Effect of non-linear soil deformation on the interaction among energy piles

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Abstract: This study investigates the effect of non-linear soil deformation on the displacement interaction among energy piles. The work is based on interaction factor analyses of full-scale pile group tests, whose results are compared with experimental evidence. The results presented highlight the tendency of interaction factor analyses that ignore non-linear soil deformation to overestimate the interaction and the displacement of energy pile groups. This outcome, in accordance with previous studies for conventional pile groups subjected to mechanical loads, may be considered in the analysis and design of energy pile groups subjected to thermal (and mechanical) loads through the interaction factor method.

Keywords: Energy pile groups, non-linear deformation, interaction factor method, continuous model, vertical displacement, thermo-mechanical behaviour

1. Introduction

Over the last thirty years, a number of experimental [1] and theoretical [2-9] investigations based on interaction factor analysis have been performed to address the effect of non-linear soil deformation on the vertical displacement of and interaction between conventional piles subjected to mechanical loads. There are

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multiple reasons for these investigations. Soil deformation can often be non-linear and may involve reversible (i.e., elastic) or irreversible (i.e., plastic) stress-strain behaviour. As a consequence of this non-linearity, different deformation modulus values can be associated with the varying shear strain levels that characterise the soil surrounding piles (e.g., as a result of loading or construction effects). Consideration of these deformation modulus values has been proven to be essential to analyse and design pile groups [2, 5, 6].

The soil region adjacent to or in the vicinity of the shaft of single piles subjected to mechanical loads undergoes high shear strains on the order of 0.005 to 0.015% [6]. These strain levels are associated with lower soil modulus values and generally correspond to either reversible or irreversible soil behaviour that remains localised close to the pile [1, 3, 4].

The bulk of the soil between piles experiences low shear strains that may be orders of magnitude smaller than those near the piles and typically less than approximately 0.001% [6]. These strain levels are associated with higher soil modulus values and generally correspond to a soil behaviour that may be considered to be reversible, as are the interactions between the piles [1, 3].

The aforementioned considerations justify a specific approach for the interaction factor analysis of pile groups [2, 4-7]. Soil modulus values that are associated with high strain levels can be used to characterise the vertical displacement behaviour of the single piles composing any group. On the other hand, soil modulus values that are associated with low strain levels can be used to estimate the interaction between the piles. With this approach, interaction factor analysis based on appropriate soil

modulus values can be used to capture the vertical displacement of conventional pile groups subjected to mechanical loads almost irrespectively of the mathematical formulation employed for describing the soil behaviour and the related pile response [7, 9]. The above is justified by the close adherence to reality of results of interaction factor analysis of conventional pile groups that exploit either different [5, 8] or varying [3, 6, 9-11] soil modulus values, and linear elastic [5, 8], non-linear elastic [6, 9] or non-linear elasto-plastic [3, 10, 11] stress-strain relationships.

Thermal loads applied to energy piles induce a pattern of the deformation field that differs from that induced by mechanical loads. However, these loads are associated with a comparable evolution of the strain levels in the soil that can be related to different deformation modulus values. The results of full-scale *in-situ* tests of single [12-17] and groups [18-21] of energy piles further highlight that the effects caused by thermal loads on the piles and the soil, although often smaller for geothermal exploitations under serviceability conditions, can be comparable and even greater than those induced by mechanical loads. This fact involves different magnitudes and combinations of mechanical and thermal loads applied to energy piles causing either a reversible or irreversible soil and pile response, similarly to what can occur when (only) mechanical loads are applied to conventional piles. The above appears to be relevant especially for friction energy piles [22]. The reason behind this is that, for the same applied load, friction piles mobilise a greater proportion of shear strain and shear stress at the pile-soil interface compared to end-bearing piles.

Currently, interaction factor analysis methods aimed at estimating the vertical

displacement of and interaction between energy piles subjected to thermal loads ignore non-linear soil deformation. This is caused by the fact that these methods consider a unique soil modulus value to characterise both the single pile response and the interactions among the piles [23-25]. In particular, prior to this study, no simplified yet rational methods have been formulated and applied to expediently address the effect of non-linear soil deformation on the interaction among energy piles subjected to thermal loads.

To address this challenge, this paper (i) modifies and extends the procedure for determining the interaction factors proposed by Poulos [5] for conventional pile groups subjected to (only) mechanical loads in non-linearly deforming soils to energy pile groups subjected to thermal (and mechanical) loads, and (ii) investigates the effect of non-linear soil deformation on the vertical displacement of and interaction between energy piles.

The use of linear elastic theory, constant material properties and linear mathematical formulations are the fundamental assumptions underlying the procedure presented here to describe non-linear soil deformation and the related vertical displacement of piles and interaction between them. Although this procedure is approximate in principle, it has in practice been capable of accurately and expediently reproducing the vertical displacement behaviour of conventional pile groups. In this paper, it demonstrates the same capability for energy pile groups.

