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A comprehensive design procedure for pile groups in liquefiable soils

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ABSTRACT: Among the various strategies to mitigate the effects of soil liquefaction one of the most effective is the design of deep foundations. The designs of piles in liquefiable soils are too often concerned with the only axial bearing capacity, addressed by simply neglecting pile resistance in the liquefiable layer. This approach is inadequate to properly face the complexity of the problem. In the present paper we intend to examine this theme throughout its multiple aspects. We have synthesized a design procedure of analysis that has been already applied in some projects in different areas of Emilia-Romagna region characterized by high risk of liquefaction. The proposed design procedure is based on the most updated theories and design references concerning piles in liquefiable soil, such as those of Cubrinovsky, Olson & Stark, Rollins, Bhattacharya, Madabhushi and others, primarily referring to CPTu.

1 INTRODUCTION

When designing a structure interacting with the soil in a site subjected to a high risk of liquefaction during earthquake, there are many solutions that can be evaluated to reduce and mitigate this risk.

For foundations resting on liquefiable soil one of the most effective solutions is the use of piles. But, in this case, their design has to properly account for the effects induced on piles by liquefaction.

The phenomena involving a pile under seismic cyclic loading in presence of liquefiable soils are complicated. There are different important aspect to be accounted for: the change in shaft resistance, the reduction of base capacity also in deeper layer not directly liquefacting, the equilibrium instability due to the loss of lateral support from liquefied soil and the modification of the geotechnical model under liquefaction, directly conditioning the analyses of pile groups with geotechnical numerical models.

In order to properly face the design of deep foundations on liquefiable soils we have resumed the principal bibliographic studies with the aim to build a comprehensive design procedure. In the paper we will describe in detail the various step of this procedure and we will also propose a real design example in which we have already applied this process.

2 GEOTECHNICAL MODEL

In a geotechnical design the first, and maybe most important, step is the construction of an accurate and reliable geotechnical model, i.e the parameters and the constitutive laws that mathematically represent the mechanical response of the soil.

In the case of high risk of liquefaction, the geotechnical model valid under seismic conditions significantly differs from the static one because of liquefaction effects on soil properties. The liquefiable soil layer is described below in terms of both modified stiffness and strength parameters.

2.1 *Stiffness parameters*

As regards stiffness parameters we referred to the theory proposed by Cubrinovski et al. (2009). In a simplified 3-layers model, in which the central one is potentially liquefiable, the pile is modelled as a beam connected to a series of springs representing the lateral stiffness of the soil.

As expected, the stiffness offered by the liquefied soil (k_2) is significantly lower than the one of the same non-liquefied soil (k_1). The results observed in full-scale tests on piles show that the stiffness degradation factor $\beta_2 = k_1/k_2$ typically varies in a range of 1/50 ÷1/10 for cyclic liquefaction (Figure 1). In our design procedure we chose to refer to the lower bound, i.e. to $\beta_2 = 1/50$.

The degradation factor β_2 is applied to the stiffness parameters of the liquefiable soil layer, in particular to the initial tangent value of soil elastic modulus within the numerical Boundary Element Method analyses perfomed adopting a non-linear hyperbolic constitutive model. The above theory (Cubrinovski et al. 2009) was originally developed for "p-δ" curves methods: we extended the same approach to numerical BEM analyses.

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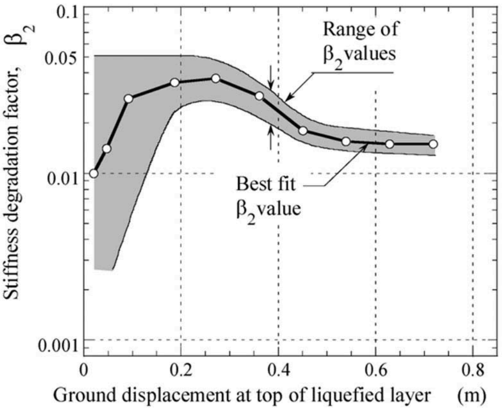

Figure 1. Degradation of stiffness in the liquefied layer observed in full-size test on piles. From Cubrinovski et al. (2009).

