated (Equation 3-53). Mele & Flora (2019) showed that a unique curve may be obtained by plotting the experimental data in the normalized plan in Figure 3-43 ($E_{s,liq}$ vs ($CRR^{ctx}\cdot(1-5\cdot E_{v,liq}/p_a)^{10}$)). The equation has been provided:

$$E_{s,liq} = 0.297 \cdot p_a \cdot e^{-16.7 \cdot CRR^{ctx} \cdot \left(1-5 \cdot \frac{E_{v,liq}}{p_a}\right)^{10}} \qquad \text{Equation 3-57}$$

Similarly, considering the cyclic resistance ratios in simple shear conditions a best fitting relationship is found as:

$$E_{s,liq} = 0.300 \cdot p_a \cdot e^{-23.7 \cdot CRR^{css} \cdot \left(1-5 \cdot \frac{E_{v,liq}}{p_a}\right)^{10}} \qquad \text{Equation 3-58}$$

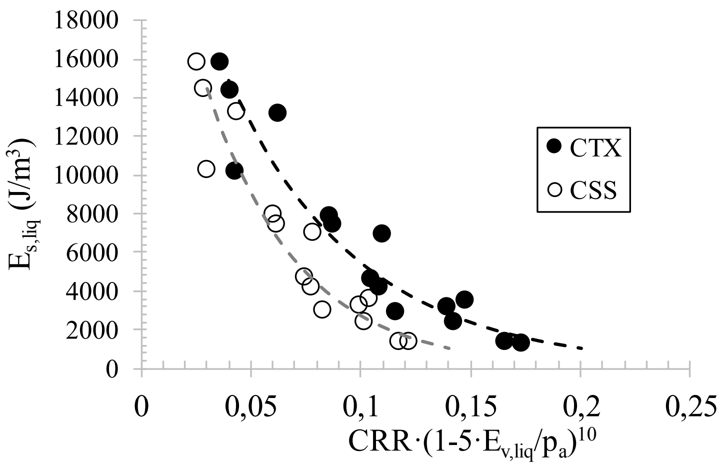

Figure 3-43: Cyclic triaxial and corrected triaxial data (Castro correlation) in the plane CRR·(1-5·$E_{v,liq}$)^{10} vs $E_{s,liq}$ (Mele and Flora, 2019).

Moreover, a relationship between $CRR^{ctx}/(1+E_{tot,liq}/p_a)^6$ and N_{liq} have been reported in Figure 3-44.

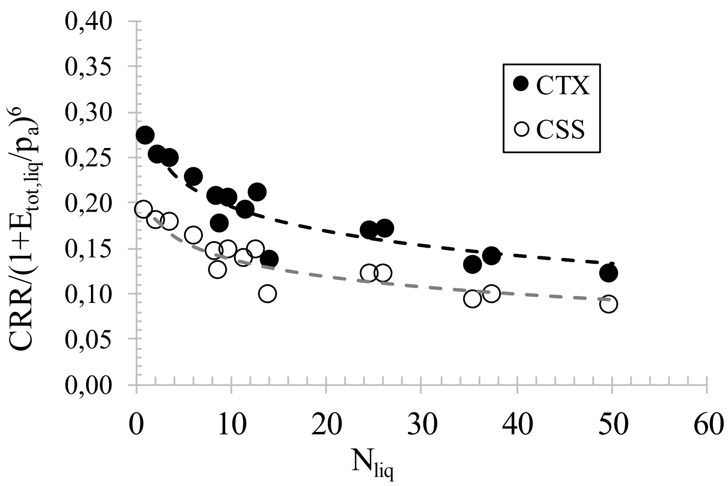

Figure 3-44: Normalized cyclic resistance curves for cyclic triaxial and corrected data (Castro correlation) (Mele & Flora, 2019).

The best fitting of the experimental results in Figure 3-44 is:

$$\frac{CRR^{ctx}}{\left(1+\frac{E_{tot,liq}}{p_a}\right)^6} = -0.039 \cdot \ln\left(N_{liq}\right) + 0.285 \qquad \text{Equation 3-59}$$

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Which can be transformed in simple shear conditions (Equation 3-55 and Equation 3-56) as:

$$\frac{CRR^{css}}{\left(1+\frac{E_{tot,liq}}{p_a}\right)^6} = -0.028 \cdot \ln(N_{liq}) + 0.202$$ Equation 3-60

In the design of IPS, the goal is to find what degree of saturation S_r is needed to guarantee for the structures to protect a satisfactory performance with reference to serviceability and limit conditions with the desired safety margins, with reference to any kind of mechanism related to liquefaction (Bray & Macedo, 2017). In particular, two scenarios may be foreseen: one in which the risk is linked to the attainment of liquefaction (i.e. a temporary but total loss of stiffness and strength of the liquefied soil), and one in which the pore pressure build up may trigger limit states in the structures (e.g. bearing capacity failure or excessive settlements) before liquefaction is reached. In the first case, an increase of CRR^{css} for the given value of N_{eq} (which is the number of cycles corresponding to the design seismic action) is needed. In the second case (which may refer to situations in which the safety margins against liquefaction may be sufficient in saturated conditions), it is simply asked to have lower pore pressures for $N=N_{eq}$. Formally, this may be seen as the need to increase, for the given value of CSR, the value of N_{liq} to a higher value N_{liq}*. Both scenarios ask for an increase of soil capacity via IPS to cope with seismic demand, and the two procedures depicted in Figure 3-45 can be alternatively considered to this aim.

- *Increase CRR*

The first procedure, on the left side of Figure 3-45, refers to the need of increasing the safety factor against liquefaction. This means that the original safety margins are known (i.e., the saturated CRR^{css}-N_{liq} curve is known). In this case, it is trivial to know what increment of liquefaction resistance (ΔCRR^{css}) is needed once the desired safety margins are given, and therefore the previously proposed approach 1 is best suited as design tool. In fact, by knowing ΔCRR^{css} it is possible to calculate $E_{v,liq}$ (Equation 3-56). For high values of S_r (as will generally be the case for IPS), the contribution of $E_{w,liq}$ is negligible, being null the suction. Therefore, $E_{v,liq}$ can be considered as the sum of two components ($E_{v,sk,liq}$ and $E_{air,liq}$). Through an iterative procedure, the design value of S_r (S_{rd}) can be finally calculated.

- *Increase N_{liq}*

In this case, the seismic action (CSR) leads for $N=N_{eq}$ to excessive pore pressures (but not to liquefaction). There is the need to reduce such pore pressures, regardless of the original safety margins against liquefaction. The saturated liquefaction resistance curve is not a necessary design tool in this case, being the design goal to increase N_{liq} till N_{liq}*.

In this case, approach 2 is best suited as design tool, as depicted on the right side of Figure 3-45: once N_{liq}* has been assigned, Equation 3-60 allows to know the ratio $CRR^{css}/(1+E_{tot,liq}/p_a)^6$ (considering in this case CRR^{css}=CSR). The total specific energy to liquefaction $E_{tot,liq}$ is the sum of two components $E_{v,liq}$ and $E_{s,liq}$, where $E_{s,liq}$ can be computed as a function of CSR and $E_{v,liq}$ (Equation 3-58).

$E_{tot,liq}$ is therefore given by:

$$E_{tot,liq} = E_{v,liq} + 0.300 \cdot p_a \cdot e^{-23.7 \cdot CRR^{css} \cdot \left(1-5 \cdot \frac{E_{v,liq}}{p_a}\right)^{10}} \qquad \text{Equation 3-61}$$

Using Equation 3-61, the design value $S_{r,d}$ can be calculated as done with approach 1 with a simple iterative procedure.

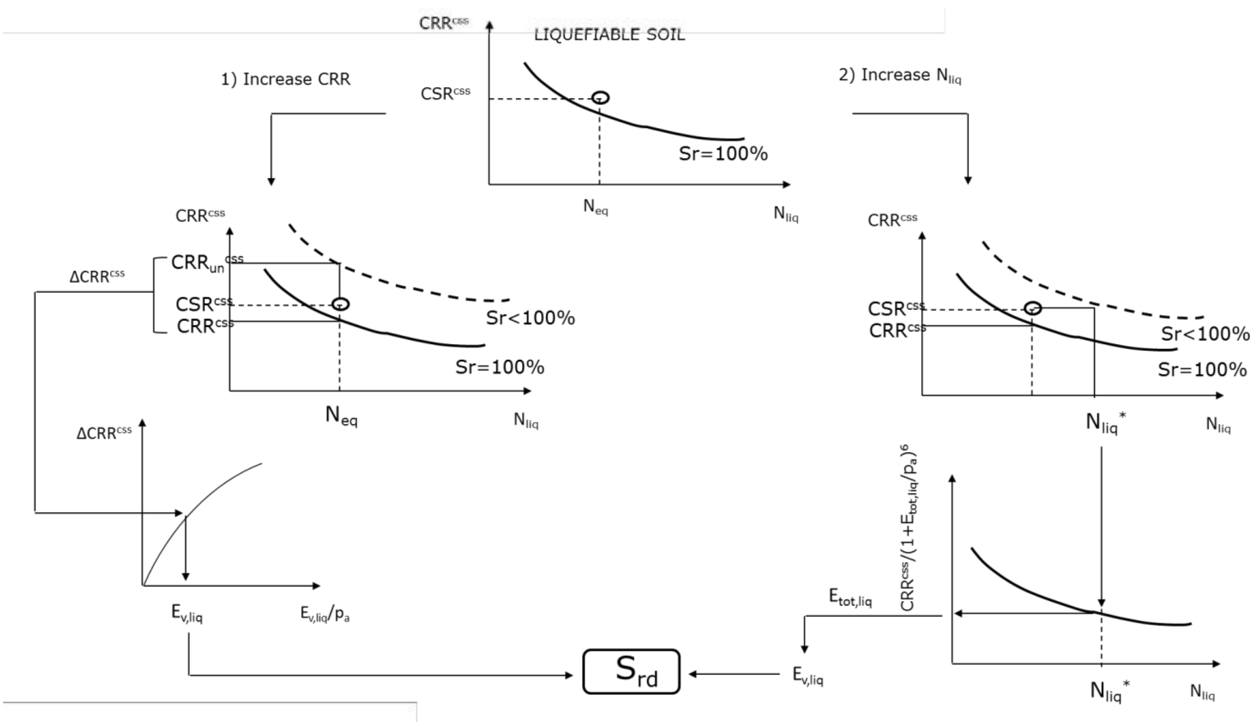

Figure 3-45: Possible procedures to calculate the degree of saturation needed to increment liquefaction resistance of sandy soils. The once on the left refers to Approach 1 (increase CRR); the one on the right to Approach 2 (increase N_{liq}).

From a technical point of view, the correlation between CRR and q_{c1Ncs} allows to evaluate the liquefaction susceptibility of a site. Based on the energetic model, unsaturated curves may be achieved as shown in Figure 3-46. The chart was obtained for several degrees of saturation translating the saturated one by using Equation 3-56, according to approach 1.

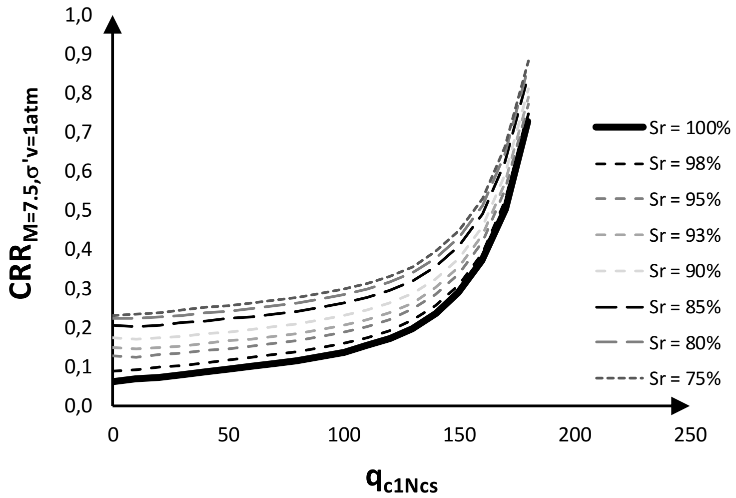

Figure 3-46: CRR vs q_{c1Ncs} for different S_r.

3.5.5 Ground reinforcement

The basic principle of ground reinforcement is to adsorb part of the stress generated in a liquefiable layer by the seismic action, reducing the strain and thus the solicitation to the liquefiable soil. A simplified mechanical analysis may be performed considering the Cyclic Stress Ratio at a generic depth in a liquefiable layer with and without reinforcement (Figure 3-47). While CSR for an unreinforced soil is equal to:

$$CSR\ (z) = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{vo}(z)}{\sigma'_{vo}(z)}\right) \cdot r_d(z)$$ **Equation 3-62**

For the reinforced soil, the shear stress reduction factor that accounts for the dynamic response of the soil profile has not to be taken into account due to the stiffening action of the reinforcement and thus the cyclic stress ratio for the reinforced soil CSR_{rs} is:

$$CSR_{rs}\ (z) = \frac{CSR(z)}{r_d(z)} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{vo}(z)}{\sigma'_{vo}(z)}\right)$$ **Equation 3-63**

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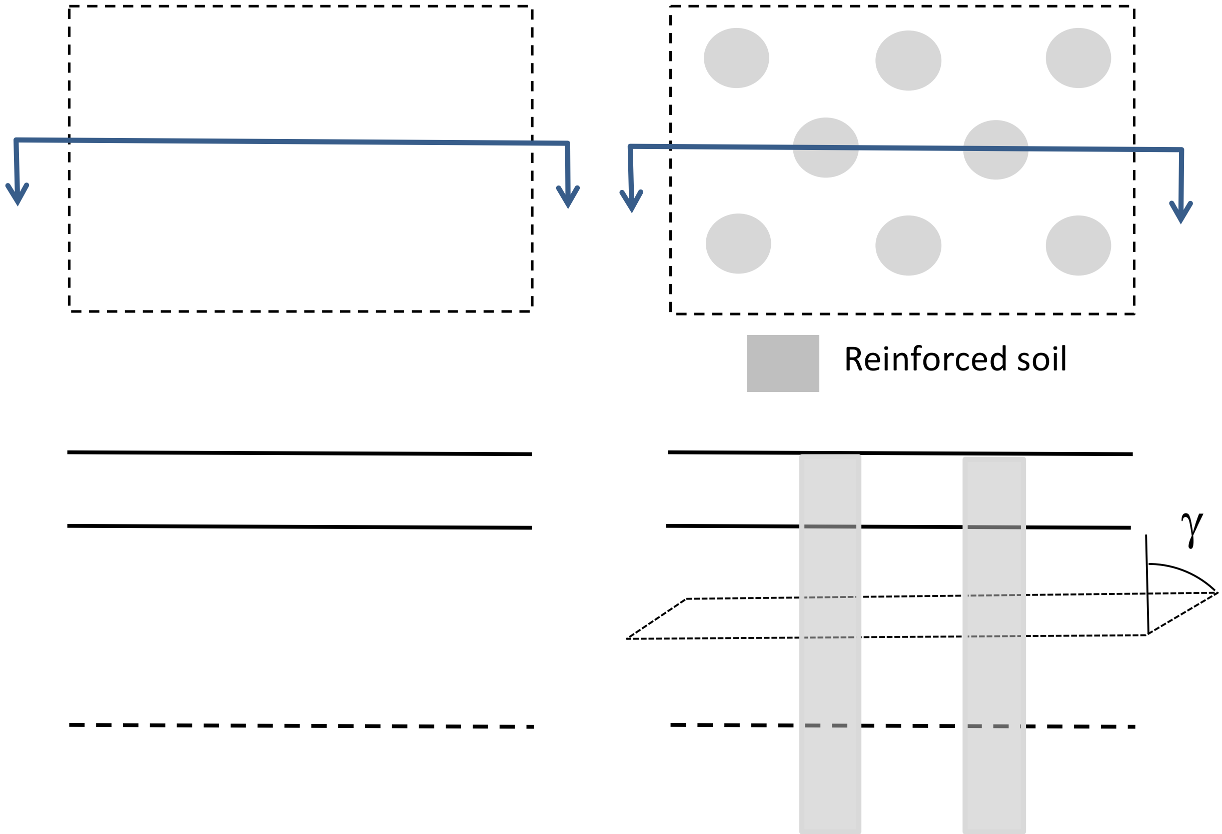

Figure 3-47: Schematic plan view and cross section of liquefiable soil with and without columnar reinforcement.