The effects of the potential irreversible behaviour of soil regions adjacent to or in the vicinity of piles are not analysed in this study because reference is made to serviceability conditions in which reversible conditions prevail. However, these

effects are considered to be comparable to those that will be analysed in this paper because they may be tackled similarly (although more approximately) during loading by considering different soil modulus values as performed in this study.

In the following, the considered procedure is first applied and validated against the experimental evidence reported by O'Neill et al. [2] involving full-scale *in-situ* tests of conventional pile groups subjected to mechanical loads in overconsolidated clay where non-linear soil deformation was observed. To address energy piles, these pile group tests are then modelled by considering the piles to be subjected to thermal loads. Finally, the effect of non-linear soil deformation on the vertical displacement of energy piles subjected to thermal loads and interaction between them is analysed and discussed, and concluding remarks are summarised.

2. Interaction factor analysis of energy pile groups

2.1 The problem, hypotheses and considerations

The problem addressed by interaction factor analysis of pile groups is the knowledge of the displacement interaction factor relationship between a receiver pile, j , due to the loading of an adjacent source pile in a pair, i , with reference to the displacement of the source pile subjected to the same loading in an isolated case.

In this problem, the piles are cylindrical solids characterised by a length, L , and a diameter, D , free to move vertically at their heads (i.e., no head restraint) and characterised by an infinitely flexible slab. Consideration of piles characterised by no-head restraint represents a worst-case scenario for their vertical displacement

estimation and involves performing conservative serviceability analyses. Although this procedure is approximate in principle, it is often used in practice because it is capable of accurately and expediently reproducing group action phenomena. The reason behind such a capability is that this procedure neglects the role of the slab (or of any other element connected above the piles and in contact with the soil) when modelling the interaction between the piles and the soil. When considered in a pair, the piles are located at a centre-to-centre distance (i.e., spacing), s , apart from each other.

In the case of thermal loading [23, 24], the source pile is assumed to be subjected to a (positive or negative) temperature change, ΔT . The receiver pile (when present) does not undergo any temperature change caused by potential thermal interactions with the source pile. In the case of mechanical loading [26, 27], the source pile is assumed to be subjected to a vertical mechanical load, P , at its head.

In interaction factors analysis, the effects induced by thermal and mechanical loads are considered separately. However, they can be superimposed at any time based on the widely used assumption of a linear elastic behaviour of the materials constituting the piles and the soil.

2.2 Classical interaction factor analysis approach

Currently, both design charts [23, 25, 28, 29] and behavioural models [3, 24, 26, 27] are available to address the displacement interaction factor relationship between any two conventional or energy piles. This relationship allows the interaction factor analysis of any pile group to be performed.

In this paper, the approach to determine the displacement interaction factor between a pair of energy piles proposed by Rotta Loria et al. [24] is considered and the influence of both thermal and mechanical loads is addressed.

The sign convention adopted in this paper is such that compressive stresses, contractive strains and downward displacements (i.e., settlements) are considered to be positive.

The interaction factor analysis of any pile group subjected to thermal and/or mechanical loads consists of five key steps:

1. The analysis of a single isolated source pile subjected to thermal or mechanical loading to define the vertical displacement, $w_i(z)$, and the shear stress, $\tau_i(z)$, along the pile shaft (z is the vertical coordinate). This analysis is developed in the present paper using finite element analysis.
2. The determination of the vertical displacement of the soil, $w(r,z)$, at any given radial distance, r , from the axis of the previously analysed single isolated pile subjected to loading, and along the vertical coordinate, z . This step can be performed by using the continuous behavioural model presented by Rotta Loria et al. [24] for energy piles subjected to thermal loads based on the work of Chow [3] for conventional piles subjected to mechanical loads. According to this model, the vertical displacement of the soil caused by a distribution of point loads associated with the effects of the thermal or mechanical load applied to the source pile may be calculated through the equations of Mindlin [30] and the elastic principle of superposition of effects

as

$$w(r, z) = \sum_{l=1}^m w_{kl} = \sum_{l=1}^m \frac{P_l}{16\pi G_{soil}(1-\nu_{soil})} \left(\frac{3-4\nu_{soil}}{R_1} + \frac{8(1-\nu_{soil})^2 - (3-4\nu_{soil})}{R_2} + \frac{(z_l - z_k)^2}{R_1^3} + \frac{(3-4\nu_{soil})(z_l + z_k)^2 - 2z_l z_k}{R_2^3} + \frac{6z_l z_k (z_l + z_k)^2}{R_2^5} \right) \quad (1)$$

where w_{kl} is the vertical displacement of any soil node, k , caused by a point load, P_l , applied in the node, l ; m are the nodes of the source pile from which the effects of the point loads are calculated; G_{soil} and ν_{soil} are the shear modulus and Poisson's ratio of the soil, respectively; $R_1 = \sqrt{r_k^2 + (z_k - z_l)^2}$ (for which r_k is the horizontal distance between node l at which the load is applied and node k at which the influence is considered, z_k is the depth of node k , and z_l is the depth of node l); and $R_2 = \sqrt{r_k^2 + (z_k + z_l)^2}$.