2.2 *Strength parameters*

The liquefiable soil is described in terms of strength parameters, in seismic conditions, via its residual undrained strength $s_{u(LIQ)}$, as proposed by Olson & Stark (2002).

$$\frac{s_{u(LIQ)}}{\sigma'_{v0}} = 0.03 + 0.0143 \cdot q_{c1} \pm 0.03 \quad \text{for } q_{c1} \leq 6.5 \text{ MPa} \tag{1}$$

The formulation proposed in Equation 1 is valid for CPT tests. As in can be seen, it is structured to define an interval of values with an amplitude of 0.03 MPa. Many researchers have studied post-seismic conditions: residual undrained strength was evaluated on the basis of the deformed configuration of the foundation after the earthquake.

From Bowen & Cubrinovski (2008) it can be seen how, in case of cyclic liquefaction, it is safe to refer to the mean value of the interval. That is what we chose to apply in the design procedure.

3 PILE AXIAL CAPACITY

Once defined the geotechnical model, next step consists in evaluating the pile axial capacity. Many different approaches can be used: correlations with strength parameters of the soil, direct correlations with in-situ soundings (CPT & CPTu, DMT, SPT, etc.), pile load tests. As concern the design procedure, we have focused on direct methods based on CPTu: in particular, the method proposed by Eslami & Fellenius (1997) improved by Niazi (2013).

3.1 *Base capacity*

The axial capacity of piles in liquefiable soils is well illustrated by Madabhushi et al. (2009).

When liquefaction occurs, the degree of growth of the pore pressure can be described by the interstitial pressure ratio ru:

$$r_u = \frac{\Delta_u}{\sigma'_{v0}} \tag{2}$$

According to Equation 2, soil shows liquefaction when $r_u = 1$. Liquefaction can easily occur in superficial sandy layers. Usually, indeed, deeper coarse grained soil layers show a higher resistance to liquefaction because their higher density and also because the greater effective pressure. So, when deep foundations are designed to resist to liquefaction, it can occurs that pile toes are placed in non-liquefiable deeper sandy layer. But also in these deeper layers pore water pressure can increase, so $r_u > 0$ and so soil resistance decrease. This fact, not immediately perceivable, has to be properly taken into account in designing piles.

Starting from the model for the base resistance of a pile (Vesic 1972) and considering the effects due to the increase of pore water pressure, Knappet & Madabhushi (2008b) showed how the tip resistance of piles in liquefiable soil is related to its corresponding value in static conditions by the relation reported in Equation (3):

$$\frac{Q_{base,E}}{Q_{base,S}} = (1 - r_u)^{\frac{3-sin\phi}{3\cdot(1+sin\phi)}} \tag{3}$$

Where r_u has been defined in Equation 2, ϕ is the angle of shearing resistance of the coarse grained soil in which the pile toe is placed while $Q_{base,E}$ and $Q_{base,S}$ are the tip bearing capacity of the pile, respectively, in seismic and static conditions.

From Equation 3 it can be seen how the resistance is related to r_u but this factor is not simple to evaluate. It would be necessary to perform advanced site effect analyses, accounting also for the liquefaction of the soil. This cannot be done in ordinary practice, so a simplified approach has been developed.

Given a liquefiable soil layer, in which liquefaction occurs ($r_u = 1$) it is assumed that excess pore pressure remain constant for depth greater than its

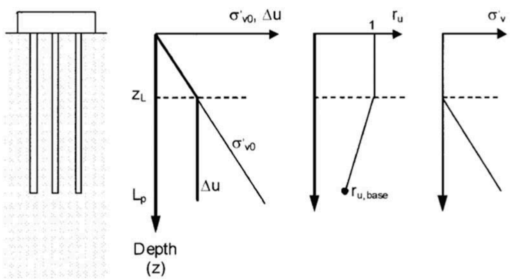

Figure 2. Effective stress conditions around piles for full liquefaction until z_L depth. From Madabhushi et al. (2009).

