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Pisano, Federico; Tradigo, F; Mussi, A.

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Settlement prediction and monitoring of a piled raft foundation on coarse-grained soil: the case of the Allianz Tower in Milan.

Prévision des affaissements et comportement observé d'une fondation en radiers de pieux sur un matériau granulaire: la tour Allianz en Milan.

Fabio Tradigo & Angelo Mussi
Arup Italia, Italy, fabio.tradigo@arup.com

Francesco Petrella
Ove Arup & Partners Denmark, Denmark

Federico Pisanò
Geo-Engineering Section/Offshore Engineering Section, Delft University of Technology, The Netherlands

ABSTRACT: The process followed in the settlement prediction of the piled raft foundation of the Allianz tower in Milan is presented. In particular, the crucial role played by pile load tests is discussed. The data gathered from the bespoke monitoring network allowed to compare the said prediction with the real behaviour of the pile raft during and post construction and to draw some conclusions on soil structure interaction as well as the relevant soil stiffness and strain levels that govern the piled raft settlements.

RÉSUMÉ : On présente le processus suivi dans la prévision des affaissement de la fondation en radiers de pieux de la tour Allianz en Milan. En particulier, le rôle crucial joué par les tests de charge de pieux est discuté. Les données recueillies à partir du réseau de surveillance conçu sur mesure ont permis de comparer cette prédiction avec le comportement réel du radier durant et après la construction et de tirer quelques conclusions sur l'interaction de la structure avec le sol ainsi que la raideur du sol et les niveaux de déformation du sol qui régissent les affaissements du radier.

KEYWORDS: foundation, piled raft, finite element modelling, settlements

1 INTRODUCTION

In recent years, Milan (Italy) experienced many urban changes and its skyline is dramatically changing. In particular, within the 366,000 m² CityLife masterplan of the city's historic Fiera district, three high-rise buildings have been designed by architects Hadid, Libeskind and Isozaki. The construction of the latter, now called Allianz Tower, is completed so that at present, according to the CTBUH criterion of "highest occupied floor", it is the tallest building in Italy. Its superstructure is 202.2 meters in height from ground level and rests on a large piled raft (PR) foundation.

As described by Allievi et al. (2013), different foundation concepts were compared for optimization design purposes. Initial analyses led to conclude that, even though a very thick unpiled raft solution was found to be sufficient for ULS requirements, the piled foundation solution was more effective in terms of SLS performance, constructability and economical optimization. On the basis of PR design philosophy and in accordance with the Italian Construction Code, a so-called "hybrid" or "mixed" foundation type was conceived. The authors noticed that the overall cost of the optimized PR was 35-45% less than the unpiled solution.

1.1 Project description

The tower consists of a mixed steel-concrete structure with rectangular plan dimensions of approximately 54.8m and 22.5m, resulting in 52 above-ground stories and three levels of basement (PI1 to PI3). The superstructure is supported by a "large" rectangular reinforced concrete piled raft 63m wide and 27m long with a variable thickness ranging from 2.5m to 3.5m in correspondence of the cores and the central columns (Fig. 1). The raft is enhanced with 10 no. 1.5m and 52 no. 1.5m diameter piles. All the piles are bored cast in-situ and are 33.2m in length. Piles have been installed in correspondence of the

columns and the cores to reduce the high bending moments that would occur in the unpiled configuration. In turn, the presence of a large and rather flexible raft may induce rotations and translations to the pile heads as a result of non-negligible deflections under vertical service loads, resulting in shear and moment distributions along the piles. Consequently the structural connection between the piles and the raft is avoided (Figure 2) resulting in a disconnected piled raft (DPR) with zero-thickness gap between the pile heads and the raft (Tradigo et al. 2015a).

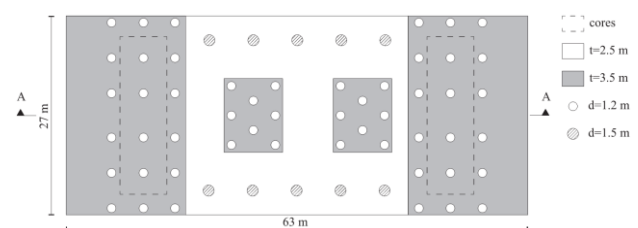


Figure 1. Allianz Tower piled raft foundation.