The angular distortion γ at any depth in the reinforced soil can be assumed equal for the original soil and for the reinforced portion, writing the following consistency equation:

$$\gamma = \gamma_r = \gamma_s \qquad \text{Equation 3-64}$$

Coupling the above condition with equilibrium and defining with a_r the replacement ratio, i.e. the ratio of between the area of columnar reinforcement (A_r) over the total area of soil (A), an equivalent shear stress τ_{eq} can be defined for the homogenised material:

$$\tau_{eq} = \tau_r \cdot \frac{A_r}{A} + \tau_s \cdot \frac{A_s}{A} \qquad \text{Equation 3-65}$$

Together with an equivalent shear stiffness G_{eq}:

$$G_{eq} = G_r \cdot a_r + G_s \cdot (1 - a_r) \qquad \text{Equation 3-66}$$

where G_s and G_r are the shear stiffness of respectively the original soil and reinforcement.

The shear stress in the untreated portion of the reinforced soil can thus be computed as:

$$\tau_s = G_s \cdot \gamma = \frac{G_s}{G_r \cdot a_r + G_s(1 - a_r)} \tau_{eq} \qquad \text{Equation 3-67}$$

Finally, the cyclic stress ratio for the untreated portion of the reinforced soil (CSR*) can be computed as:

$$CSR^* = CSR_{rf} \cdot \frac{\tau_s}{\tau_{eq}} = \frac{CSR_{rf}}{\frac{G_r}{G_s} \cdot a_r + (1 - a_r)} \qquad \text{Equation 3-68}$$

Figure 3-48 shows the reduction of CSR produced on a liquefiable soil by reinforcement as a function of the replacement ratio a_r and of the relative stiffness between reinforcement and original soil (G_r/G_s).

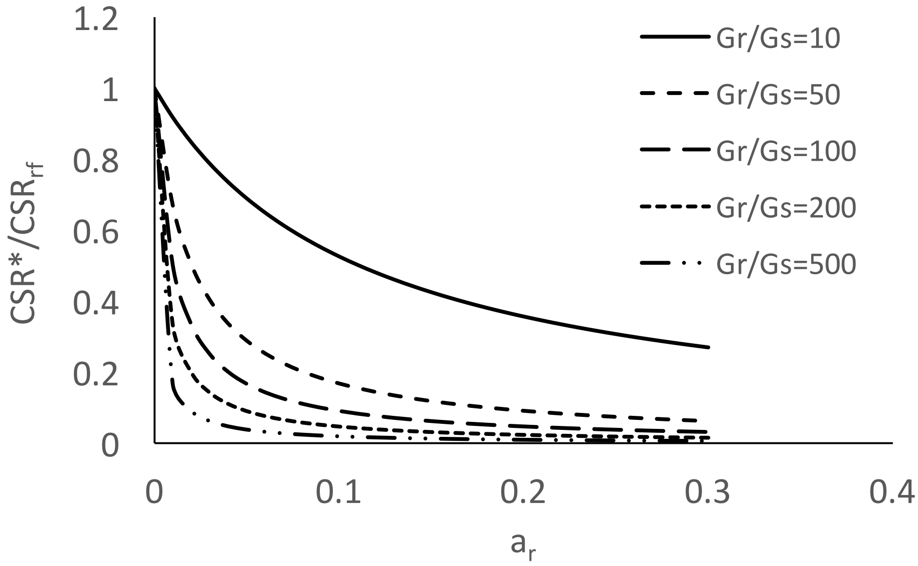

Figure 3-48: Schematic plan view and cross section of liquefiable soil with and without columnar reinforcement.

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4. QUALITY ASSURANCE / QUALITY CONTROL

4.1 Principle

The main objective of controls is to guarantee that products are able to meet the requirements for which they have been created. In geotechnical engineering, control may be particularly cumbersome because the products of the relevant industrial activities are normally buried underground. However, control is becoming an important issue also in geotechnical engineering, especially when ground improvement techniques are involved. Moreover, because there is still a relevant degree of uncertainty on the effects of ground improvement techniques, production and performance controls are often the best way to refine the decision-making process for subsequent projects.

The controls are generally codified by manuals prescribing a series of "pass or fail" tests to be carried on at fixed steps of the production process. The supervision of the construction process and workmanship is performed by means of in situ and laboratory tests and the monitoring of the performance of structures is carried out during and after their construction. Since not all ground improvement techniques have their own guidelines, where available they will be taken into consideration, if not available will be given suggestions on possible techniques that can be used for the control.

The set of controls to verify that the requirements of ground improvement are met can be summarised as follows:

- *Quality assurance (QA)*;
- *Quality control (QC)*;
- *Field trial*.

4.1.1 Quality assurance

Quality assurance focuses on the *process*. QA is aimed at checking the entire execution process, to ensure that all steps are accomplished correctly. QA procedures serve to confirm the quality of the basic components and the effectiveness of each working phase. QA ensures that treatment process, materials, equipment and treatment parameters are those defined in the project. The type of test to be performed and the parameters to be checked depend on the treatment that has been carried out.

4.1.2 Quality control

The quality control is aimed at evaluating the properties of the treated elements with reference to the specification given in the project. QC mainly focuses on the *product*, testing for quality and detecting defects. As for QA, the type of test to be performed and the parameters to be checked depend on the treatment that has to be carried out.

4.1.3 Field trial

The field trial consists of performing preliminary ground improvement treatments and appropriate tests for the verification of the results with respect to the design requirements. The field trial is therefore an essential phase for connecting the design and the execution of the treatments. Its most important aims are summarised as follows:

- to choose the procedure and select the most appropriate treatment parameters;
- to assess the treatment results with respect to the project requirements;
- to check the effects of treatments on the surrounding environment and structures;
- to refine the control procedure to be implemented during construction.

The significance of the field test results strongly depends on the similarity of the geotechnical conditions to the actual site. Therefore, field tests should be performed in the immediate vicinity of the work or at least in a similar geotechnical context.

It is suggested to plan the field trial at the design stage, stating the objectives, the extent and the method of the investigation to be performed. Following this strategy, the design quality would be certified, thus avoiding possible litigation that may subsequently arise between the client and the contractor.

The measurement of the possible undesired effects on the surrounding environment should be investigated by field trials. A monitoring system should be conceived at the design stage and then tested during the field trial to detect possible ground movements that may be harmful to neighbouring structures. Pore pressure and temperature recording may be also useful to foresee possible environmental modifications.

Moreover, the monitoring systems should be carefully tested and then used during construction to provide active QC by sounding alarm signals in case the prescribed threshold values of relevant variables are exceeded.

The monitoring of the surrounding environment is an important aspect both in field trials and during construction. Critical issues from an environmental point of view are the diffusion of contaminants, noise and vibrations. In particular:

- in the case of risk of contamination, the concentration of pollutants should be measured periodically;
- noise and vibration measurements should be performed, (for example by using a network of geophones strategically positioned in the surroundings);
- heave and/or settlements have to be measured.

Accurate real-time monitoring can be implemented, such as vertical movements by GPS aerials or by settlement cells; differential settlements by a sequence of electronic liquid-level gauges or sub-horizontal inclinometers placed in the ground; tilting of buildings by wall inclinometers; deformation by extensometers and cracks by displacement transducers.

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4.2 Deep dynamic compaction

Deep dynamic compaction densifies the soil by means of high energy tamping, using a weight repeatedly dropped on the ground surface. In Deep dynamic compaction, the controls are aimed at:

- ensuring that an adequate energy is transferred onto the soil;
- checking that the construction procedure is correctly carried out and that the equipment is working properly;
- verifying the effectiveness of ground improvement (densification of the treated soil);
- monitoring the effects on the surrounding environment and structures.

Monitoring on the surrounding environment should be carried out during and after treatment, and also in the field trial, with regard to the following aspects:

- vibrations and noises;
- induced settlement/ground heave;
- pore water pressure evolution over time.

4.2.1 Quality assurance

Quality assurance for deep dynamic compaction treatments is aimed at controlling the following aspects of the process:

- size and mass of the tamper;
- drop height;
- number of drops;
- position of drops;
- number of tamping passes;
- mean energy applied.

Size and mass of tamper are typically fixed with the equipment and represent the initial set-up. The other factors should be recorded during treatment and information stored on a specific report of daily activities. A particularly delicate issue is represented by the mean energy discharged onto the soil, which could be affected by a malfunctioning of the pulley, rope and brake system adopted for lifting the tamper. A monitoring of the falling speed should be periodically performed to avoid possible negative effects.

4.2.2 Quality control

Quality control for deep dynamic compaction are aimed at verifying the occurrence of densification and performance of the treated area. The effectiveness of improvement can be verified by comparing the results before and after treatment.

This goal can easily be accomplished by means of standard in-situ tests, that include:

- Cone penetration test (CPT), (ASTM D5778, 2012);
- Standard penetration test (SPT), (ASTM D1586 / D1586M, 2018);
- Shear wave velocity measurement (e.g. Cross-hole tests - ASTM D4428 / D4428M, 2014; Seismic tomography - ASTM D5777, 2018);
- Loading test (LT), (ASTM D1143 / D1143M - 07(2013)e1, 2013);
- Dynamic cone penetrometer test (DCP), (ASTM D6951 / D6951M, 2018);
- Dilatometer test (DMT), (ASTM D6635 , 2015);
- Pressumeter test (ASTM D4719 (2000) or Menard Pressumeter test, MPT);
- Direct density measurement (DDM), (ASTM D1556 / D1556M - 15e1, 2015), (ASTM D6938 - 17a, 2017), (ASTM D2937 - 17e2, 2017).

Since the results obtained after treatment may differ somehow with time, it is suggested to wait several days before performing quality control testing. A useful list of suitable testing methods to assess the compaction in sand, with advantages and disadvantages for each method, is provided by Kirsch & Kirsch (2010) and reported in Table 4-1.

Table 4-1: Suitable testing methods to measure compaction in sand (modified from Kirsch & Kirsch, 2010).

Test	Available data	Repeatability	Depth	Measures	Soil sample?	Detection of soil variability	Cost
SPT	Abundant	Poor to good	Deep	Index	Yes	Good	Low
CPT	Abundant	Very poor	Deep	Index	No	Very good	Low
MPT	Sparse	Poor	Medium	Property	No	Fair	Moderate
V_s	Limited	Good	Deep	Property	No	Fair	Low
LT	Limited	Very good	Low	Property	No	Poor	High
DDM	Limited	Poor	Low	Property	Yes	Very good	Very high

4.2.3 Field trial

Different combinations of parameters (namely grid spacing, drop height, number of drops and size of tamper) should be tested in order to find the one giving the required degree of improvement. After each experiment the increase of soil density should be measured in an intermediate position among the footprints of the tamper. The different outcomes can be compared on the basis of the results obtained from the in-situ tests (mentioned for quality control). Simultaneously instruments for measuring vibration at some distance from the field trial should be installed to assess the effects on the surrounding environment. At the end of the field trial, the whole procedure, including execution and control, should be implemented.

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4.3 Vibro compaction

Deep - vibro compaction (or vibro compaction) involves the use of a depth vibrator to densify granular soils. Once the vibrator penetrates at the required depth, accompanied by air and/or water jetting, the horizontal vibrations allow a denser configuration of the soil. The treatments are carried out at prescribed intervals retracting the vibrator to the top. During compaction, additional backfill is added from the top to fill the depression.

Shallow – vibro compaction /Replacement involves the removal of the native liquefiable soil and replacement with a non-liquefiable soil. The replacement material is compacted by means of vibratory rollers.

In vibro compaction (deep and shallow), the controls are aimed at:

- checking that the construction procedure is correctly carried out and that the equipment is working properly;
- verifying the ground improvement (densification of the treated soil and replacement in the case of *Shallow-vibro compaction /Replacement*);
- monitoring the effects on the surrounding environment and structures.

Monitoring in treated area must be carried out during and after treatment, as well as during field trials including the following aspects:

- vibrations and noises produced by the treatment;
- induced settlement/ground heave;
- pore water pressure and its evolution with time.

For *Shallow-vibro compaction/Replacement* the controls are simpler and mostly related to the quality of the replacement material (grain size), the compaction energy and procedure for QA and for QC the same tests used for *Deep-vibro compaction*, reported below (in particular SPT).

The following controls are related to *Deep-vibro compaction* (generally reported as vibro compaction). Useful information can be taken from Kirsch & Kirsch (2010).