The point loads (i.e., finite forces applied at a point) and the nodes (i.e., elements described by surfaces of zero area) used to characterise the interaction between a loaded pile and the surrounding soil are indeed a simplification of reality. However, although being mathematical artefacts, they allow a thorough description of many practical problems related to piles among other foundations [31].

3. The correction of the vertical displacement of the soil, $w(r, z)$, to address the actual vertical displacement of a receiver pile, $w_j(s, z)$, located at a centre-to-centre spacing, s , from the source pile that was previously considered to be

isolated, and generally characterised by a greater stiffness than that of the soil.

This step can be performed by solving the second-order differential equation governing the equilibrium of each node of the receiver pile [27]

$$\frac{\partial^2 w_j}{\partial z^2} - \lambda_{lt}^2 (w(r, z) - w_j(s, z)) = 0 \quad (2)$$

The parameter λ_{lt} is a load-transfer coefficient given by

$$\lambda_{lt} = \sqrt{\frac{\pi D K_s}{A_{EP} E_{EP}}} \quad (3)$$

where A_{EP} is the cross-sectional area of the pile, E_{EP} is the Young's modulus of the pile and K_s can be determined for example as [32]

$$K_s = \frac{1.3 G_{soil}}{\pi D} \left(\frac{E_{EP}}{E_{soil}} \right)^{-\frac{1}{40}} \left(1 + 7 \left(\frac{L}{D} \right)^{-0.6} \right) \quad (4)$$

where E_{soil} is the Young's modulus of the soil.

Equation (2) needs two boundary conditions to be solved. The first boundary condition is chosen with reference to the state of restraint that characterises the head of the receiver pile. Because the pile is free to displace vertically at its head no vertical stress will be present in this setting. This condition can be mathematically expressed through the constitutive equations as [27]

$$\left. \frac{\partial w_j}{\partial z} \right|_{z=0} = 0 \quad (5)$$

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213 The second boundary condition needs to be preferably chosen by considering
 214 whether the source pile is subjected to thermal or mechanical loading. The
 215 application of this boundary condition must also refer to the shear stress
 216 induced by either thermal loading or mechanical loading separately.

217 In the case of thermal loading, the second boundary condition is chosen with
 218 reference to the so-called “null point” of the shear stress of the receiver pile.

219 The thermally induced deformation of the source pile includes the formation
 220 along its length of a setting where zero thermally induced displacement occurs
 221 (i.e., the null point of the vertical displacement) and a setting where zero
 222 thermally induced shear stress occurs (i.e., the null point of the shear stress)
 223 [24]. The continuity of the displacement fields of the piles and soil involves a
 224 transmission of the null points of the vertical displacement and of the shear
 225 stress from the source pile to the receiver pile through the soil. It is worth
 226 noting that the locations of the null points are generally not the same for the
 227 source and receiver piles. Because the shear stress is equal to zero along the
 228 soil profile adjacent to the receiver pile, the displacement of this pile is the
 229 same as that of the soil (i.e., $w(r, z) - w_j(s, z) = 0$). This fact can be
 230 mathematically expressed to become the second boundary condition for the
 231 case of thermal loading as [24]

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$$w_j(s, z = z|_{\tau=0}) = w(r, z = z|_{\tau=0}) \quad (6)$$

Such a boundary condition can be easily determined with reference to the distribution of the shear stress in the soil surrounding any source pile obtained from the equations of Mindlin [30], as shown by Rotta Loria et al. [24].

In the case of mechanical loading, the second boundary condition is chosen with reference to the base of the receiver pile. Because the pile base acts as a rigid punch on the lower layer of soil [26], there is a specific relation between the point load applied at the pile base, $P_b = A_{EP}\sigma_z(z = L)$, (where $\sigma_z(z = L)$ is the normal stress acting on A_{EP} at the pile base) and the displacement of the rigid circular disc, $w_j(z = L)$ [33]. This relation can be mathematically expressed through the constitutive equations [27] to become the second boundary condition for the case of mechanical loading as

$$E_{EP}A_{EP}\left.\frac{\partial w_j}{\partial z}\right|_{z=L} + K_b w_j(z = L) = 0 \quad (7)$$

where [34]

$$K_b = \frac{2DG_{soil}}{1-\nu_{soil}} \quad (8)$$

4. The determination of the corrected pile-soil-pile interaction factor defined as [23]

$$\Omega(s, z) = \frac{\text{additional displacement due to adjacent pile}}{\text{displacement of single isolated pile}} = \frac{w_j(s, z)}{w_i(z)} \quad (9)$$

5. The analysis of the vertical displacement $w_j(z)$ of any pile, j , composing a general group with a total number of piles, n_{EP} , in which some or all of the piles may be subjected to loading through the interaction factor method. In the case of piles subjected to thermal loads, this analysis can be performed as [23]

$$w_j(z) = w^{1,th}(z) \sum_{i=1}^{i=n_{EP}} \Delta T_i \Omega_{ij}^{th}(s, z) \quad (10)$$

where $w^{1,th}(z)$ is the vertical displacement along the length of a single isolated pile per unit temperature change, ΔT_i is the applied temperature change to pile i , and Ω_{ij}^{th} is the interaction factor for two piles subjected to thermal loading in a pair corresponding to the centre-to-centre distance between pile i and pile j .