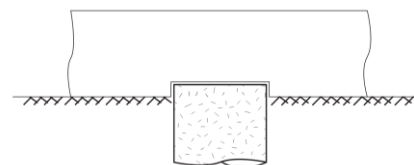


Figure 2. Pile head set-up.

1.1.1 Ground conditions

Geotechnical characterization confirmed the presence of a 100m thick deposit of Quaternary alluvial granular materials with the original ground level at +124m asl approximately. From a geotechnical point of view, soil layers consists of a sand deposits with thin cohesive lenses at a depth of approximately $z = 30$ m from the foundation basement (+108m asl). For the

purpose of the present study, which is mainly related to short term conditions, the presence of these lenses is not expected to significantly affect the overall response of the pile. Material parameters obtained from in situ tests and from the single pile back-analysis are summarized in Table 1.

Table 1. Soil model parameters.

Layer	Top of layer (m a.s.l.)	Material	Φ (°)	E (MPa)	E_{inc} (MPa/m)	ν
A	124	Made Ground	-	-	-	-
B	120	Gravel & Sand	36	228	1.32	0.2
C	89	Sand & Gravel	36	269	1.32	0.2
D	79	Clayey Silt	28	132	-	0.2
E	76	Sand & Gravel	36	286	1.32	0.2
F	59	Clayey Silt	28	150	-	0.2
G	55	Sand & Gravel	36	314	1.32	0.2
H	45.5	Clayey Silt	28	150	-	0.2
I	43.5	Sand & Gravel	36	390	-	0.2
J	36	Clayey Silt	28	600	-	0.2

1.1.2 Loading

After the completion of the foundation in April 2012, the tower construction began in November 2012 and ended in July 2014. A few months after, the claddings were fully installed. In the present publication, monthly measurements between November 2012 and May 2014 are presented: at that time the total dead load (structure and cladding) resting on the foundation was about $V = 950$ MN (including the raft self-weight). The live loads acting on the structure during the construction are not expected to play a significant role on the measured settlements and are therefore ignored in the calculations. Column load take down calculations resulted in concentrated equivalent vertical loads applied in the finite element (FE) model to the columns and to the cores.

1.1.3 Monitoring

The monitoring process, which continued throughout construction, started in November 2012 when the raft and floors PI3 and PI2 were already built for a total estimated applied load of approximately $V_0 = 188.5$ MN. During the construction of the foundation, load cells placed between the raft and the underlying soil measured an average increase of about $q_c = 2$ bar, corresponding to $\alpha_{R,exp} = 35\%$. In addition to the preliminary and contract pile load tests (see section 2.1.2), pile axial strains were also recorded by means of strain gauges but these are not reported here.

2 NUMERICAL ANALYSIS

2.1 The Finite Element model

In addition to the models employed during design stage, a new model was developed by using TNO DIANA FE software package. The finite element mesh consists of about 690,000 tetrahedral linear elements (soil block, raft, structural cores columns and walls) and 62 embedded pile (EP) elements pinned to the raft (Tradigo et al. 2016).

The behaviour of both raft and pile elements is assumed to be linear elastic, while a non-associated Mohr-Coulomb constitutive relationship is employed for the soil and for pile shaft interfaces. A linear dependence of the soil Young's modulus with depth (z axis) is also introduced to mimic the soil

stiffening induced by the increase in normal confinement (see Eq. 1).

$$E(z) = E_0 + E_{inc} z \quad (1)$$

Preliminary analyses on the influence of the element size showed that in the present case (which refers to Serviceability Limit State only) plasticity was not playing a significant role so that a good trade-off between accuracy and analysis time was found. In the proximity of structural elements an average edge length of about half of the pile radius (e.g. 0.5-0.7 m) is considered to avoid excessive soil strain concentration in correspondence of EPs.