4.3.1 Quality assurance

Quality assurance for vibro compaction is aimed at verifying the following aspects that characterize the treatment process:

- spacing and grid of the compaction points;
- probe penetration and withdrawal rates;
- power of vibrator;
- treatment depth;

- water management;
- quantity and quality of the added backfill.

Current vibro compaction apparatuses are provided with real time monitoring devices that record many parameters such as penetration depth, energy consumption, execution time and pressure and amount of water/air used during penetration. In particular, the vibrator amperage draw is a real-time improvement measure since when the densification occurs it causes a reduction in the horizontal movement of the vibrator and therefore an increased draw to maintain the frequency constant.

4.3.2 Quality control

Quality control for vibro compaction, as for the other techniques that have the main purpose of densifying the soil, are aimed at verifying the occurrence of densification and performance of the treated area. The effectiveness of improvement can be verified by comparing the pre-treatment and post-treatment results, by means of standard in-situ tests with the same standards reported in §4.2.2:

- Cone penetration test (CPT);
- Standard penetration test (SPT);
- Shear wave velocity measurement (e.g. Cross-hole tests; Seismic tomography);
- Loading test (LT);
- Dynamic cone penetrometer test (DCP);
- Dilatometer test (DMT);
- Pressumeter test (e.g. Menard Pressumeter test, MPT);
- Direct density measurement (DDM).

As previously reported for deep dynamic compaction (§4.2.2), a useful list of suitable testing methods to assess the compaction in sand are reported in Table 4-1.

During treatment, the vibration produces an increase in pore water pressure, so it is recommended to wait for their dissipation before running control tests (usually 1 week). To this aim, it may be convenient to monitor the trend of the pore water pressure over time.

4.3.3 Field trial

In field trials, different combinations of treatment parameters (such as penetration depth of the probe, mean extraction intervals, vibration frequency, duration of compaction, pressure of the water/air jets, grid of treatment) can be tested to obtain the desired degree of improvement. The different combinations can be compared on the basis of the results obtained from the in-situ tests (mentioned for quality control).

An example of in situ test for the evaluation of vibro compaction effectiveness is reported in Figure 4-1. In this case SPT test have been run before and after compaction (Figure 4-1.a), showing the increase of blow counts obtained with compaction. The test can be arranged with different grid spacing of the compaction

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holes (Figure 4-1.b), measuring the variation of density as a function of the distance (Figure 4-1.c) and finding the one giving the desired result.

Simultaneously instruments for measuring vibration at some distance from the field trial should be installed to assess the effects on the surrounding environment. At the end of the field trial, the whole procedure, including execution and control, should be implemented.

(a) (b) (c)

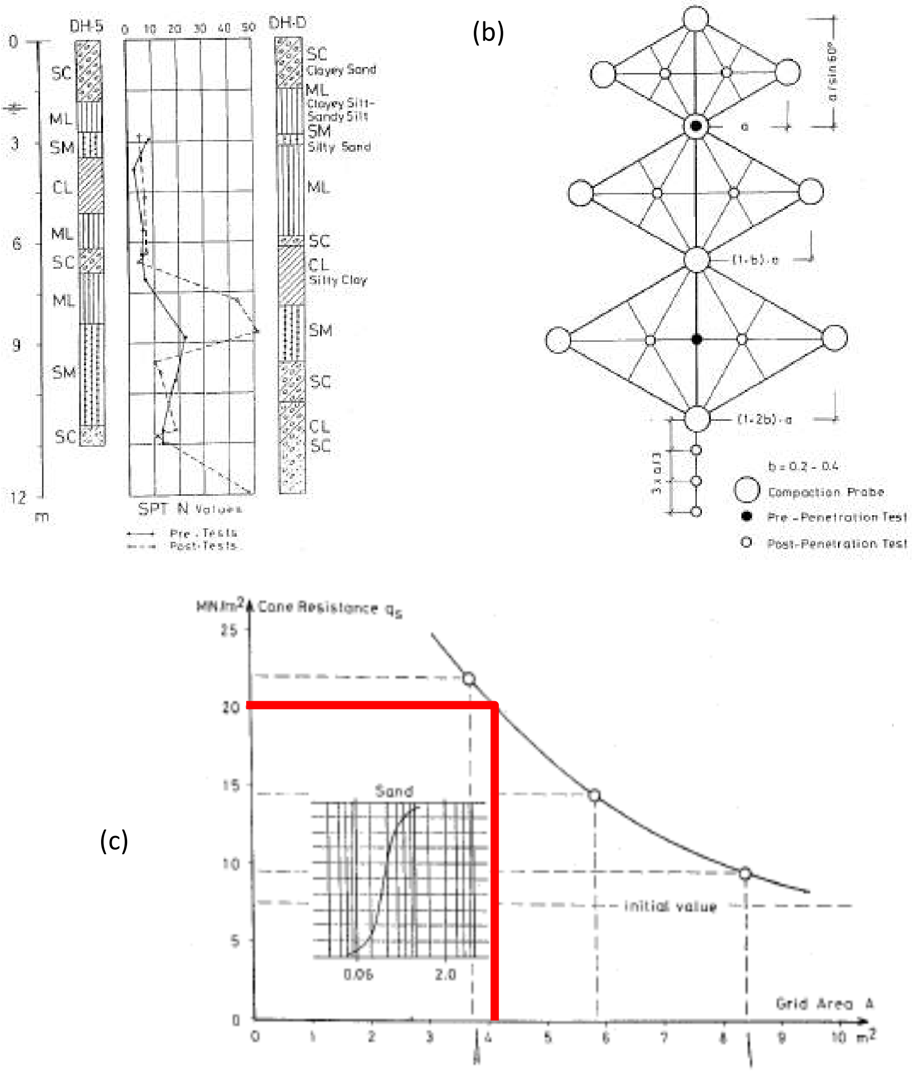

Figure 4-1: Set up of field trial for vibro compaction: (a) example of SPT run before and after treatment; (b) possible layout of tests to evaluate the influence of grid spacing; (c) example of results.

4.4 Blasting compaction

Blasting compaction uses the detonation of explosive charges to densify the surrounding soil. In this technique, the main purposes of controls are:

- to ensure adequate charge and type of explosive;
- to check that the construction procedure is correctly carried out and that the equipment is working properly;
- to verify the ground improvement (densification of the treated soil);
- to monitor the effects on the surrounding environment and structures.

Monitoring must be carried out during and after treatment, as well as during field trials. Particular attention is devoted to the control of vibration and noise caused by the treatment, especially in sensitive areas where buildings are located, to the ground settlement or heave and to the pore water pressure evolution over time.

4.4.1 Quality assurance

The main quality assurance controls for blasting compaction concern the following items, that must comply with the project requirements:

- type of explosive;
- blasting procedure and equipment;
- depth of charge;
- vertical distance between charges;
- distance between the holes;
- phasing and number of blast stages;
- sequence of explosions.

4.4.2 Quality control

The quality control can be carried out in a similar way to that reported for deep dynamic compaction (§4.2.2) and vibro compaction (§4.3.2).

4.4.3 Field trial

In field trials for basting compaction, different configuration of the charges, sequence of explosions, phasing and number of the stages can be tested and verified by means of the results obtained from the in-situ tests (mentioned for quality control). A set-up similar to that carried out in Figure 4-1 may be implemented, considering different spacing between charges, and maybe different amount of explosive, and verifying results with in situ tests.

4.5 Compaction grouting

Compaction grouting consists of injecting a stiff grout into the ground causing a growth of the bulb of the injected material that displaces and compacts the surrounding soil. In compaction grouting, the aims of controls are:

- to ensure adequate quality and properties of the grout;
- to check that the construction procedure is correctly carried out and that the equipment is working properly;
- to verify the ground improvement (mostly densification of the treated soil);
- to monitor the effects on the surrounding environment and structures.

Monitoring must be carried out during and after treatment, as well as during field trials. Particular attention is devoted to the control of ground heave that can be caused by the treatment.

Guidelines and specifications for compaction grouting are provided by ASCE/G-I 53-10 (2010) and European Standard (EN 12715, 2000). Useful indications on QA/QC are also reported by Hussin (2013).

4.5.1 Quality assurance

Quality assurance for compaction grouting is aimed at controlling the following aspects of the process:

- quality of the grout, quality of the base materials, proportions of the components;
- drilling of boreholes;
- injection of the grout.

In particular, automatic record systems should be used to monitor the drilling of boreholes and to control the drilling and grouting parameters.

With regard to the grout, the design of the mix is carried out in the laboratory before the treatment, to achieve the required properties of the grout. Then, the mortar to be injected is subject to specific checks; on site the most used tests are the density and workability tests. Furthermore, during treatment, the quality and the proportions of components have to be monitored. If the treatment requires a minimum final strength, several specimens should be prepared at regular intervals (based on time, e.g. twice a day, or on grout volume, e.g. every 76 m^3) and tested in laboratory for mechanical characterisation. Indications for common measurements on mortar are provided by EN 12715 (2000) and reported in the Table 4-2.

Table 4-2: Measurement of mortar parameters.

Parameter	Method	Application
Outflow time (Cone viscosity)	Marsh cone or other flow cones	Laboratory and site
Density	Pycnometer, Beaker, Baroid mud balance	Laboratory and site
Water retention capacity	Baroid filter press (low pressure)	Laboratory and site
Bleeding rate, sedimentation	Measuring cylinder	Laboratory and site
Workability	Abrams cone	Laboratory and site
Setting time	Overturned glass beaker, Vicat needle	Laboratory and site
Hardening time	Vane test, shear box, Unconfined compression test	Laboratory and site
Hardening, deformation, final strength	Unconfined compression test with stress-strain record, Triaxial test, Point load tests	Laboratory
Durability	Mechanical: flow test Chemical	Laboratory
Shrinkage/Expansion	Shrinkage limit determination	Laboratory and site
Granulometry	Particle size measurement	Laboratory and site

According to EN 12715 (2000), during boreholes drilling, the following parameters are usually automatically recorded and controlled, providing geological and geotechnical information at each treatment point:

- rate of penetration;
- fluid pressure;
- flow rate;
- reflected energy;
- rotational speed;
- torque;
- pull down force;
- borehole length.

During injection process, several parameters should be monitored and controlled, as reported below:

- grout injection pressure (near the pump and near the top of the injection pipe);
- grout injection rate (the control is often performed by calibrating the pump piston and counting the number of piston strokes during injection (Hussin, 2013)).
  A progressive increase in injection pressure confirms that a densification process is occurring. A drop in pressure may indicate hydraulic fracturing. If the injection rate is too high the pore water pressure has no time to drain resulting in an increase of pumping pressure, giving a false indicator of improvement.
- volume of grout injected (usually 5÷15% of the volume of the soil being treated);
- ground heave and surface movements (crack monitor, tiltmeters measurements, string lines, plumb bobs, spirit levels, fluid levels, rotating laser levels).

4.5.2 Quality control

Verification of densification is generally carried out using indirect methods that are correlated to density, the most common tests are based on the penetration resistance measurement (SPT, CPT), performed before and after treatment. If not significant improvement in penetration resistance is attained, load test can be performed on individual column or on treated area.

Quality control may include in-situ or laboratory tests, aimed at analysing the mechanical, physical and geophysical properties. Common verifications include the following tests, as for the other densification techniques:

- Cone penetration test (CPT);
- Standard penetration test (SPT);
- Shear wave velocity measurement (e.g. Cross-hole tests; Seismic tomography);
- Loading test (LT);
- Dynamic cone penetrometer test (DCP);
- Dilatometer test (DMT);
- Pressumeter test (e.g. Menard Pressumeter test, MPT);
- Direct density measurement (DDM).

Moreover, one of the most useful tools for assessing the effectiveness of the treatment performed, supported with quality control tests, is to record, monitor and assess the treatment parameters during time.

The direct control of the treatment consisting in excavation with visual observation can be conducted in particular conditions and for shallow treatments.

Sometimes, the control may include permeability tests.

4.5.3 Field trial

On large projects and on sensitive areas, field trial is an efficient way of optimizing the treatment procedure. Different setup, mix design of the grout, injection intervals and treatment parameters can be tested using the same verification methods as for QA and QC. In this context, excavations with visual observation of the treatment can be useful to calibrate the indirect verification methods.

4.6 Low pressure grouting

Low pressure grouting involves injecting low pressure grouts into the soil, filling the pores without altering the original structure. The injection pressure is kept below the value that causes the fracture of the soil.

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Controls in low pressure grouting are aimed at:

- ensuring the quality and properties of grout;
- checking that the construction procedure is correctly carried out and that the equipment is working properly;
- verifying the ground improvement (stabilisation of the treated soil);
- monitoring the effects on the surrounding environment and structures (during and after treatment, as well as during field trials).

Guidelines and specifications for low pressure grouting are provided by the European Standard on grouting (EN 12715, 2000). Useful indications on QA/QC are also reported by Stadler & Krenn (2013).

4.6.1 Quality assurance

Quality assurance for low pressure grouting is directed at controlling the following aspects of the process:

- quality of the grout, quality of the base materials, proportions of the components;
- drilling of boreholes;
- injection of the grout.

Automatic record systems should be used to monitor the drilling of boreholes and to control the drilling and grouting parameters.

With regard to the grout, the design of the mixture (solution, suspension or mortar) is carried out in the laboratory before the treatment, to achieve the required properties.

The quality and consistency of grout should be controlled by performing control tests. According to the European Standard, the routine tests that are commonly carried out on site, on the material to be injected, with regard to the type of grout (suspensions, solutions and mortars), are reported in Table 4-3.

Table 4-3: Grout control tests.

<table>
<tr><th>Suspensions</th><th>Microfine suspensions</th><th>Solutions (chemical grout)</th><th>Mortars</th></tr>
<tr><td>Density</td><td>Density</td><td rowspan="2">Density</td><td rowspan="2">Density</td></tr>
<tr><td>Marsh viscosity</td><td>Grainsize / Sand column tests</td></tr>
<tr><td>Setting time</td><td>Viscosity</td><td rowspan="2">Setting time</td><td rowspan="2">Workability</td></tr>
<tr><td>Bleeding</td><td>Bleeding</td></tr>
</table>

Grouting components and quantity should be controlled during the treatment. The grout to be injected is then subject to specific checks performed on site and in laboratory. Common tests are provided by EN 12715 (2000) and shown in the Table 4-4 for the different types of substance injected.