In the case of piles subjected to mechanical loads, this analysis can be performed as [28]

$$w_j(z) = w^{1,m}(z) \sum_{i=1}^{i=n_{EP}} P_i \Omega_{ij}^m(s, z) \quad (11)$$

where $w^{1,m}(z)$ is the vertical displacement along the length of a single isolated pile per unit mechanical load, P_i is the mechanical load applied to the

head of pile i , and Ω_{ij}^m is the interaction factor for two piles subjected to mechanical loading in a pair corresponding to the centre-to-centre distance between pile i and pile j .

It is worth noting that the described interaction analysis can consider pile groups in both uniform and non-uniform (e.g., stratified) soil deposits. The response of any single isolated pile in non-uniform soil deposits can be considered with accuracy by developing the finite element analysis highlighted in step 1. The definition of the vertical displacement of the soil surrounding the source pile and that of the receiver pile addressed in steps 2 and 3, respectively, can approximately be performed by considering a mean value of the shear modulus of the soil layer where the displacement is calculated at any soil node, k , and the shear modulus of the soil layer where the point load is applied at any pile node, l , through step 2. The resulting definition of the interaction factor addressed in step 4 and the subsequent interaction factor analysis targeted in steps 5 and 6 can finally refer to situations involving pile groups in both uniform and non-uniform soil deposits.

2.3 Modified interaction factor analysis approach

Currently, both $w_i(z)$ (cf., step 1 of the analysis approach presented in Subsection 2.2) and $w_j(s, z)$ (cf., step 3) are computed using the same value of soil Young's modulus adjacent to the pile, i.e., the “near-pile” soil modulus E_{soil} [23-25]. The resulting interaction factor analysis is linear elastic and does not account for non-linear soil deformation.

In the following, a modified procedure for addressing non-linear soil deformation is proposed for energy piles. This procedure has been extended and modified from that of Poulos [5] based on the different definitions of the interaction factor used in this paper and in the referenced one, and the related mathematical needs to rigorously describe the deformation problem of pairs of piles.

In this procedure, the vertical displacement of the single isolated source pile, $w_i(z)$, is calculated (step 1) using the near-pile soil modulus, E_{soil} , whereas the vertical displacement of the receiver pile in the pair, $w_j(s, z)$, is calculated (steps 2 and 3) using an average soil modulus, $E_{soil,ave}$, which accounts for the smaller strain levels between the piles. The resulting modified interaction factor analysis is linear elastic and accounts for non-linear soil deformation.

The average soil modulus is given by the following expressions (cf., Figure 1)

$$E_{soil,ave}(s) = E_{soil} \left(1 + \frac{(\mu-1)(s-D)}{2s_t} \right) \quad \text{for } s < s_t + D \quad (12a)$$

$$E_{soil,ave}(s) = E_{soil} \left(\mu + \frac{(1-\mu)s_t}{2(s-D)} \right) \quad \text{for } s \geq s_t + D \quad (12b)$$

where $\mu = E_{soil,m}/E_{soil}$ (for which $E_{soil,m}$ is the soil “mass” modulus between the piles) is likely to lie within the range of 3 and 10 and s_t is a transition distance likely to be between $3D$ and $6D$ [5].

The described distribution of soil modulus to determine $E_{soil,ave}$ for calculating $w_j(s, z)$ is indeed simplified for a number of reasons. First, because the non-linear

soil deformation is considered through a linear distribution of soil modulus between the piles. Next, because the ratio of the soil mass to the near-pile soil modulus needed to define this linear distribution may be somewhat arbitrarily defined [9]. Then, because the problem of the non-linear soil deformation is solved by considering a constant and unique value of average soil modulus between the piles, and linear elastic soil behaviour. Finally, because the interaction factor calculated with reference to two different distributions of soil modulus characterising the single pile and the pair of piles will generally differ from the interaction factor calculated with reference to a unique distribution of soil modulus that merges the previous ones and characterises both the single pile and pair of piles. In principle, extensive experimental evidence may be of help for formulating a procedure more adherent to reality. In practice, (i) the lack of such evidence, (ii) the resulting increase in complexity of the procedure that would not commensurably improve the results of the analysis and (iii) the success of the analogous procedure proposed by Poulos [5] corroborate the use of the considered approach.