thickness z_L. Considering the growth of effective stresses with depth as almost linear, this lead to a bilinear shape of the factor r_u. Once the trend of r_u and the geometry of the pile are known it is easy to evaluate the interstitial pressure ratio at pile tip level L_p. It can be derived (see Figure 2).

$$r_{u,\ base} = \frac{z_L}{L_p} \tag{4}$$

3.2 *Shaft capacity*

Madabhushi et al. (2009) have extended their studies also to the shaft bearing resistance of a pile, obtaining the formulation reported in Equation 5, that follows a philosophy similar to Equation 3.

$$Q_{shaft,E} = Q_{shaft,S} \cdot (1 - r_u) = Q_{shaft,S} \cdot \frac{L_p - z_L}{L_p} \tag{5}$$

Where $Q_{shaft,E}$ and $Q_{shaft,S}$ are the shaft bearing capacity of the pile evaluated, respectively, in seismic and in static conditions.

So, in a more general way, it is possible to define the total bearing capacity of a pile in a liquefiable soil P_{bc} from Equation 6.

$$P_{bc} = Q_{base,S} \cdot (1 - r_u)^{\frac{3 - sin\phi}{3\cdot(1+sin\phi)}} + Q_{shaft,S} \cdot \frac{L_p - z_L}{L_p} \tag{6}$$

According to Equations 5 and 6, the shaft resistance of a pile in a liquefiable soil can be significantly reduced when the factor r_u increase, but it is not strictly equal to zero. Also other researches (Rollins 2015), basing on experimental data, have demonstrated that the contribution of the liquefiable layer to shaft resistance is not null, but can decrease to about 50% of its corresponding static value. Anyway, in the present design procedure we propose to completely neglect the contribution of the liquefiable soil layers to the shaft resistance of the piles.

4 PILE INSTABILITY

Liquefaction causes the loss of lateral support of the soil to the piles and the subsequently significant variation of lateral stiffness of the foundation. In these conditions piles can face a crisis for buckling.

Pile instability due to liquefaction has been studied by Bhattacharya (2003) and Bhattacharya et al. (2004) considering a simplified model in which liquefied soil has no strength and stiffness. The pile crossing this layer will behave like an axially loaded column. Under these hypotheses the critical load P_{cr} corresponding to the loss of the elastic equilibrium is given by the well-known Eulero's expression.

Anyway, experimental tests (Knappet 2006) have shown how critical loads are not so small as those predicted by theoretical formulations. This evidence has been explained with the fact that liquefied soil has a small, but not null, stiffness neglected in the theoretical model. So Eulero's expression has been updated by Madabhushi et al. (2009), accounting for finite, small, stiffness of the liquefied layer by the factor $r_{u,base}$, as reported in Equation 7:

$$P_{cr} = \frac{\pi^2 \cdot E \cdot I}{(\beta \cdot r_{u,base} \cdot h)^2} \tag{7}$$

Where E and I are elastic modulus and moment of inertia of the section of the pile, h is the length of the pile crossing the liquefiable layer and β is a factor accounting for the fixity at both ends of the pile. (β·h) represent the equivalent length of the unsupported pile tract. Introducing the radius of gyration of the section r_g, the slenderness ratio λ is defined:

$$\lambda = \frac{\beta \cdot h}{r_g} \tag{8}$$

Bhattacharya & Lombardi (2012), basing on data collected from real cases, have defined an admissible domain, bounded by a slenderness ratio λ = 50 (dashed black line in Figure 3), separating deep foundations that have shown good performances from those who have not. From Figure 3 it can be seen how the choice of the limit value of slenderness λ = 50 can be excessively precautionary. For that reason we propose to refer to a limit value equal to λ = 75 (continuous red line in Figure 3).