Table 4-4: Measurement of grout parameters.

Parameter	Method	Application	Solution	Suspension	Mortar
Outflow time (Cone viscosity)	Marsh cone or other flow cones	Laboratory and site	X	✓	✓
Viscosity (dynamic or apparent)	Coaxial viscometer, Rheometers	Laboratory Laboratory and site	✓	✓	X
Density	Pycnometer, Beaker, Baroid mud balance	Laboratory and site	✓	✓	✓
Cohesion, yield, shear strength	Coaxial viscometer, Rheometer, Plate Cohesion meter, Kasumeter, Shearometer	Laboratory Laboratory and site	X	✓	X
Water retention capacity	Baroid filter press (low pressure)	Laboratory and site	X	✓	✓
Bleeding rate, sedimentation	Measuring cylinder	Laboratory and site	X	✓	✓
Workability	Abrams cone	Laboratory and site	X	X	✓
Setting time	Overturned glass beaker, Vicat needle	Laboratory and site	✓	✓	✓
Hardening time	Vane test, Shear box, Unconfined compression test	Laboratory and site	✓	✓	✓
Hardening, deformation, final strength	Unconfined compression test with stress-strain record, Triaxial test, Point load tests	Laboratory	✓	✓	✓
Durability	Mechanical: flow test Chemical	Laboratory	✓	✓	✓
Thixotropy	Rheometer, Viscometer, Hydrometer	Laboratory	X	✓	X
Syneresis	Volume of water expelled from sample with time	Laboratory	✓	X	X
Shrinkage/Expansion	Shrinkage limit determination	Laboratory and site	✓	✓	✓
Granulometry	Particle size measurement	Laboratory and site	X	✓	✓
Penetrability	Grouting test, Sand column test	Site Laboratory	✓	✓	X

According to EN 12715 (2000), during boreholes drilling, the following grouting parameters are usually automatically recorded and controlled, providing geological and geotechnical information, at each treatment point:

- rate of penetration;
- fluid pressure;
- flow rate;
- reflected energy;
- rotational speed;
- torque;
- pull down force;
- borehole length.

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During injection process, several parameters should be monitored and controlled, as reported below:

- grout injection pressure;
- grout injection rate;
- volume of grout injected;
- ground heave and surface movements.

4.6.2 Quality control

Quality control may include in-situ or laboratory tests, aimed at analysing the mechanical, physical and geophysical properties. Verification of the effectiveness of stabilisation treatment is generally carried out using indirect methods. Most common verification tests include:

- Shear wave velocity measurement (e.g. Cross-hole tests, Seismic tomography);
- Cone penetration test (CPT);
- Standard penetration test (SPT);

and, other possible tests include:

- Pressumeter test (e.g. Menard Pressumeter test, MPT);
- Dynamic cone penetrometer test (DCP);
- Dilatometer test (DMT);
- Loading test (LT).

The direct control of the treatment consisting in excavation with visual observation can be conducted in particular conditions and for shallow treatments. Mechanical tests can be performed in samples cored from the treated soil.

The control can include permeability tests if the treatment is performed to obtain a reduction in soil permeability or an impermeable barrier.

4.6.3 Field trial

Field trial is an efficient way of optimizing the treatment procedure and to assess the effectiveness of the treatment. Different setup, mix design of the grout, type of injection and treatment parameters can be tested using the methods for QA and QC. Direct observation of the treated soil, with excavation of soil portions, can be useful to assess the effectiveness of the treatment and to calibrate the indirect verification methods.

4.7 Earthquake drains

The Earthquake drains are prefabricated vertical drains with high flow capacity, consisting of perforated corrugate plastic pipes sheathed in a geosynthetic filter to prevent the particles flow into the drain. The EQ drains provide a dissipation of pore water pressure excess generated into saturated cohesionless soils during the earthquake before liquefaction occurs. For the earthquake drains, the most important aspects of the controls are aimed at:

- ensuring the quality of the base materials of the drain;
- checking that the installation procedure is correctly carried out;
- verifying the ground improvement (drainage);
- monitoring the effects on the surrounding environment and structures (during and after treatment, as well as during field trials).

Specifications for vertical drainage are provided by the European Standard (EN 15237, 2007) moreover, helpful indications on QA/QC are also reported by Chu & Raju (2013) on the basis of ASTM and other specifications. The following indications are based on the European Standard.

4.7.1 Quality assurance

According to the European Standard, the most important aspects of the quality assurance checks concern the following items:

- *Dimensions of the core*: diameter and thickness.
- *Durability*: the drains must be protected to weathering during storage on site, to avoid damage and deterioration (EN 13252, 2000)
- *Tensile strength and elongation of the drain*: high enough to prevent breakage during and after installation. As suggested by European Standard, testing should be carried out in accordance with the standard tensile test with modified clamps.
- *Strength of the seam*: measured according to EN ISO 10321(1992) shall be at least 1 kN/m.
- *Discharge capacity and filtration characteristics*: the discharge of ED is higher than is required for soil consolidation. For the mitigation of liquefaction, it is important that the drain pipe and filter resist the effect of aging during the design lifetime of the structure.
- *Visual inspections for damage*: should be carried out regularly to identify any damaged parts of the drains, in particular of the filter.
- *Tensile strength per unit width of filter*: high enough to prevent breakage during and after installation. Testing should be performed in accordance with EN ISO 10319 (1993). The European Standard (EN 15237, 2007) suggests to ensure an average individually value of tensile strength higher than 3 kN/m and for deep installations (>25 m) higher than 6 kN/m.
- *Velocity index of filter*: testing should be performed in accordance with EN ISO 11058 (1999). The European Standard (EN 15237, 2007) suggests to ensure an average individually measured value of the velocity index (v_{h50}) higher than 1 mm/s.

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- *Pore size of filter*: the characteristic opening size O_{90}, measured in accordance with EN ISO 12956 (1999) should be lower than 80 µm.

During installation, the following parameters are usually recorded:

- drain identification number; σ✓
- date and time; σ✓
- depth of installation; σ✓
- drain length; σ
- verticality and location. σ

The construction process must be monitored, in particular with regard to the ground conditions and the construction tolerances. Moreover, if contaminated water is squeezed out of the soil through the drains (during consolidation), it must be treated.

4.7.2 Quality control

According to the European Standard, the identification of prefabricated drains on site shall be carried out according to EN ISO 10320 (1999), or similar procedures for specific characteristics.

The process of consolidation can be monitored and verified by measurement of settlements (settlement gauges) and pore water pressure (piezometers). In Figure 4-2 and Figure 4-3 are reported typical instrumentation for monitoring the efficiency of vertical drainage. When relevant, laboratory and in-situ tests can be performed to assess the increase in resistance.

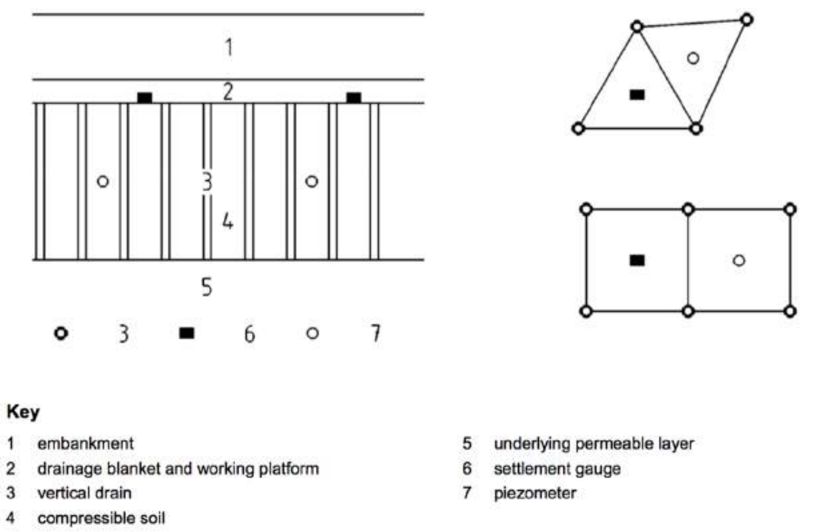
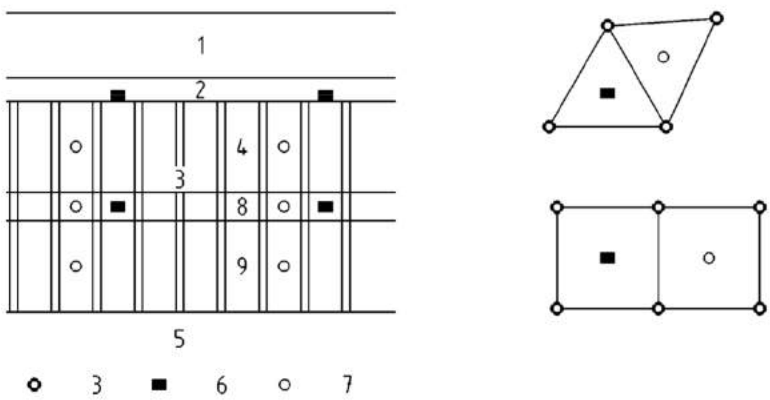


Figure 4-2: Typical instrumentation for monitoring the efficiency of vertical drainage – Homogeneous stratification (from EN 15237, 2007).



Key

1 embankment
2 drainage blanket and working platform
3 vertical drain
4 compressible soil
5 underlying permeable layer
6 settlement gauge
7 piezometer
8 permeable sand layer
9 compressible soil

Figure 4-3: Typical instrumentation for monitoring the efficiency of vertical drainage – Site with different layers (from EN 15237, 2007).

4.7.3 Field trial

The field trials can be created by setting different spacing between the drains (and different types of drains) to evaluate the best possible solution. The process of consolidation can be monitored by measurement of settlements (settlement gauges) and pore water pressure (piezometers). Laboratory and in-situ tests can be performed to assess the increase in resistance.

For liquefaction mitigation trial blasting tests can be performed in order to evaluate the dissipation of the excess of pore water pressure over time generated by explosions (Rollins et al., 2004).

4.8 Induced partial saturation

Induced partial saturation consists in introducing gas bubbles into the soil reducing the degree of saturation and increasing the liquefaction resistance. The most important aspects of the controls are:

- to check that the procedure is correctly carried out;
- to verify the ground improvement (desaturation);
- to monitor the effects on the surrounding environment and structures (during and after treatment, as well as during field trials).

Since induced partial saturation it is a relatively new technology, to date no guidelines or specifications are available.

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In this type of treatment, it is essential to monitor the permanence of air bubbles over time to ensure the improvement over time.

4.8.1 Quality assurance

Quality assurance for induced partial saturation is aimed at controlling the following aspects of the process:

- installation of the injection pipes (horizontal or vertical);
- depth/length of the pipes;
- correct location of the pipes;
- injection pressure and rate of the air/gas;
- volume of air/gas injected.

4.8.2 Quality control

Quality control may include in-situ and laboratory tests, aimed at analysing the reduction in the degree of saturation. In particular, several samples can be taken from the treated area and analysed in the laboratory to assess the degree of saturation before and after treatment (and also the change in mechanical properties).

Since the electric resistivity of a soil depends of a several parameters including water content, the degree of saturation can be evaluated by measuring electric resistivity (Archie's law). Measurements of 3D electrical resistivity tomography can be used in-situ to check that the desaturation has taken place and monitor the degree of saturation over time.

Moreover, the bulk wave velocity V_p is affected by the degree of saturation, thus the saturation degree can be estimated by means of V_p measurements.

4.8.3 Field trial

Field trial can be used to evaluate different treatment parameters and configurations, as well as to verify their effectiveness over time.

4.9 Vibro replacement

Vibro replacement is a deep compaction technique for cohesive soils and granular soil with high fine content that involves the use of depth vibrator. The vibrator penetrates at the required depth accompanied by a water/air jetting and the granular backfill is added from the top or the bottom. The horizontal vibrations allow a densification of the backfill forming granular columns (REPLACEMENT and/or DISPLACEMENT). The treatments are carried out at prescribed intervals retracting the vibrator to the top.

In vibro replacement, the controls are aimed at:

- ensuring proper type and properties of the backfill;
- checking that the construction procedure is correctly carried out and that the equipment is working properly;
- verifying the ground improvement (mainly densification and reinforcement);
- monitoring the effects on the surrounding environment and structures.

Monitoring in treated area must be carried out during and after treatment, as well as during field trials including the following aspects:

- vibrations and noises produced by the treatment;
- induced settlement/ground heave;
- pore water pressure evolution over time.

For the controls, useful information can be taken from Kirsch & Kirsch (2010).

4.9.1 Quality assurance

Quality assurance for vibro replacement is intended to verify the following aspects of the process:

- quantity and quality of backfill added;
- location of the compaction points;
- probe penetration rate and probe withdrawal rate;
- vibrator amperage draw;
- treatment depth;
- water management.

Suitability of backfill in terms of hardness and abrasion resistance may be verified by related testing methods such as the Los Angeles Test, following ASTM C131 (2001) or similar regulations such as the European Standards (EN 1097-2, 2010; and EN 13450, 2013).

Real time acquisition systems can record the most important treatment parameters such as identification number, penetration depth, energy consumption, execution time, pressure and amount of water/air used during penetration and stone volume (measured directly or deduced from measured stone weight). As for vibro compaction, the vibrator amperage draw is a real-time improvement measure since when the densification occurs it causes a reduction in the horizontal movement of the vibrator and therefore an increased draw to maintain the constant frequency. Where the amount of backfill cannot be measured directly versus depth, a realistic estimate of the total volume of stone needs to be carried out.

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4.9.2 Quality control

Quality control for vibro replacement are aimed at verifying the occurrence of the improvement and performance of the treated area. Quality control should include:

- verification of the diameter of the column (verification of the displacement-replacement process);
- records of the installation depth;
- volume of backfill installed.

The effectiveness of improvement (mainly densification) can be verified by comparing the pre-treatment and post-treatment results, by means of standard in situ tests, as mentioned for vibro compaction technique:

- Cone penetration test (CPT);
- Standard penetration test (SPT);
- Shear wave velocity measurement (e.g. Cross-hole tests, Seismic tomography);
- Loading test (LT);
- Pressumeter test (e.g. Menard Pressumeter test, MPT);
- Dynamic cone penetrometer test (DCP);
- Dilatometer test (DMT);
- Direct density measurement (DDM).