3. Modelling of pile group tests

3.1 Experimental tests reported by O'Neill et al. [2]

3.1.1 *Site and pile group characterisation*

The results of a comprehensive programme of full-scale *in-situ* tests performed on eleven conventional piles subjected to vertical mechanical loading were reported by O'Neill et al. [2]. The piles were socketed in an overconsolidated clayey soil deposit

and consisted in closed-end tubular steel pipes of an external diameter of $D = 273$ mm, a wall thickness of $t_w = 9.27$ mm, and an embedded length of $L = 13.11$ m (cf., Figure 2). Two piles were driven as isolated reference piles. Nine other piles were driven in a square 3×3 arrangement. The two reference piles were located at a spacing of $s = 10.4D$ from the centre of the pile group. The piles in the group were located at spacing of $s = 3D$ apart from each other (cf., Figure 2). A rigid slab made of reinforced concrete connected the piles in the group with the possibility to be detached from any pile to form subgroups. A geotechnical characterisation of the site was performed through standard penetration tests, cone penetration tests, pressuremeter tests, unconsolidated undrained triaxial tests, laboratory consolidation tests and crosshole seismic tests. Further information is given by O'Neill et al. [2].

3.1.2 *Pile group tests*

Different tests were performed in the aforementioned campaign [2]. Those included (cf., Figure 3) the vertical mechanical loading of the two reference piles (reference pile tests 1), of the 9-pile group (9-pile group test 1), of a 5-pile subgroup (5-pile subgroup test) and of a 4-pile subgroup (4-pile subgroup test). Measurements of the vertical displacement of the piles and of the soil as well as of the load redistribution in the pile groups were made and are detailed by O'Neill et al. [2].

3.2 Models

3.2.1 *Modelling choices*

Two stationary thermo-mechanical finite element analyses characterised by the

mathematical formulation reported by Rotta Loria and Laloui [23] are performed using the software COMSOL Multiphysics [35]. The first analysis simulates the response of the single isolated piles tested by O'Neill et al. [2] under mechanical loading. Because the considered numerical analysis was performed after the occurrence of the modelled event and with the associated results available, it is classified as a Class C1 prediction [36]. The second analysis simulates the response of the same single isolated piles under thermal loading.

Two series of analytical analyses carried out using the classical and modified interaction factor approaches are also performed. The first series of analyses (*i*) applies the classical and modified interaction factor approaches for simulating the real case study tested by O'Neill et al. [2] involving pile groups subjected to mechanical loads where non-linear soil deformation was observed, (*ii*) validates the proposed analysis approach by comparing the experimental and modelling results, and (*iii*) addresses the effects of non-linear soil deformation on the vertical displacement of pile groups subjected to mechanical loads following the analyses of Poulos [5]. The second series of analyses (*i*) applies the classical and modified interaction factor approaches for investigating pile groups subjected to thermal loads and (*ii*) expands on the effect of non-linear soil deformation on the vertical displacement of and interaction among energy pile groups subjected to thermal loads.

In all of the analyses: (i) the piles are assumed to be isotropic, homogeneous and uniform cylindrical solids that are considered to be representatively described by linear thermo-elastic behaviour; (ii) the soil layer is assumed to be an isotropic, homogeneous and uniform mass that can be considered to be semi-infinite and

representatively described by linear elastic behaviour (i.e., the soil is an infinite heat reservoir that remains at a fixed constant temperature); and (iii) the same loads that are applied to the piles in the single isolated case are also considered to be applied uniformly to the piles in the groups.

3.2.2 *Finite element model*

An axisymmetric finite element model characterised by a depth of $h = 25L$ and a width of $x = 500D$ is made to simulate the response of the single isolated piles (cf., Figure 4). In this model, extremely fine tetrahedral meshes are used to describe the pile and soil domains.

3.2.3 *Boundary and initial conditions*

In the analyses, restrictions are applied to both the vertical and horizontal displacements on the bottom horizontal boundary that may be considered to characterise the deep soil domain surrounding the piles (i.e., pinned boundary), and to the horizontal displacements on the vertical boundary (i.e., roller boundary). No restrictions are applied to the displacements on the top boundary of the pile and soil surfaces.

No residual stresses from the installation of the piles are considered in these elements and in the adjacent soil region. This hypothesis may not be completely representative of reality but can be applied successfully in methods of pile deformation analysis by choosing appropriate soil modulus values [37].

When addressing the response of the piles subjected to mechanical loads, the

experimentally observed vertical mechanical load of $P_{ult} = 678$ kN for which the single isolated pile failure was observed [2] is considered to be applied at the pile heads. However, the results presented in the following generally refer to the reference mechanical load $P_{ref} = 356$ kN.

When addressing the response of the piles subjected to thermal loads, the temperature change of $\Delta T = 10$ °C is considered to be applied to these elements. The soil domain is treated as an infinite heat reservoir. The initial temperature of the piles is assumed to be $T_0 = 15$ °C.