In reality piles can suffer buckling for axial load lower than the one predicted by Equations 7 also because imperfections not accounted in theoretical models, inducing geometrical second order problem (P-δ effect). Named δ_0 the displacement induced by the horizontal action (earthquake), the total lateral

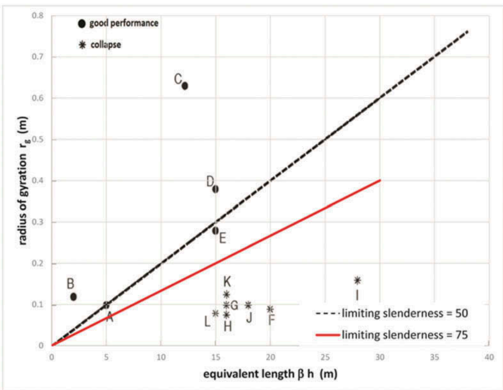

Figure 3. Admissible domain: measured performances of deep foundations after real earthquakes. From Bhattacharya & Lombardi (2012).

displacement δ in presence of a vertical action P will be greater. Equation 9 describes the normalized trend of the amplification factor δ/δ_0.

$$\frac{\delta}{\delta_0} = \frac{1}{1 - \frac{P}{P_{cr}}} \tag{9}$$

The normalized movement δ/δ_0 is almost linear for values of vertical load $\psi = P/P_{cr} \approx 0.30$ so it can be assumed:

$$P_{ult} = \psi \cdot P_{cr} \tag{10}$$

As suggested by Bhattacharya & Lombardi (2012), and also as considered in the proposed design procedure, it is adopted $\psi = 0.35$ as limit.

According to Equations 6 and 7 both the total axial bearing capacity of the pile P_{bc} and its critical load P_{cr} depend from factor r_u. So critical values of r_u can be derived, corresponding to reaching these two limiting conditions.

Following this approach Madabhushi et al. (2009) describe a series of graphs in which, depending on pile geometry (diameter D_0 and elastic properties E, I) and required factor of safety FOS, the domain of use of the pile is defined (see Figure 4). These graphs can be derived for the specific design case considering the liquefaction conditions of the site, and can be enriched by also plotting the hyperbolic trend of $r_{u,base}$, in that way also accounting for the thickness and depth of the liquefiable layer.

The piles have to be anyway verified against their structural strength because of the stresses (bending moment and shear) due by the inertial and kinematic interaction with the superstructure.

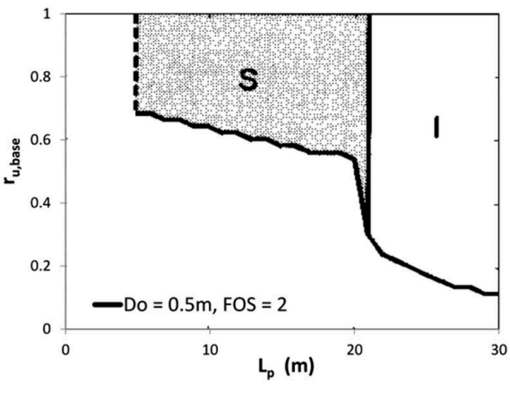

Figure 4. Design chart for a solid circular RC pile in loose sand (D_r= 35%): D_0= 0.5m, FOS= 2. From Madabhushi et al. (2009). In the blank areas the pile is verified, in those identified by “S” the pile encounter a crisis by bearing capacity while in those identified by “I” the crisis is because of instability.

5 CALCULATION METHODS

Designing a deep foundation on liquefiable soil is complex and, as seen in the previous sections, many aspects have to be properly considered to correctly face the problem. So it is clear how also the adoption of a correct method to analyze the pile group is fundamental in order to not nullify the benefits of the adopted advanced theories, as previously reported.

We used numerical BEM analyses with the software Repute, developed by Geocentric (Bond & Basile 2010, Basile 1999), adopting for the soil a non-linear hyperbolic constitutive model. In that way pile group effects and other phenomena, as pile plasticization and shadowing, can be properly accounted.

The procedures described in the previous section mainly refers to single piles. Instead we propose to extend these analyses to pile groups by adopting correct calculation methods, such as the one mentioned above, in addition to the definition of a correct geotechnical model, accounting for liquefaction (see section 2).

6 DESIGN EXAMPLE

The design procedure that we propose to design a piled foundation on liquefiable soil has been described in detail by theoretical point of view in the previous sections of the paper. Now we want to describe how this procedure has already been applied is some real case in which the authors designed deep foundations to mitigate the liquefaction risk.