During treatment, the vibration produces an increase in pore water pressure, so it is recommended to wait for the total dissipation before post-compaction testing. For this reason, it may be convenient to monitor the trend of the pore water pressure over time.

The performance verification is usually performed on a group of columns or on the total area of improvement, depending on the purpose of the treatment.

If the treatment is performed on cohesive soils, the improvement is mainly based on the reinforcing effect, verified by ensuring a high replacement value (i.e. column diameter and spacing) and the friction angle of the backfill. It is recommended to verify the friction angle of the materials with preliminary laboratory tests.

4.9.3 Field trial

Different combinations of treatment parameters (such as penetration depth of the probe, mean extraction intervals, vibration frequency, duration of compaction, pressure of the water/air jets, grid of treatment) can be tested in the field trial in order to obtain the desired degree of improvement. The different combinations can be compared on the basis of the results obtained from the in-situ tests (mentioned for quality control).

4.10 Deep mixing method

Deep mixing techniques involve the mixing of deep in-situ soil with binder materials, like cement, lime, fly ash, slag or other types of binder, using wet or dry method. The improvement is based on the chemical interactions of the clayey soils with the binder, the bond between the particles and the filling of the voids with the products of the reactions (stabilisation+reinforment effects).

In deep mixing method, the most important features at the base of the controls are related to:

- ensuring the quality of the binder;
- checking that the construction procedure is correctly carried out and that the equipment is working properly;
- verifying the ground improvement (stabilisation and reinforcement of the treated soil);
- monitoring the effects on the surrounding environment and structures (during and after treatment, as well as during field trials).

Specifications for deep mixing are provided by the European Standard (EN 14679, 2005).

4.10.1 Quality assurance

Quality assurance for deep mixing treatments is aimed at analysing the following aspects:

- quality of the binder and water/binder proportion;
- delivery of the binder;
- penetration/withdrawal and mixing operations.

Real time monitoring systems should be used to monitor and to control the process of treatment. The most important aspects to check are (for each column):

- length of the treatment;
- penetration and withdrawal rates of the mixing tool;
- rotational speed and torque of the mixing tool;
- overlapping width and rate and pressure of delivery of binder/slurry;
- quantity of binder/slurry along the column and total consumption.

In particular, according to the European Standard the following parameters, reported in Table 4-5, shall be continuously monitored during execution, or at least at depth interval of 0.5 m

By monitoring the operation parameters from real time acquisition, some geotechnical information can be obtained.

The density of the slurry has to be tested least twice per working shift at each batching/mixing plant (higher frequency for manual batching).

Moreover, when the continuity of the treated soil is an important aspect of the design, it is required to monitor the position and the vertically of the mixing tool.

Vertical and lateral movements of the ground, and if useful pore water pressure, have to be monitored.

Table 4-5: Construction parameters (modified from EN 14679, 2005)

Dry method	Wet method
Air tank pressure	Slurry pressure; air pressure (if any)
Penetration and retrieval rate	Penetration and retrieval rate
Rotation speed (during penetration and retrieval)	Rotation speed (during penetration and retrieval)
Quantity of binder per meter of depth during penetration and retrieval	Quantity of slurry per meter of depth during penetration and retrieval

4.10.2 Quality control

According to EN 14679 (2005), quality control tests should be uniformly distributed in time and between the mixing tools, and should cover a number of columns suitable to evaluate the distribution and the average value of the analysed property. The extent of the tests and the methods used depend on the specific application of the treatment as reported in Table 4-6.

Table 4-6: Parameters of interest.

Objective	Main interest
settlement reduction	elastic modulus
improvement of stability	strength of the columns
immobilisation/confinement of waste/containment	overlapping and low permeability of the columns

Verification tests can be performed in laboratory and on site:

- In the laboratory, in addition to preliminary tests performed on laboratory mixed samples, core samples can be tested in order to study deformation characteristics, strength and uniformity of treated soil. The most common tests performed are mechanical tests (including unconfined compression tests, triaxial tests, oedometer tests) and hydraulic tests.
  Moreover, wet grabs samples can be taken from fresh stabilised columns prior to initial set of the treated soil.

- On site, mechanical properties can be determined using direct and indirect methods.
  Direct methods include:
  - Pressumeter test (mechanical properties of the columns: shear strength, compressibility);
  - Pressumeter-permeameter test (permeability of the column in radial direction).

Indirect methods include:
- CPT;
- Static/dynamic penetration tests;
- Column penetration test.

When the treatment is performed to immobilise contaminants or to stabilise waste deposits, since the binder could interact with in-situ material, specific tests should be performed (e.g. chemical tests like determination of content of chemically active substances, pH value, carbonate content, chloride content, sulphate and sulphide content).

If overlap is an important aspect of the design, the width of the overlapping between adjacent columns shall be checked (by using inclinometers during penetration and retrieval, by drilling across the columns or direct observation).

4.10.3 Field trial

When similar previous experience is not available, field trials shall be performed to confirm that the design requirements can be obtained and to establish the critical control values. Mechanical and hydraulic properties can be investigated realising field trial and performing several installations (usually two-three) with varied binder content and varying the treatment parameters.

4.11 Jet grouting

Jet grouting treatment consists of high-pressure jets (water-cement grout/water/air) to break the soil structure and mix the native soil with the grout in order to form a material with enhanced mechanical properties (soilcrete). Jet grouting treatments are mainly affected by three sources of uncertainty:

- *effectiveness of treatments*: dimensions and properties of the jet-grouted elements;
- *mechanical interaction between jet-grouted elements and soils*: overall behaviour of the jet-grouted structure;
- *possible undesired effects on the surrounding environment* (with particular regard to the neighbouring structures).

In jet-grouting treatments the main purposes of controls are:

- to ensure adequate characteristics for the basic materials adopted;
- to check that the construction procedure is correctly carried out and that the equipment is working properly;
- to quantify the dimensions and properties of the jet-grouted elements;
- to verify the performance of jet-grouted structures;

- to monitor the jet grouting effects on the surrounding environment and structures.

Although approached from different perspectives, quality control is widely discussed in the existing guidelines on jet grouting, including:

- Guidelines of the Japanese Jet Grouting Association, JJGA (2005);
- Guidelines of the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers (GI-ASCE 2009);
- European Standard for the Execution of special geotechnical works: Jet grouting, EN 12716 (2001).

According to the European rule, if jet grouting is performed in subsoil conditions similar to previous experiences (for which a detailed documentation is available), and if the static role of the jet-grouted elements is not critical for safety, the field trial can be omitted unless specifically requested in the project. In this case, however, the properties of the jet-grouted material should be measured on a relevant number of columns produced at the beginning of the work to check that the results meet the design requirements. In any case, continuous monitoring of the treatment parameters with periodically calibrated instruments and a record of the properties of returned spoil should be carried out all over the production process. Mechanical tests (unconfined compression, extension and shearing) on samples cored from the columns are finally prescribed with a specific frequency (e.g., four samples, each with 1000 m^3 of treated material). Checking the permeability is also required both for single columns, by performing pumping tests in holes previously drilled into the column, and for the whole jet-grouted elements made of several overlapped columns.

4.11.1 Quality assurance

Quality assurance for jet grouting treatments is aimed at analysing the following aspects:

- grout preparations;
- drilling and grouting;
- spoil return.

A preliminary aspect for jet grouting grout preparation concerns the material qualification as the materials used in the treatment affect the effectiveness of the injection and the properties of the jet-grouted elements and the treated area. The most relevant aspects of the material qualification are reported in Table 4-7.

The preparation of the grout requires periodic calibration of the dosing equipment to ensure that the designed composition of the mix is respected. A series of tests should be periodically performed on the grout to guarantee the design requirements (injection, erosion, mixing, develop the final designed properties) and to confirm, indirectly, a proper functioning of the mixing plant and the quality of the basic materials, as reported in Table 4-8.

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Table 4-7: Material qualification for jet-grouting.

Material	Quality	Standard	Typical type	Typical controls
CEMENT	Certified (supplier)	ASTM C150 (2004) BS 8500-1 (2015)	Ordinary Portland cement (Type I) Best type: depending on the sulphate concentration (of soil and water)	Compatibility of cement with the chemical characteristics of water and additives used to prepare the grout, groundwater, soil, etc.
WATER	Controlled on site	ASTM C1602 / C1602M (2018) Standard for concrete	Drinkable water, water coming from public nets, clear and non-corrosive groundwater or surface water (without unfavourable agents, especially sulphates and chlorides)	- Temperature (<60°) σ - Amount of sodium and magnesium chloride (<3%) - Amount of sulphate (<6%) - Amount of organic acids (>0.1%) - Amount of organic matter and suspended clay (<2 g/L)
ADMIXTURES	Certified (supplier)	ASTM C33/ C33 M (2018) Standard for concrete	Bentonite, fly ash, sodium silicates, barite, hematite, chlorides, magnesium and aluminium silicates	For bentonite: grain size distribution, consistency limits, pH, moisture content, settling time, Marsh viscosity.
REINFORCEMENTS	Certified (supplier)	-	Steel, fibreglass frames σ	Visual inspection: surface defects (small cracks, coating, oxidation, corrosion, etc).

Table 4-8: Quality assurance for the grout preparation.

Control of …	Parameter	Method
GROUT PREPARATION (preliminary)	Density	Baroid balance (American Petroleum Institute, 2009) Calibrated pycnometer
	Apparent viscosity	Marsh funnel test (ASTM D6910 / D6910M, 2019)
	Loss water	Bleeding test (ASTM C940, 2016)
	Setting time (also useful to confirm the right mix and composition)	Vicat needle test (ASTM C191, 2019)
	Compressive strength - development with time	Uniaxial compression tests at different times after hardening (ASTM C109 / C109M, 2016)
GROUT PREPARATION (during treatment)	Density, viscosity, loss water with the same standards reported for preliminary checks	

The drilling and grouting need a check of several treatment parameters. The most important control is the initial position and inclination of the perforation tools to ensure the continuity of the jet-grouted elements (i.e., column overlapping). The control can be performed in several ways, including topographical levelling, laser-aided systems, GPS, for the position, and pendulum or gyroscope based systems, and GPS for the inclination. In modern machinery, drilling and grouting are continuously monitored by automatic acquisition systems that include a set of periodically calibrated measuring instruments (pressure gauges, displacement transducers, load-measuring devices), a data logger and software to process data and to plot them on a computer screen. Typical parameters automatically recorded are reported in Table 4-9.

Table 4-9: Typical parameters automatically recorded during the execution of jet grouting.

Drilling	Grouting
Advancing rate Rotary speed Tool pressure Torque Drilling mud flow rate and pressure	Withdrawal speed Rotary speed Air pressure Water flow rate and pressure Grout flow rate and pressure Injected volume of grout

During construction, it is also important to control the amount, continuity and quality of the spoil returning to the borehole's head. Unexpected reductions in spoil return may be indicators of a critical and undesired malfunctioning of the jet action. In case of negligible spoil return, it is compulsory to check that there is no clogging of the borehole annulus.

4.11.2 Quality control

Quality control for jet grouting can be subdivided in the following complementary categories (Table 4-10):

- geometrical and mechanical properties of the jet-grouted elements;
- performance of the jet-grouted elements and structures.

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Table 4-10: Quality control for the grout preparation.

Control of …	Parameter	Direct/Indirect Method	Method
Geometrical and mechanical properties of jet-grouted elements	Diameter of columns (Average diameter and variations)	Direct (trial purposes)	DISCOVERY OF COLUMNS
			HYDRAULIC CALLIPER Inserting the tool at different levels into the freshly injected column, varying the pressure of the fluid and measuring the volume in the jack chamber. An increase in resistance, is noticed when the arms reach the undisturbed soil, that is, the sides of the column.
		Indirect (trial purposes – during execution)	INSPECTION HOLES Visual inspection of coloured pipes/ temperature measurements/ noise measurements.
			SONIC LOGGING TEST (ASTM D5753, 2018) The wave is triggered from a source and recorded by a sequence of receivers placed within the same borehole at the centre of the column.
			SEISMIC TOMOGRAPHY Shear waves velocity v_s measurements using borehole outside the treated volume.
	Continuity and homogeneity	Direct	CORING SAMPLES FROM BOREHOLES RQD Index (ASTM D6032 / D6032M , 2017), Core Recovery Index (Yoshitake et al., 2003)
		Indirect	INSTRUMENTED DRILLING (ASTM D5434, 2012)
			DYNAMIC TESTS determining the propagation velocity of compression and shear waves: - SONIC LOGGING TEST (ASTM D5753, 2018) - CROSS-HOLE TESTS (ASTM D6760, 2016) - SEISMIC TOMOGRAPHY
	Physical and mechanical properties	Direct	LABORATORY TESTS usually performed on cylindrical specimens cored from treated elements (sometimes cubic specimens): - DRY UNIT WEIGHT (ASTM D7263-09, 2018) - WATER CONTENT (ASTM D2216, 2019) - SHEAR STRENGTH: Referring to Mohr-Coulomb criterion: triaxial (ASTM D7012 - 14e1, 2014) / Referring to Tresca criterion: (ASTM C39 / C39M, 2018), (ASTM D2166 / D2166M, 2016), (ASTM D7012 - 14e1, 2014). ON SITE: Density with Nuclear Methods (ASTM D5195, 2014)
		Indirect	After calibration with direct methods PENETROMETER TESTS, PRESSIOMETER TESTS, WAVE VELOCITY
Performance of elements and structures		Direct	Isolated foundation reinforcements: LOAD TESTS (ASTM D1143 / D1143M - 07(2013)e1, 2013) Sealing barriers: PERMEABILITY TESTS (ASTM D5084 - 16a, 2016), (ASTM D4750, 2001)

4.11.3 Field trial

During the field trials, with the aim of calibrating the execution procedure and pursuing cost effectiveness, given the specific design constraints, the following characteristics of the treatment procedure can be tested:

- different jet grouting systems (single, double or triple fluid);
- different combinations of treatment parameters.

This goal can be attained by producing prototype columns and by comparing the results obtained. Whenever possible, the surrounding soil should be excavated after treatment to measure the column diameter and to check the continuity of the jet-grouted material.

The geometrical and mechanical properties of each prototype column (diameter, axis direction, compressive strength, etc.) should be measured by retrieving a sufficient amount of data to apply statistical criteria that are needed to quantify the variability produced by the coupling of each treatment procedure with the subsurface properties. The overall performance of the treated area should be also tested.