Figure 4 summarises the boundary conditions considered in the analyses, with reference to the axisymmetric finite element model.

3.2.4 *Material parameters*

Table 1 presents the material parameters that were reported by O'Neill et al. [2] as characteristic of the experiment performed. Table 2 reports the material parameters used in this study. Differently to the experimental observation, the piles are characterised by a uniform and homogeneous cross-section made of steel, and the soil is characterised by constant values with depth of the near-pile modulus (steps 1-3 in the classical interaction factor approach and step 1 in the modified interaction factor approach) and mass modulus (steps 2-3 in the modified interaction factor approach). The near-pile soil modulus was determined according to the pressuremeter test data and the considerations of O'Neill et al. [2]. The soil mass modulus was determined according to the crosshole seismic test data and the considerations of Poulos [5]. Figure 5 presents the evolution of the normalised soil

Young's modulus with the normalised pile depth presented by O'Neill et al. [2] and assumed for this study. A transition distance of $s_t = 3D$, as suggested by Poulos [5], is considered in the modified interaction factor analysis approach.

4. Results

4.1 Validation of the numerical model against experimental results

Figure 6 (a) presents a comparison between the numerically modelled and experimentally observed load-displacement curves for the single isolated pile. The vertical head displacement of the pile is normalised by the pile diameter. The applied vertical load is normalised by the value of mechanical load corresponding to failure. The numerical results are in close agreement with the experimental observations, although the former fail in capturing the latter when plastic strains occur in the soil surrounding the pile for a normalised vertical load approximately greater than $P/P_{ult} = 0.6$. This result is in accordance with the linear elastic soil behaviour considered in the numerical analysis and the associated incapability of the analysis of capturing plastic strains. The agreement between the results for a normalised vertical load of approximately up to $P/P_{ult} = 0.6$, i.e., within the elastic branch of the load-settlement curve, justifies the hypothesis of using a constant value of soil Young's modulus for characterising the elastic response of the single isolated pile in the numerical model.

Figure 6 (b) presents a comparison between the numerically modelled and experimentally observed evolution of the normalised vertical load in the pile with the

normalised pile depth. The normalised reference mechanical load applied at the pile head of $P_{ref}/P_{ult} = 0.52$ is considered. The numerical results are highly comparable to the experimental data and together with the results presented in Figure 6 (a) prove the capability of the axisymmetric analysis of capturing the response of the single isolated pile under the serviceability conditions that are of interest for this study.

Figure 7 presents a comparison between the modelled and experimentally observed evolutions of the normalised vertical displacement with the normalised radial distance from the pile axis. Reference is made to a normalised depth of $z/L = 0.047$. The results of two different analyses are presented:

- An analysis that addresses linear soil deformation based on steps 1 and 2 of the classical interaction factor approach. In this analysis, the soil displacement, $w(r, z)$, is calculated analytically (step 2) based on the vertical displacement, $w_i(z)$, and shear stress, $\tau_i(z)$, distributions characterising the single isolated source pile using the same value of near-pile soil modulus, E_{soil} , that is used in the numerical analysis to define $w_i(z)$ and $\tau_i(z)$ (step 1).
- An analysis that addresses non-linear soil deformation based on steps 1 and 2 of the modified interaction factor approach. In this analysis, the soil displacement, $w(r, z)$, is calculated analytically (step 2) based on the vertical displacement, $w_i(z)$, and shear stress, $\tau_i(z)$, distributions characterising the single isolated source pile using a different value of soil

mass modulus, $E_{soil,m}$, compared to the near-pile soil modulus, E_{soil} , that is used in the numerical analysis to define $w_i(z)$ and $\tau_i(z)$ (step 1).

The close comparison between the modelling results addressing the non-linear soil deformation and the experimental results proves the occurrence of the considered phenomenon. This phenomenon was originally remarked by O'Neill et al. [2]. The agreement between the obtained results validates also the choice of the transition distance of $s_t = 3D$ suggested by Poulos [5] for the considered site.

The above validates the numerical model against the results of the full-scale *in-situ* reference pile tests 1.

4.2 Pile group vertical displacement induced by mechanical loading

Figure 8 presents a comparison between the modelled and experimentally observed evolutions of the normalised average vertical head displacement of the piles, w_{ave} , with the number of directly (mechanically) loaded piles, n_{LP} , for the reference pile tests 1, the 9-pile group test 1, the 5-pile subgroup test and the 4-pile subgroup test. The normalised reference mechanical load applied at the pile heads of $P_{ref}/P_{ult} = 0.52$ is considered. The results of two different modelling analyses addressing linear soil deformation based on the classical interaction factor approach and non-linear soil deformation based on the modified interaction factor approach are presented. The results of the modified interaction factor analysis approach considering non-linear soil deformation are in close agreement with the experimental observations for the pile groups considered, differently from the results obtained

through the classical interaction factor analysis approach. This evidence underscores the tendency of the classical interaction factor analysis approach to overestimate group effects and displacement interactions among the piles, and thus, the average vertical displacement of pile groups subjected to mechanical loads. This result also highlights the capability of the modified interaction factor analysis approach of capturing the vertical displacement of pile groups in non-linearly deforming soils.