The two major works in which we adopted our design procedure are the enlargement and seismic retrofit of the Cento (Ferrara, Italy) sports hall and the construction of the new Pavillion 37 as part of the revamping project of the fair quarter of Bologna (Italy). Due to the space available in the paper, we will only describe the first of these two projects.

6.1 *Investigation campaign and geotechnical characterization*

A first investigation campaign, consisting in four cone penetration tests with piezocone CPTu and seismic piezocone SCPTu, two flat dilatometer DMT tests, geophysical tests and laboratory ones, has been firstly carried out. From these data an high risk of liquefaction was found out so it was decided to perform an integrative campaign to better go insight the liquefaction problem. Four CPTu and dynamic laboratory tests (resonant column RC and cyclic triaxial TX CYC), on samples taken from two boreholes, have been performed.

Liquefaction risk has been assessed with different approaches: from CPTu (Robertson 2009), from DMT (Monaco et al. 2005), from dynamic lab tests.

The liquefaction potential index LPI, evaluated adopting the Sonmez (2003) approach, results ranging from the various investigations from about 8 to 18, identifying an high risk of liquefaction.

Liquefaction is induced by a layer of loose sands that extends from about 1 to 6 meters of depth from ground level. The geotechnical model in seismic condition accounting for liquefaction, reported in Table 1, has been defined according to what has been described in section 2.

Table 1. Geotechnical model in seismic conditions, accounting for liquefaction.

Layer	From -	To [m]	Undrained strength S_u [kPa]	Frinction angle ϕ [°]	Tangent modulus E_0 [MPa]
Liquefi-able sand	1.0	6.0	1	-	2
Clay	6.0	11.0	30	-	90
Clay	11.0	18.0	75	-	105
Sand	18.0	26.0	-	32	135
Clay	26.0	-	75	-	150

6.2 *Description of the intervention*

The project primarily consists in the construction of a new grandstand: this element, as enlargement of the existing sport hall, contains two reinforced concrete cores that are the principal part of the bracing system adopted to increase seismic resistance of the whole structure. These cores absorb the great amount of the seismic forces because they are designed to support and to brace the roof of the hall. Since there where no limits on this side for operating machines, we chose to adopt FDP (Full Displacement Piles) piles: due to the particular shape of the drilling tip the pile is realized without removing soil, so also a positive densifying effect is induced.

In the opposite side of the hall there is the existing tribune, supported by an RC frame. Also this zone has been involved in the seismic retrofitting, and the existing shallow foundation have been reinforced with micropiles designed according to the proposed procedure. Here they will not be discussed.

6.3 *Foundation analysis*

Given the loads acting on foundation from superstructures and defined the geotechnical properties of the soil, each foundation has been analyzed adopting a numerical approach: BEM analyses have been carried out with the software Repute (see section 5).

For each seismic core a deep foundation consisting in 24 FDP piles, diameter 600mm, with a length of 22.0 meters (depth of pile tip from g.l.) has been adopted. The pile cap have dimensions 12.10 × 9.20 meters and is 1.2m thick. The overall foundation plan is reported in Figure 5.

Following the design procedure described in the previous sections of the paper, after have being defined the geotechnical model valid in seismic conditions under liquefaction, the pile ultimate axial capacity have been defined.

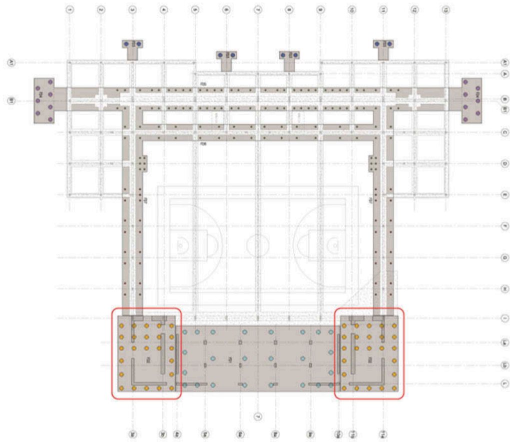

Figure 5. Foundation plan of the sport hall. In red the two seismic cores are highlighted.