5. CONCLUSIONS

Given the above defined ground improvement methods to mitigate liquefaction, the choice of the optimal solution must come out for each application from a series of consideration involving technical efficiency, feasibility of treatments in relation with the scope of the project and the existing boundary conditions, taking into account also environmental issues and, last but not least, cost effectiveness. A classification of the considered methods considering all these aspects is summarised in Table 5-1 where a score is given to each technique. This classification serves for a primary judgement on the suitability of the different techniques to different possible situations and as a tool for the first selection of mitigation strategies following a risk analysis. Therefore, a grade and a relative weight are firstly given to the following fields, considering their relevance for the project:

- *Site conditions*, considering if the ground improvement concerns free field or is addressed to existing buildings/infrastructures, distinguishing in this case if the structure is in operation or out of order;
- *Subsoil characteristics*, distinguishing the type of soil to be treated (with the presence of fine component), stratigraphy (crust of non-liquefiable soil), depth of the portion to be treated (<3m; 3-12 m; 12-18 m; 18-25 m);
- *Extension of the ground* to be treated (<1000 m^2; 1000-5000 m^2; >5000 m^2);
- *Foundation type* of the building/infrastructure under concern (shallow or deep);
- *Constraints* like presence of buildings nearby (presence of buildings or utilities);
- *Environmental restraints*;
- *Cost*.

Then the considered ground improvement methods have been evaluated with reference to each of the above issues. For instance, use of impactant techniques like deep dynamic compaction or blasting is discouraged near existing buildings giving a nil grade. Finally, each grade given to a technique with reference to a specific issue is weighted for the relevance of the issue. In this way, a score is obtained for all the techniques, that drives the stakeholders (managers, technical and non-technical personnel) to get oriented on the most suitable solution.

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Table 5-1: Evaluation of ground improvement methods for liquefaction mitigation.

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
RELEVANCE	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

List of G.I. technologies:
Deep dynamic compaction
Vibro compaction
Blasting compaction
Compaction grouting
Low pressure grouting
Earthquake drains
Induced partial saturation
Vibro replacement
Deep soil mixing
Jet grouting

Question		RELEVANCE	Relative weight	DEEP DYNAMIC COMPACTION		VIBRO COMPACTION				BLASTING COMPACTION		COMPACTION GROUTING		LOW PRESSURE GROUTING		EARTHQUAKE DRAINS				INDUCED PARTIAL SATURATION				VIBRO REPLACEMENT		DEEP MIXING		JET GROUTING	
						Deep		Shallow / Replacement								Vertical		Horizontal		Vertical		Horizontal							
			(%)	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score	Applicability	Weighted score
1. Site conditions	1.1) Free field	4	16.0	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48	3	48
	1.2) Existing buildings in operation			0	0	0	0	0	0	0	0	1	16	1	16	1	16	3	48	1	16	3	48	0	0	0	0	1	16
	1.3) Existing buildings out of order			0	0	0	0	0	0	0	0	2	32	2	32	2	32	3	48	2	32	3	48	0	0	0	0	2	32
2. Soil type	2.1) Gravel soils	4	16.0	2	32	2	32	2	32	2	32	2	32	3	48	1	16	1	16	2	32	2	32	1	16	1	16	3	48
	2.2) Sandy soils			3	48	3	48	3	48	2	32	3	48	3	48	3	48	3	48	3	48	3	48	2	32	2	32	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity			1	16	0	0	0	0	0	0	1	16	0	0	1	16	1	16	1	16	1	16	3	48	3	48	2	32
3. Stratigraphy	3.1) Soil crust	2	8.0	1	8	2	16	3	24	1	8	3	24	3	24	3	24	3	24	3	24	3	24	2	16	3	24	3	24
	3.2) No soil crust			3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24	3	24
	3.3) Layered liq/non liq soils			1	8	2	16	3	24	1	8	1	8	3	24	3	24	0	0	3	24	3	24	3	24	3	24	2	16
4. Depth of the zone to be treated (based on case histories)	4.1) <3 m	4	16.0	3	48	3	48	3	48	2	32	1	16	1	16	1	16	2	32	3	48	1	16	3	48	3	48	2	32
	4.2) 3-12 m			3	48	3	48	1	16	3	48	3	48	3	48	2	32	3	48	3	48	3	48	3	48	3	48	3	48
	4.3) 12-18 m			1	16	2	32	0	0	2	32	3	48	3	48	3	48	2	32	3	48	2	32	2	32	3	48	3	48
	4.4) 18-25 m			0	0	1	16	0	0	0	0	2	32	2	32	2	32	1	16	3	48	1	16	1	16	2	32	3	48
5. Size of area to be improved	5.1) Small (<1000 m^2)	1	4.0	0	0	0	0	3	12	0	0	3	12	3	12	3	12	3	12	3	12	3	12	0	0	3	12	3	12
	5.2) Medium (1000-5000 m^2)			1	4	1	4	2	8	1	4	3	12	3	12	3	12	2	8	3	12	2	8	1	4	3	12	3	12
	5.3) High (>5000 m^2)			3	12	3	12	1	4	3	12	3	12	3	12	3	12	1	4	3	12	1	4	3	12	3	12	3	12
6. Foundation type	6.1) Shallow foundations	1	4.0	0	0	3	12	3	12	1	4	3	12	3	12	3	12	3	12	3	12	3	12	3	12	3	12	3	12
	6.2) Deep foundations			0	0	1	4	0	0	0	0	1	4	3	12	3	12	1	4	3	12	3	12	0	0	3	12	2	8
7. Project constrains	7.1) Low overhead clearance	2	8.0	0	0	0	0	0	0	0	0	2	16	3	24	2	16	3	24	2	16	3	24	0	0	0	0	2	16
	7.2) Adjacent structures			0	0	0	0	1	8	0	0	3	24	3	24	3	24	2	16	3	24	2	16	0	0	3	24	3	24
	7.3) Existing utilities			0	0	0	0	1	8	0	0	2	16	2	16	2	16	2	16	2	16	2	16	0	0	0	0	2	16
8. Presence of subsurface obstructions		2	8.0	0	0	0	0	0	0	0	0	2	16	2	16	2	16	3	24	2	16	3	24	0	0	1	8	2	16
9. Environmental compatibility		2	8.0	3	24	3	24	3	24	1	8	2	16	2	16	3	24	3	24	3	24	3	24	3	24	2	16	2	16
10. Cost (per sq. m. of treated area)		3	12.0	3	24	3	24	1	8	3	24	1	8	1	8	2	16	2	24	2	16	2	16	3	24	1	8	1	8
Total			100.0																										

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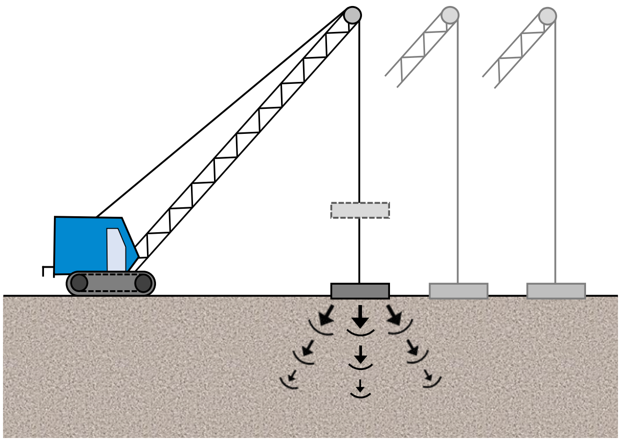
APPENDIX A "Technical Charts"

DEEP DYNAMIC COMPACTION
Technical Chart

Deep Dynamic Compaction (DDC) is a technique that densifies the soil by means of high energy tamping, using a weight repeatedly dropped on the ground surface. In saturated cohesionless soils, the drops induce increasing pore water pressure and liquefaction, resulting in a denser configuration after dissipation of pore water pressure. The treatment provides increased mechanical properties and liquefaction resistance.



EFFECT		
	DENSIFICATION	✓
	STABILISATION	
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	✓ *
	REINFORCEMENT	

* dynamic deep replacement

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	
SUITABLE SOILS	Best treatment for granular materials Treatment possible for mixed, cohesive and refuse-contaminated soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Increased density • Reduced deformability • Higher strength • Reduced permeability • Reduced liquefaction potential	• Additional number of blows can be required in the presence of obstructions • The treatment apparatus requires sufficient headroom • DDC causes vibration, noises and possible lateral movements • DDC can disperse contaminants: barrier layers have not to be damaged and possible changes in water level have to be considered

TREATMENT PARAMETERS

- Tamper mass and size
- Number of passes

For each pass:
- Grid spacing
- Drop height
- Number of drops

QUALITY ASSURANCE / QUALITY CONTROL

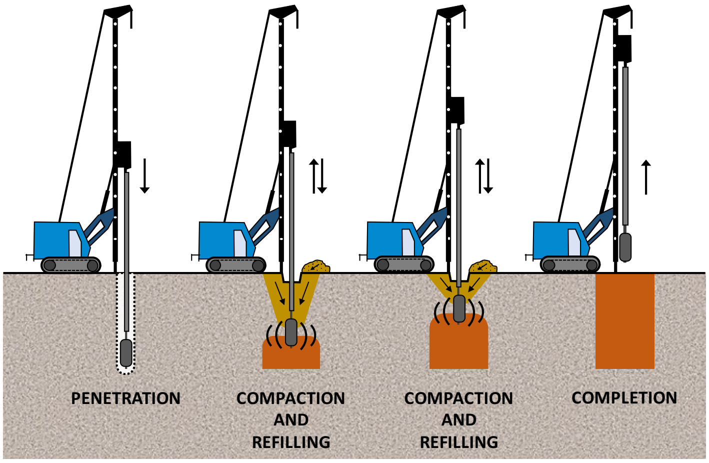
QA	• Size and mass of the tamper, drop height, number of drops, position of drops, number of tamping passes, average energy applied. • The degree of improvement can be assessed with the same in situ tests used for QC performed between tamping passes.
QC	CPT, SPT, Shear wave velocity measurement (Cross-hole tests, Seismic tomography), Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	0	0
	1.3) Existing buildings out of order	0	0
2. SOIL TYPE	2.1) Gravel soils	2	32
	2.2) Sandy soils	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	1	16
3. STRATIGRAPHY	3.1) Soil crust	1	8
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	1	8
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	3	48
	4.2) 3-12 m	3	48
	4.3) 12-18 m	1	16
	4.4) 18-25 m	0	0
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	0	0
	5.2) Medium (1000-5000 m^2)	1	4
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	0	0
	6.2) Deep foundations	0	0
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	0	0
	7.2) Adjacent structures	0	0
	7.3) Existing utilities	0	0
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		0	0
9. ENVIRONMENTAL COMPATIBILITY		3	24
9. COST (PER sq. m. OF TREATED AREA)		3	24

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

VIBRO COMPACTION (Deep)
Technical Chart

Vibro compaction (VC) is a deep compaction technique for granular soils that involves the use of a depth vibrator. Once the vibrator penetrates at the required depth, accompanied by air and/or water jetting, the horizontal vibrations allow a denser configuration of the soil (relative density 70-85%). The treatments are carried out at prescribed intervals retracting the vibrator to the top. During compaction, additional backfill is added from the top to fill the depression. The treatment provides increased mechanical properties to the soils and increased liquefaction resistance.



EFFECT		
	DENSIFICATION	✓
	STABILISATION	
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	
SUITABLE SOILS	Well graded gravel and sand	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Homogeneity • Higher density • Higher shear strength • Reduced compressibility • Reduced permeability • Reduced liquefaction potential	• Fine fraction inhibits the treatments • The low permeability of the soils to be treated leads a slow rate of penetration • The high permeability of the soils to be treated causes a loss water that can obstruct the penetration of the vibrator • VC can disperse contaminants • The turbid water coming from the penetration process should be purified from the sediments before being discharged • Backfill: in situ materials for treatments in coarser soils and imported coarser materials for treatments in finer sand • Potential sources of noises and settlements • VC apparatus requires sufficient headroom

TREATMENT PARAMETERS

- Penetration depth of the probe
- Mean extraction intervals
- Vibration frequency
- Duration of compaction
- Pressure of the water/air jets
- Grid of treatment

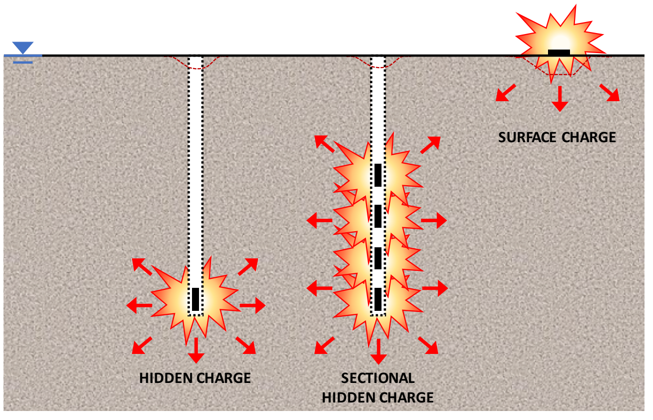
QUALITY ASSURANCE / QUALITY CONTROL	
QA	Quantity and quality of backfill added, location of the compaction points, probe penetration rate and probe withdrawal rate, vibrator amperage draw, treatment depth, water management.
QC	CPT, SPT, Shear wave velocity measurement (Cross-hole tests, Seismic tomography), Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	0	0
	1.3) Existing buildings out of order	0	0
2. SOIL TYPE	2.1) Gravel soils	2	32
	2.2) Sandy soils	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	0	0
3. STRATIGRAPHY	3.1) Soil crust	2	16
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	2	16
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	3	48
	4.2) 3-12 m	3	48
	4.3) 12-18 m	2	32
	4.4) 18-25 m	1	16
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	0	0
	5.2) Medium (1000-5000 m^2)	1	4
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	1	4
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	0	0
	7.2) Adjacent structures	0	0
	7.3) Existing utilities	0	0
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		0	0
9. ENVIRONMENTAL COMPATIBILITY		3	24
9. COST (PER sq. m. OF TREATED AREA)		3	24

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

BLASTING COMPACTION
Technical Chart

Blasting compaction, or explosive compaction (EC), employs the detonation of explosive charges to densify the surrounding soil. In saturated loose sand deposits the process cause liquefaction, displacement, remoulding and settlement. The treatment is also possible in soft fine-grained soils to improve the drainage by filling the cavity caused by the explosion with granular materials (sand columns). The charges can be placed on the ground surface or at depth, usually activated in sectional explosions. Several passes are usually adopted to increase the densification.