Figure 9 presents the values of the normalised average soil Young's modulus corresponding to the centre-to-centre spacing between different pairs of piles that are used in the modified interaction factor analysis approach to define the interaction factor and the vertical displacement for different numbers and locations of piles subjected to loading. The average Young's modulus of soil is normalised by the near-pile Young's modulus. Although for any given distance between two piles a constant soil mass modulus value is used, the normalised average soil Young's modulus varies non-linearly for different pile spacing according to equations (12a) and (12b). This fact highlights the capability of the modified interaction factor analysis approach to consider non-linear soil deformation (through a linear elastic behaviour of the soil surrounding the piles) depending on the geometrical configuration of the piles in any group, in addition to the material properties characterising the group.

4.3 Pile group vertical displacement induced by thermal loading

Figure 10 (a) presents a comparison between the modelled evolutions of the normalised average vertical head displacement of the piles with the number of

directly (thermally) loaded piles for the reference pile tests 1, the 9-pile group test 1, the 5-pile subgroup test and the 4-pile subgroup test. The temperature change applied to the piles of $\Delta T = 10\text{ }^{\circ}\text{C}$ is considered. The results of two different modelling analyses addressing linear soil deformation based on the classical interaction factor approach and non-linear soil deformation based on the modified interaction factor approach are presented. In both of these analyses, the corrected vertical displacement of the piles in the groups is determined by referring to the boundary condition expressed in equation (6) for solving equation (2), i.e., the boundary condition referred to the position of the null point of the shear stress. As previously observed for the case in which the piles are mechanically loaded, the classical interaction factor analysis approach determines a greater average vertical head displacement of the pile groups compared to the modified interaction factor analysis approach. The overestimation of the group displacement associated with the application of the classical interaction factor analysis approach compared to the estimate achieved through the application of the modified interaction factor analysis approach increases with the number of thermally loaded piles in the group.

Figure 10 (b) presents the results of equivalent analyses compared to those used for obtaining the results summarised in Figure 10 (a), except for the use of the boundary condition expressed in equation (7) for solving equation (2) and determining the corrected vertical displacement of the piles in the groups, i.e., the boundary condition referring to the pile base, instead of the boundary condition expressed in equation (6). The considered modelling approach effectively coincides with that proposed by Randolph and Wroth [26] and Mylonakis and Gazetas [27] for

the vertical deformation of piles subjected to mechanical loads. This modelling approach does not account for the key distinguishing feature characterising the vertical deformation of piles subjected to thermal loads compared to that of piles subjected to mechanical loads, i.e., the presence of the null point. Therefore, this modelling approach yields a less accurate though valid analysis of the vertical deformation of piles subjected to thermal loads. This result confirms the preferable use of the boundary condition expressed in equation (6) for solving the equilibrium equation (2) that was proposed by Rotta Loria et al. [24] for piles subjected to thermal loads. The reason for this is that the approach based on the use of equation (6) estimates a group displacement that is considered to be more adherent to reality and lower than the vertical displacement that may be estimated through the approach based on the use of equation (7).

4.4 Interaction factor for energy piles in non-linearly deforming soil

Figure 11 shows the fundamental reason why the modified interaction factor analysis approach addressing non-linear soil deformation estimates a smaller group displacement compared to the classical interaction factor analysis approach addressing linear soil deformation. The evolution of the interaction factor with the centre-to-centre spacing between a pair of piles characterised by the features considered thus far is shown for interaction factor analyses considering and neglecting non-linear soil deformation, respectively. It is evident that considering non-linear soil deformation through the modified interaction factor analysis approach involves a smaller estimate of the interaction factor among the piles compared to that

obtained when neglecting such a phenomenon through the classical interaction factor analysis approach. The relative reduction in the interaction factor described by the modified analysis approach compared to that estimated by the classical approach decreases with the increasing pile spacing. This result is caused by the smaller impact of non-linear soil deformation on pile-soil-pile interaction with increasing distance between the piles as a consequence of the localisation of non-linear soil deformation in the close vicinity of the piles. In the interaction factor analysis approach, higher estimates of the interaction factor result in greater computed group displacements for the same displacement of the single isolated reference pile. Therefore, the classical interaction factor analysis approach is considered to be remarkably conservative for design situations in which soil deformation may be non-linear compared to the modified interaction factor analysis approach. The decrease in the interaction factor relationship characterising situations where non-linear soil deformation is observed may suggest negligible displacement interactions among piles. This may be the case for design situations characterised by very few short and stiff piles located in a group. However, displacement interactions among piles are generally considered to deserve attention in the analysis and design of pile groups, especially when many long and compressible piles may be subjected to thermal and/or mechanical loads.