Pile base capacity in static conditions has been evaluated adopting the formulation proposed by Berezantsev (1965), because pile tip in placed in the lower sandy layer, finding out a value of $Q_{base,S}$ = 1190 kN. Considering the development of excess pore pressures also in the lower sandy layer, it does not reach liquefaction but the tip resistance decrease. According to Equation 3 a ratio of 0.86 between the seismic and static value of base resistance has been evaluated, corresponding to a value of 0.238 for the factor $r_{u,base}$ (Eq. 4). This means that in seismic conditions the pile base resistance decrease to:

$$Q_{base,E} = 0.86 \cdot 1190 \approx 1000 \text{ kN} \tag{11}$$

Pile shaft capacity has been calculated via a direct correlation with data from CPTu adopting the method of Eslami & Fellenius (1997) improved by Niazi (2013). We referred to the 6 CPTu performed on site, finding out values of $Q_{shaft,S}$ ranging from 1390 kN to 1778 kN, with a mean value of 1600 kN.

As described in § 3.2, in our proposed design procedure the shaft capacity is completely neglected in the liquefiable layer. Adopting the same calculation procedure of the static case, in seismic conditions we found out values of $Q_{shaft,E}$ ranging from 1070 kN to 1448 kN, with a mean value of 1263 kN.

The BEM analyses of the piled foundation of the core (Figure 6) give axial forces on piles as reported in Table 2. According to the design criteria defined by the Italian code NTC pile axial capacity check is satisfied in static condition with a maximum exploitation of 81% and in seismic condition at 92%.

The last step of the design involve the check of piles against instability. It can be considered that FDP piles are rigidly connected at the top to the pile cap. The same fixity can be considered at the base, because the liquefiable sandy layer is near the ground and the piles

Table 2. Maximum axial forces on piles from BEM analyses.

Action	u.m.	Static	Seismic
Maximum compressive force	[kN]	1047	964
Maximum tensile force	[kN]	-	15
Maximum horizontal shear force	[kN]	81	79
Maximum vertical settlement	[mm]	6.9	5.9
Maximum horizontal displacement	[mm]	0.5	11.2

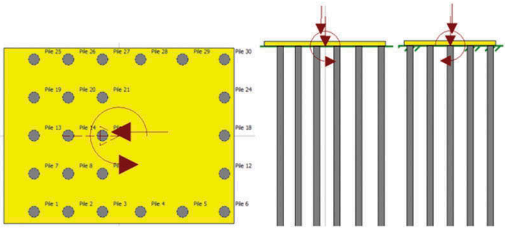

Figure 6. Repute calculation model of the piled foundation of the core.

continue below in the non-liquefiable clayey layers. Nevertheless, as a precaution, we assumed the equivalent length (β·h) as twice the thickness of the liquefiable layer (equal to 5.0m, see Table 1). Considering the elastic properties of the cross-section of the piles (E·I = 159.04 MNm^2), the interstitial pressure ratio ($r_{u,base}$ = 0.238) and limiting the critical load by the factor ψ = 0.35 (see Eq. 10) the ultimate axial load can be evaluated as $P_{ult} \approx 96900$ kN, more than 100 times the maximum axial force acting on piles under seismic loading (see Table 2). The slenderness ratio $\lambda = 67$ is lower than the assumed upper admissible limit of 75.

The piled foundations, designed according to the proposed procedure, satisfy all the safety criteria.

7 CONCLUSIONS

In the paper a design procedure to correctly deal with the design of pile groups on liquefiable soil has been presented. The procedure provides guidance as concern the geotechnical model (modified strength and stiffness parameters under liquefaction), the pile axial capacity (shaft and base components) and the pile instability. Each step of the procedure is based on specific theories from various authors, in order to base the design on solid scientific bases.

Cone penetration tests are fundamental in many steps of the procedure: just think to the geotechnical characterization and the pile capacity evaluation.

The procedure is intended as a guide for designers who have to face the design of a deep foundation on liquefiable soil. This is a very sensitive problem, so we hope the guide can help geotechnical engineers in this challenging work.

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