EFFECT		
	DENSIFICATION	✓
	STABILISATION	
	DRAINAGE	✓ *
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	

*= for treatment in soft fine-grained soils with sand columns formation

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	
SUITABLE SOILS	Cohesionless soils are the most suitable Treatment possible in soft fine-grained soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Higher strength and stiffness • Reduced compressibility and permeability • Improved drainage for treatment in fine-grained soils • Reduced liquefaction potential	• Clay particles can reduce the efficiency of the treatment • High environmental impact for the emission of noise, vibration, gases and fumes • The treatment cause settlement of the surrounding ground

TREATMENT PARAMETERS

- Charge in each hole
- Depth of charge
- Scattering pattern of charges (in height)
- Distance between the holes
- Phasing and number of blast stages
- Sequence of explosions

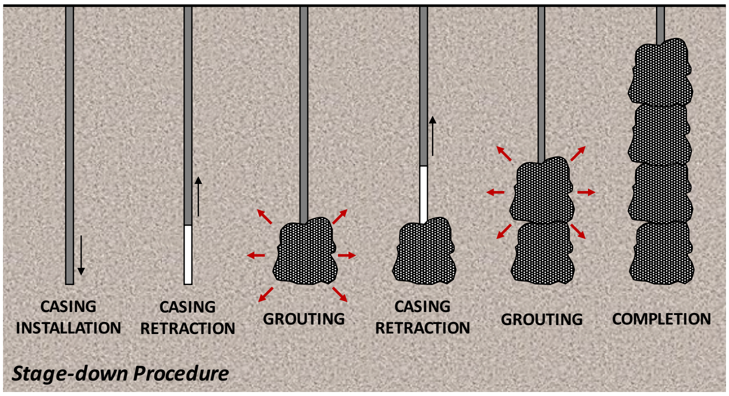
QUALITY ASSURANCE / QUALITY CONTROL	
QA	Type of explosive, blasting procedure and equipment, depth of charge, vertical distance between charges, distance between the holes, phasing and number of blast stages, sequence of explosions.
QC	CPT, SPT, Shear wave velocity measurement (Cross-hole tests, Seismic tomography), Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	0	0
	1.3) Existing buildings out of order	0	0
2. SOIL TYPE	2.1) Gravel soils	2	32
	2.2) Sandy soils	2	32
	2.3) Inorganic silts, clays silts of low to medium plasticity	0	0
3. STRATIGRAPHY	3.1) Soil crust	1	8
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	1	8
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	2	32
	4.2) 3-12 m	3	48
	4.3) 12-18 m	2	32
	4.4) 18-25 m	0	0
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	0	0
	5.2) Medium (1000-5000 m^2)	1	4
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	1	4
	6.2) Deep foundations	0	0
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	0	0
	7.2) Adjacent structures	0	0
	7.3) Existing utilities	0	0
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		0	0
9. ENVIRONMENTAL COMPATIBILITY		1	8
9. COST (PER sq. m. OF TREATED AREA)		3	24

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

COMPACTION GROUTING
Technical Chart

In Compaction grouting technique, a very stiff grout (soil-cement-water mixtures with high viscosity) is injected into the ground without permeate into the native soil causing a growth of the bulb of the injected grout that displaces and compacts the surrounding soil. Moreover, the grout injected can represent a reinforcement for the treated area. The grout is usually injected from open-end pipes into pre-drilled hole in several stages, from the top to the bottom (Stage-down procedure) or vice versa (Stage-down procedure) resulting in a column of connected grout bulbs.



Stage-down Procedure

EFFECT		
	DENSIFICATION	✓
	STABILISATION	
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	✓

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	Loose cohesionless soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Increased density • Reduced deformability • Higher strength • Reduced permeability • Reduced liquefaction potential	• Clay fraction may reduce the effectiveness of the treatment • The treatment is influenced by confinement pressure • Shallow soils are not suitable for the treatment for the low confining pressure (ground heaves) • Heaves can occur for shallow treatments and treatments in natural dense, or already compacted, soils • Unbalanced injection rate (high) and permeability (low) can cause fracturing (ineffective treatment) • The treatment can cause ground movements, changes of groundwater level, spreading of grout, pollution of groundwater, distribution of dust

TREATMENT PARAMETERS
• Grout composition • Grout hole spacing (grid of treatment) • Maximum depth of treatment • Grouting stage length • Injection pipe diameter • Injection rate • Limiting injection pressure • Injected volume

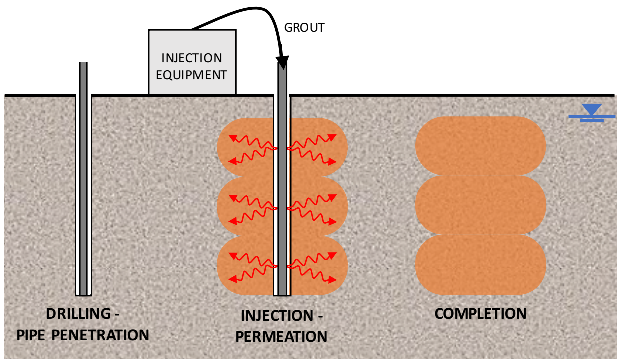
QUALITY ASSURANCE / QUALITY CONTROL	
QA	• Quality of the grout, quality of the base materials, proportions of the components • Drilling of boreholes • Injection of the grout
QC	CPT, SPT, Shear wave velocity measurement (Cross-hole tests, Seismic tomography), Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	1	16
	1.3) Existing buildings out of order	2	32
2. SOIL TYPE	2.1) Gravel soils	2	32
	2.2) Sandy soils	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	1	16
3. STRATIGRAPHY	3.1) Soil crust	3	24
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	1	8
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	1	16
	4.2) 3-12 m	3	48
	4.3) 12-18 m	3	48
	4.4) 18-25 m	2	32
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	3	12
	5.2) Medium (1000-5000 m^2)	3	12
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	1	4
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	2	16
	7.2) Adjacent structures	3	24
	7.3) Existing utilities	2	16
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		2	16
9. ENVIRONMENTAL COMPATIBILITY		2	16
9. COST (PER sq. m. OF TREATED AREA)		1	8

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

LOW PRESSURE GROUTING
Technical Chart

Low-pressure grouting (or permeation grouting) consists of low pressure injections of grout (suspensions, solutions or mortars) in the soil without altering the original structure, filling most of the porosity (70÷80%). During the treatment, the injection pressure is kept below the value that causes the fracture of the soil. The grout penetrates the soil filling the voids and gels/hardens resulting in a bond with the soil particles. The treated soil has increased mechanical properties and reduced interconnected porosity. In liquefiable soils, this process reduces the volume contraction of the soil and thus the generation of excess pore water pressure, preventing liquefaction.



EFFECT		
	DENSIFICATION	
	STABILISATION	✓
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	From silt to gravel	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Reduced deformability • Increased cohesion • Higher shear and compressive strength • Reduced permeability • Reduced interconnected porosity • Reduced liquefaction potential	• Uncertainly of the result obtained (extension/mechanical properties) • Ungroutable lenses may reduce the effectiveness of the treatment • There are some concerns about the permanence in time of the some types of grout • Excessive dilution may prolong the setting time and inhibit chemical reactions • Several grouts are not stable with time or the rheological and mechanical properties change with time • Some chemical grouts can cause pollution or can be toxic • The treatment can cause ground movement, changes of groundwater level, spreading of grout, pollution of groundwater, distribution of dust

TREATMENT PARAMETERS

- Type of drilling
- Type of injection pipes
- Grout composition (types and proportions) and characteristics
- Number of passes
- Grout volume to be injected, pressure and duration for each pass

Moreover:

- Grid of treatment
- Maximum depth of treatment

QUALITY ASSURANCE / QUALITY CONTROL

QUALITY ASSURANCE / QUALITY CONTROL	
QA	Quality of the grout, quality of the base materials, proportions of the components, drilling of boreholes, injection of the grout.
QC	Common tests: • Shear wave velocity measurement (e.g. Cross-hole tests, Seismic tomography), CPT, SPT. Other possible tests: • Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

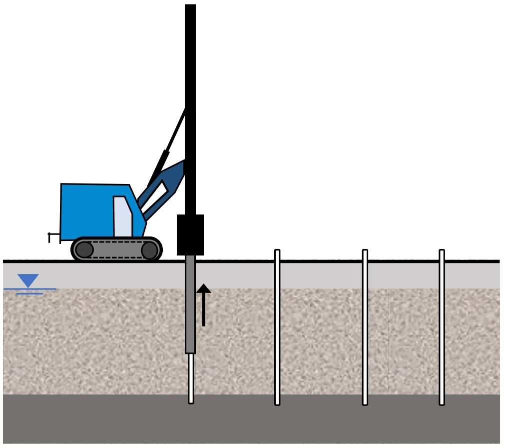
		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	1	16
	1.3) Existing buildings out of order	2	32
2. SOIL TYPE	2.1) Gravel soils	3	48
	2.2) Sandy soils	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	0	0
3. STRATIGRAPHY	3.1) Soil crust	3	24
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	3	24
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	1	16
	4.2) 3-12 m	3	48
	4.3) 12-18 m	3	48
	4.4) 18-25 m	2	32
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	3	12
	5.2) Medium (1000-5000 m^2)	3	12
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	3	12
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	3	24
	7.2) Adjacent structures	3	24
	7.3) Existing utilities	2	16
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		2	16
9. ENVIRONMENTAL COMPATIBILITY		2	16
9. COST (PER sq. m. OF TREATED AREA)		1	8

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

EARTHQUAKE DRAINS

Technical Chart

The Earthquake (EQ) drains are prefabricated vertical drains with high flow capacity. The EQ drains consist of perforated corrugate plastic pipes sheathed in a geosynthetic filter to prevent the particles flow into the drain. The drain is installed by means of a mandrel, inserted by vibration and removed at the end of the procedure and fixes by an anchor fixed base. The EQ drains provide a dissipation of pore water pressure excess generated into saturated cohesionless soils during the earthquake before liquefaction occurs. For this reason, they can reduce the liquefaction potential of susceptible soils.



EFFECT		
	DENSIFICATION	
	STABILISATION	
	DRAINAGE	✓
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	Saturated cohesionless soils (liquefaction mitigation) Highly compressible soils with low permeability (To faster the consolidation process)	

BENEFITS

- Dissipation of pore water pressure excess during earthquake
- Reduced consolidation time in low permeability soils
- Reduced liquefaction potential

LIMITATIONS AND DRAWBACKS

- It is not easy to evaluate the effectiveness of the treatment
- The installation procedure can cause a soil disturbance that can be reduced by adopting smaller mandrel
- Installation by static pushing is preferred in sensitive soils but could cause a deviation of the mandrel
- Stiff layers or obstructions may require predrilling to penetrate at depth
- To improve a stiff stratum, the predrilling is not advisable causing a soil disturbance
- In very soft layers is difficult to anchor the drains, additional depth may be necessary
- Treatment could not be economically on slope and on not regular and unstable surface
- Depth drains require specialised installation apparatus
- Correct storage of the drain material to prevent degradation degradation
- Sufficient headroom: with limited space the drains can be installed in segment (increasing the cost)
- Treatment of water in contaminates sites
- In contaminates sites the drains should not penetrate into highly permeable layers
- Vertical drains can be installed near existing structures, but not below

TREATMENT PARAMETERS

- Flow capacity of the drain
- Dimension of the drain
- Mandrel dimension
- Installation method
- Anchor depth
- Grid of treatment

QUALITY ASSURANCE / QUALITY CONTROL

QA
- Dimensions of the core , durability , tensile strength and elongation of the drain, strength of the seam, discharge capacity and filtration characteristics, visual inspections for damage, tensile strength per unit width of filter, velocity index of filter, pore size of filter.

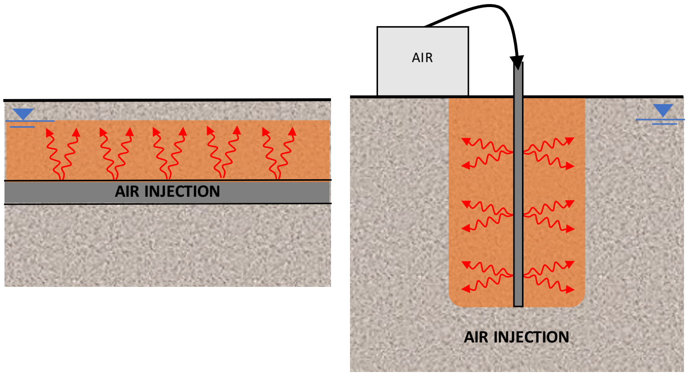
QC
- Identification of prefabricated drains on site (according to specification).
- Measurement of settlements (settlement gauges) and pore water pressure (piezometers).

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	1	16
	1.3) Existing buildings out of order	2	32
2. SOIL TYPE	2.1) Gravel soils	1	16
	2.2) Sandy soils	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	1	16
3. STRATIGRAPHY	3.1) Soil crust	3	24
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	3	24
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	1	16
	4.2) 3-12 m	2	32
	4.3) 12-18 m	3	48
	4.4) 18-25 m	2	32
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	3	12
	5.2) Medium (1000-5000 m^2)	3	12
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	3	12
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	2	16
	7.2) Adjacent structures	3	24
	7.3) Existing utilities	2	16
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		2	16
9. ENVIRONMENTAL COMPATIBILITY		3	24
9. COST (PER sq. m. OF TREATED AREA)		2	16

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

INDUCED PARTIAL SATURATION
Technical Chart

Induced Partial Saturation (IPS) consists in introducing gas bubbles into the soil reducing the degree of saturation and increasing the liquefaction resistance. Several methods have been proposed to introduce gas bubbles into the sand, including air injection, sand compaction pile, use of sodium perborate, electrolysis and drainage-recharge. IPS is an innovative technique that is still little used today although it has many advantages (easy, cheap, environmental friendly, suitable in build-up area).