5. Concluding remarks

This paper presents an analysis of the effect of non-linear soil deformation on the

displacement interaction among energy pile groups. The work is based on a modified interaction factor analysis approach that (i) has been validated against experimental evidence involving full-scale pile group tests available in the literature and (ii) may be expediently used to estimate the vertical displacement of general configurations of pile groups subjected to thermal and/or mechanical loads in situations where the soil deformation is non-linear.

The main conclusions that can be drawn from this study are as follows:

- The displacement interaction among piles subjected to thermal loads decreases in situations where soil deformation is non-linear compared to situations in which soil deformation is linear.
- This result is in accordance with evidence for pile groups subjected to mechanical loads that has been expanded in this paper.
- For given material properties and shapes of the piles composing any group, the extent of the decrease of the displacement interaction depends on (i) the ratio between the soil modulus characterising the bulk between the piles (i.e., the soil mass modulus) and the regions close to the piles (i.e., the near-pile soil modulus); (ii) the number of piles; and (iii) the centre-to-centre spacing between the piles.
- The classical interaction factor analysis approach tends to overestimate the vertical displacement of pile groups subjected to thermal and/or mechanical loads in situations where soil deformation is non-linear.
- Although the classical interaction factor analysis approach may be

622 remarkably conservative in situations where soil deformation is non-linear,
623 it is considered to be easier to apply than the modified interaction factor
624 analysis approach and thus still valuable for practical analysis and design
625 of pile groups subjected to thermal and/or mechanical loads.

- 626 • Although non-linear soil deformation involves lower displacement
627 interactions among piles subjected to loading compared to the case in
628 which soil deformation may be linear, interaction factor analyses are
629 considered of paramount importance for a comprehensive design of pile
630 groups. This consideration appears to be particularly valuable for groups
631 of numerous long and compressible piles.

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Table 1: Material parameters reported by O'Neill et al. [2] characterising the considered problem.

Experiment	
Soil parameters	
Near-pile Young's modulus, $E_{soil}(z = 0)$: [MPa]	17.2
Near-pile Young's modulus, $E_{soil}(z = L)$: [MPa]	79.2
Mass Young's modulus, $E_{soil,m}(z = 0)$: [MPa]	92.0
Mass Young's modulus, $E_{soil,m}(z = L)$: [MPa]	453.3
Poisson's ratio, ν_{soil} : [-]	0.5

726

Table 2: Material parameters used to model the considered problem.

Model			
Soil parameters		Pile parameters	
Near-pile Young's modulus, E_{soil} : [MPa]	49	Young's modulus, E_{EP} : [MPa]	210000
Mass Young's modulus, $E_{soil,m}$: [MPa]	272.6	Poisson's ratio, ν_{EP} : [-]	0.3
Poisson's ratio, ν_{soil} : [-]	0.5	Normalised transition distance, s_t/D : [-]	3
Linear thermal expansion coefficient, α_{soil} : [1/°C]	-	Linear thermal expansion coefficient, α_{EP} : [1/°C]	$1.2 \cdot 10^{-5}$

Figure 1: Assumed distribution of soil modulus between two piles. E_{soil} = near-pile soil modulus (characteristic of high strain levels); $E_{soil,m} = \mu E_{soil}$ (with $\mu \geq 1$) = soil mass modulus (characteristic of low strain levels); $E_{soil,av}$ = average soil modulus for computing interactions; s_t = transition distance.

Figure 2: The features of the considered site, based on the information reported by O'Neill et al. [2].

Reference pile tests 1

9-pile group test 1

5-pile subgroup test

4-pile subgroup test

● Directly loaded piles

Figure 3: Geometrical configuration of the piles tested by O'Neill et al. [2] for mechanical loading, further considered in this paper for thermal loading.

Figure 4: The finite element model of the single isolated pile.

Figure 5: Evolution of the normalised soil Young's modulus with the normalised pile depth presented by O'Neill et al. [2] and assumed for this study.

Figure 6: Comparison between the numerically modelled and experimentally observed (a) load-displacement curves for the single isolated pile and (b) evolution of the normalised vertical load with the normalised depth in the pile.

Figure 7: Comparison between the modelled and experimentally observed evolutions of the normalised vertical displacement with the normalised radial distance from the pile axis.

Figure 8: Comparison between the modelled and experimentally observed evolutions of the normalised average vertical head displacement of the piles with the number of directly mechanically loaded piles for the different pile tests considered.

Figure 9: Values of the normalised average soil Young's modulus corresponding to the centre-to-centre spacing between different pairs of piles that are used in the modified interaction factor analysis approach.

Figure 10: The modelled evolutions of the normalised average vertical head displacement of the piles with the number of directly thermally loaded piles for the different pile tests considered.

Figure 11: Evolution of the interaction factor with the centre-to-centre spacing between a pair of piles characterised by linear or non-linear soil deformation.