EFFECT		
	DENSIFICATION	
	STABILISATION	
	DRAINAGE	
	DESATURATION	✓
	REPLACEMENT	
	REINFORCEMENT	

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	Saturated cohesionless soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Reduced degree of saturation • Reduced liquefaction potential	• It is not easy to evaluate the effectiveness of the treatment • Clay or silt lenses can reduce the effectiveness of the treatment • An important unresolved issue for IPS is whether or not gas bubbles can remain in the soil for a long time • Periodic treatments of restoration may be required in the case of increase in the degree of saturation

TREATMENT PARAMETERS

- Depth/length of treatment
- Volume of air to be injected
- Air pressure
- Injection rate
- Injection spacing
- Treatment layout

QUALITY ASSURANCE / QUALITY CONTROL

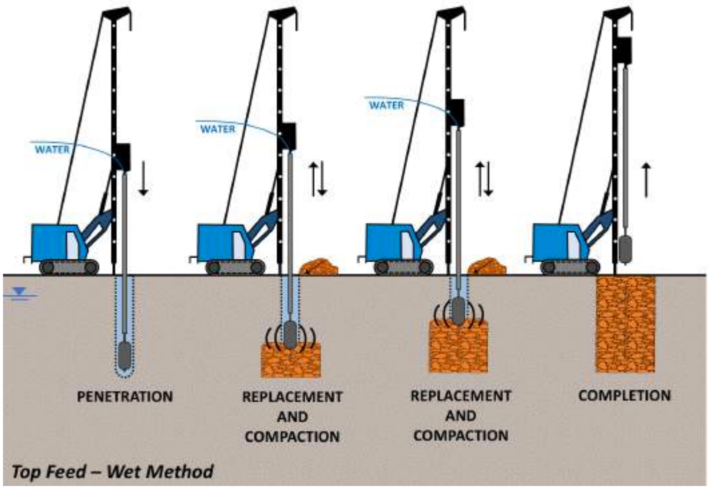
QA	• Installation of the injection pipes (horizontal or vertical), depth/length of the pipes, correct location of the pipes, injection pressure and rate of the air/gas, volume of air/gas injected.
QC	• Laboratory tests on samples taken from the treated area to assess the degree of saturation before and after treatment. • 3D electrical resistivity tomography and the bulk wave velocity V_p.
Monitoring the permanence of air bubbles over time to ensure the improvement over time and monitoring the effects on the surrounding environment and structures.	

		VERTICAL		HORIZONTAL	
		APPLICABILITY	WEIGHTED SCORE	APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48	3	48
	1.2) Existing buildings in operation	1	16	3	48
	1.3) Existing buildings out of order	2	32	3	48
2. SOIL TYPE	2.1) Gravel soils	2	32	2	32
	2.2) Sandy soils	3	48	3	48
	2.3) Inorganic silts, clays silts of low to medium plasticity	1	16	1	16
3. STRATIGRAPHY	3.1) Soil crust	3	24	3	24
	3.2) No soil crust	3	24	3	24
	3.3) Layered liq/ non liq soils	3	24	3	24
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	3	48	1	16
	4.2) 3-12 m	3	48	3	48
	4.3) 12-18 m	3	48	2	32
	4.4) 18-25 m	3	48	1	16
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	3	12	3	12
	5.2) Medium (1000-5000 m^2)	3	12	2	8
	5.3) High (>5000 m^2)	3	12	1	4
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12	3	12
	6.2) Deep foundations	3	12	3	12
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	2	16	3	24
	7.2) Adjacent structures	3	24	2	16
	7.3) Existing utilities	2	16	2	16
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		2	16	3	24
9. ENVIRONMENTAL COMPATIBILITY		3	24	3	24
9. COST (PER sq. m. OF TREATED AREA)		2	16	2	16

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

VIBRO REPLACEMENT
Technical Chart

Vibro-replacement (VR) is a deep compaction technique for cohesive soils and granular soil with high fine content that involves the use of depth vibrator. The vibrator penetrates at the required depth accompanied by a water/air jetting (*wet/dry method*) and the granular backfill is added from the top or the bottom (*top or bottom feed method*). The horizontal vibrations allow a densification of the backfill forming granular columns (DISPLACEMENT process, and for wet method also REPLACEMENT process). The treatments are carried out at prescribed intervals retracting the vibrator to the top.



EFFECT		
	DENSIFICATION	✓
	STABILISATION	
	DRAINAGE	✓
	DESATURATION	
	REPLACEMENT	✓
	REINFORCEMENT	✓

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	
SUITABLE SOILS	All types of soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Higher shear strength, reduced compressibility and higher permeability of the columns • Reinforcement of soils • Excess of pore water pressure dissipation • Reduced settlements and consolidation time • Increased mechanical properties • Reduced liquefaction potential	• A minimum strength of the in-situ soil is required to give sufficient containing pressure • For stiff soils, pre-drilling may be required for probe penetration • Generation of sludge to be treated (wet method) • Potential sources of noise and vibration, and thus can cause settlements • VR can disperse contaminants • Backfill: sufficient hardness and strength; not containing organic or deleterious materials

TREATMENT PARAMETERS
• Backfill (type and gradation) • Penetration depth of the probe • Mean extraction rate • Vibration frequency • Duration of compaction • Pressure of the water/air jets • Grid of treatment • Diameter of the columns and replacement ratio

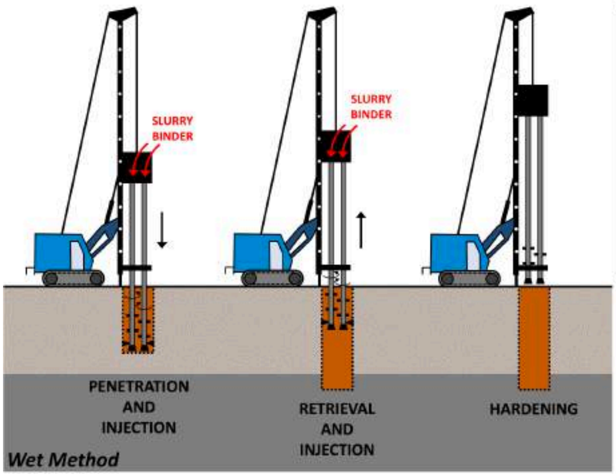
QUALITY ASSURANCE / QUALITY CONTROL	
QA	Quantity and quality of backfill added, location of the compaction points, probe penetration rate and probe withdrawal rate, vibrator amperage draw, treatment depth, water management.
QC	• Verification of the diameter of the column (verification of the displacement-replacement process), records of the installation depth, volume of backfill installed. • CPT, SPT, Shear wave velocity measurement (Cross-hole tests, Seismic tomography), Loading test, Pressumeter test, Dynamic cone test, Dilatometer test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

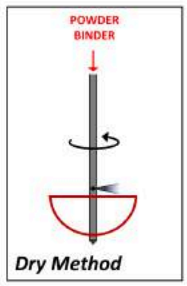
		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	0	0
	1.3) Existing buildings out of order	0	0
2. SOIL TYPE	2.1) Gravel soils	1	16
	2.2) Sandy soils	2	32
	2.3) Inorganic silts, clays silts of low to medium plasticity	3	48
3. STRATIGRAPHY	3.1) Soil crust	2	16
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	3	24
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	3	48
	4.2) 3-12 m	3	48
	4.3) 12-18 m	2	32
	4.4) 18-25 m	1	16
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	0	0
	5.2) Medium (1000-5000 m^2)	1	4
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	0	0
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	0	0
	7.2) Adjacent structures	0	0
	7.3) Existing utilities	0	0
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		0	0
9. ENVIRONMENTAL COMPATIBILITY		3	24
9. COST (PER sq. m. OF TREATED AREA)		3	24

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

DEEP MIXING
Technical Chart

Deep mixing method (DMM) involves mixing at depth of in-situ soil with binder materials, such as cement, lime, fly ash, slag or other types of binder. The improvement is based on the chemical interactions of the clayey soils with the binder, the bond between the particles and the filling of the voids with the products of the reactions. The most common procedure uses vertical shafts mixing tools to form columns. The binder can be injected in dry form (dry method) or like slurry (wet method) and mixed with the in-situ soil during the penetration or/and the withdrawal of the mixing tool.





EFFECT		
	DENSIFICATION	
	STABILISATION	✓
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	✓

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	Wet method: Coarse and fine-grained soils, organic soils and peat. Dry method: Soft fine-grained soils, organic soils and peat.	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Higher strength and stiffness • Reduced compressibility and permeability • Stabilisation of contaminants • Reduced liquefaction potential	• Not suitable for very stiff soils/very dense conditions • Soil obstructions can interfere with the penetration of the apparatus • Soft soils may require a work platform • Some types of organics can interfere with the chemical reactions of the binder • The dry method is possible in soils with sufficient water content to allow the chemical reactions • Spoils production for the wet method (to reuse or dispose) • The wet DMM require large space for the treatment plant and a slurry plant • High treatment costs (economically advantageous to treat large areas)

TREATMENT PARAMETERS

- Air pressure
- Slurry pressure (for wet method)
- Penetration rate and retrieval rate
- Rotation speed (penetration and retrieval)
- Quantity of binder per meter of depth (penetration and retrieval)

Moreover:

- Grid of treatment
- Maximum depth of treatment

QUALITY ASSURANCE / QUALITY CONTROL

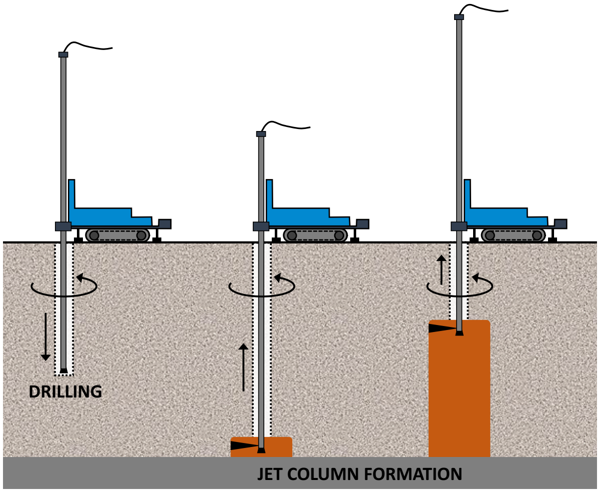
QA	• Quality of the binder and water/binder proportion, delivery of the binder, penetration/withdrawal and mixing operations. • For each column: length of the treatment, penetration and withdrawal rates of the mixing tool, rotational speed and torque of the mixing tool, overlapping width and rate and pressure of delivery of binder/slurry, quantity of binder/slurry along the column and total consumption.
QC	• Laboratory tests: mechanical and hydraulic tests on core samples and wet grabs samples. • Field tests: Pressumeter test, Pressumeter-permeameter test, CPT, Static/dynamic penetration tests, Column penetration test.
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
1. SITE CONDITIONS	1.1) Free field	3	48
	1.2) Existing buildings in operation	0	0
	1.3) Existing buildings out of order	0	0
2. SOIL TYPE	2.1) Gravel soils	1	16
	2.2) Sandy soils	2	32
	2.3) Inorganic silts, clays silts of low to medium plasticity	3	48
3. STRATIGRAPHY	3.1) Soil crust	3	24
	3.2) No soil crust	3	24
	3.3) Layered liq/ non liq soils	3	24
4. DEPTH OF THE ZONE TO BE TREATED (BASED ON CASE HISTORIES)	4.1) <3 m	3	48
	4.2) 3-12 m	3	48
	4.3) 12-18 m	3	48
	4.4) 18-25 m	2	32
5. SIZE OF AREA TO BE IMPROVED	5.1) Small (<1000 m^2)	3	12
	5.2) Medium (1000-5000 m^2)	3	12
	5.3) High (>5000 m^2)	3	12
6. FOUNDATION TYPE	6.1) Shallow foundations	3	12
	6.2) Deep foundations	3	12
7. PROJECT CONSTRAINS	7.1) Low overhead clearance	0	0
	7.2) Adjacent structures	3	24
	7.3) Existing utilities	0	0
8. PRESENCE OF SUBSURFACE OBSTRUCTIONS		1	8
9. ENVIRONMENTAL COMPATIBILITY		2	16
9. COST (PER sq. m. OF TREATED AREA)		1	8

LEGEND		
APPLICABILITY	Good	3
	Medium	2
	Low	1
	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0

JET GROUTING
Technical Chart

Jet grouting (JG) technique uses high-pressure jets of water-cement grout/water/air to break the soil structure and mix the native soil with the grout to form an improved material known as "soilcrete".
On the base of the number of fluids injected, three conventional types of system are available: *single* (grout), *double* (grout and air) and *triple fluid systems* (grout, air and water). Different shapes of the treated volume can be obtained by adjusting the translation and rotation of the monitor and combining several elements (cellular structures).



EFFECT		
	DENSIFICATION	
	STABILISATION	✓
	DRAINAGE	
	DESATURATION	
	REPLACEMENT	
	REINFORCEMENT	✓

APPLICABILITY		
STRUCTURES	NEW STRUCTURES	✓
	EXISTING STRUCTURES	✓
SUITABLE SOILS	From gravel to cohesive soils	

BENEFITS	LIMITATIONS AND DRAWBACKS
• Higher strength • Higher stiffness • Cutoff groundwater • Cutoff contaminants	• Uncertainties on the dimensions and properties of the treated elements • Possible defects of treatment: insufficient overlapping or vertical deviation of the elements or inhomogeneous soil conditions • Treated soil has not adequate resistance to horizontal actions (possible reinforcement) • Spoil production (waste) • JG can cause vibration and noises • JG can disperse contaminants

TREATMENT PARAMETERS	
FUNDAMENTAL	• **Geometrical**: number of nozzles, nozzle diameter • **Kinematic**: time interval per step, rotational velocity, lifting step • **Injected fluids**: W-C ratio by weight, fluid pressure, fluid flow rate
DERIVED	• Average lifting speed of the monitor • Monitor rotation for each lifting step • Injected grout volume per treatment unit length • Mass of injected cement per treatment unit length

QUALITY ASSURANCE / QUALITY CONTROL	
QA	• Quality of the materials used (cement, water, admixtures, possible reinforcements). • Construction procedure and equipment (grout preparation, drilling, grouting, inclination and orientation, spoil returns).
QC	• Quantify the properties of the elements (dimension, continuity, homogeneity, physical and mechanical properties). • Verify the performance of jet-grouted structures (load tests, permeability tests).
Monitoring of vibration and noises, induced settlement/ground heave and pore water pressure evolution over time.	

		APPLICABILITY	WEIGHTED SCORE
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	1.2) Existing buildings in operation	1	16
	1.3) Existing buildings out of order	2	32
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	Not applicable	0
WEIGHT	Very important	4
	Important	3
	Medium important	2
	Less important	1
	Not applicable	0