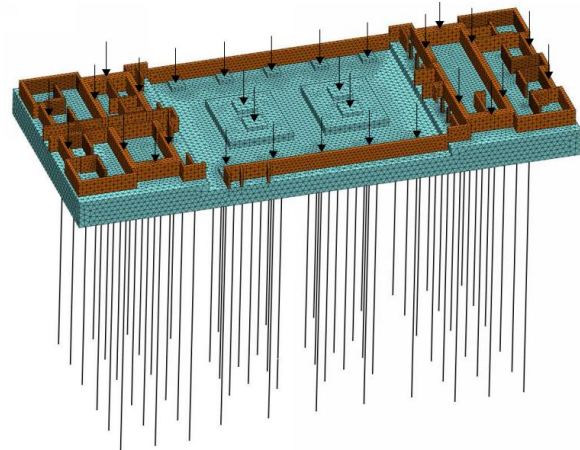


Figure 3. Piled raft FE: mesh and point loads

2.1.1 Soil parameters

While a more detailed description the site soil parameters can be found in the literature (Allievi et al. 2013, Tradigo 2015), this section focuses on the choice of the more adequate soil Young's modulus. On the basis of the soil small strain shear modulus profile with depth $G_0(z)$ obtained from cross hole tests, various Young's modulus profiles can be obtained depending on the soil shear strain (Seed and Idriss 1970). In case of PR foundations, soil domain is expected to attain a very wide range of shear strains, depending on the spatial distribution of piles and on the load level. Nonetheless, typical values ranging from $\gamma = 0.01\%$ and $\gamma = 0.1\%$ are expected. For this reason, the Young's modulus profile obtained from cross hole tests (E_{CH} , $\gamma \approx 10^{-4}\%$) and the profile obtained from SPT (E_{SPT} , related to very high shear strains $\gamma \geq 1\%$) are considered in the analyses as an upper and lower bound, respectively (Fig. 4).

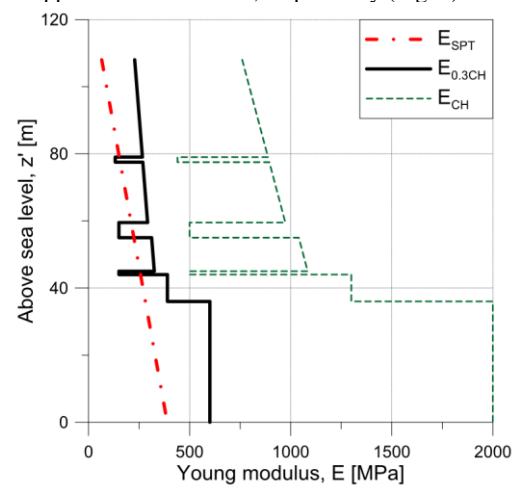


Figure 4. Young's modulus profiles for different shear strain levels.

It is worth noting that, in case of models which are accounting for a fully coupled raft-soil-pile mechanism, soil Young's modulus influences not only the pile but also the raft response. Therefore, one should consider that, although the two mechanisms may be governed by different strain levels, only one stiffness parameter can be set by employing a simple Mohr-Coulomb constitutive model. To this end, in case of PR/DPR foundations, the overall response at service load levels is mainly dominated by the pile response which is in fact considered for calibration purposes. In the present case, single pile load test back-analyses (see section 3.1) suggested to select a $G_{analysis}/G_0=0.3$ ratio ($E_{0.3CH}$). Similar values were employed during design (Allievi et al. 2013).

2.1.2 Pile interfaces

Pile interfaces have been set on the basis of pile capacity obtained from two preliminary (P1 and P2) and three contract (P3, P4, P5) pile load tests (Table 2).

The stress-dependent behaviour of the soil-shaft interface is modelled according to a non-dilative MC interface relationship. The non-linear pile base failure mechanism is modelled with a point-to-solid base interface element. Pile tip interface responds to a non-linear equation which was originally proposed by Butterfield (1980) to analytically interpolate the failure mechanism response of shallow footings: (see Eq. 2, where R is the initial tangent stiffness parameter and Q_b the pile base bearing capacity). All the interface parameters are summarised in Table 2 and Table 3.

$$Q = Q_b \left(1 - e^{-\frac{wR}{Q_b}} \right) \quad (2)$$

Table 2 Preliminary and contract pile estimated bearing capacity.

Pile	d (m)	Shaft Q_s (kN)	Base Q_b (kN)	Total Q_{tot} (kN)
P1 and P2	1.0	9150	10450	19600
P3 and P4	1.2	11000	15000	26000
P5	1.5	13700	18800	32500

Table 3 Preliminary and contract pile estimated interface parameters.

Layer	Int. friction angle Φ_{int} (°)	Interface stiffness $k_n=k_t$ (-)	Tangent stiffness R (-)
B	38	1e7	-
C	38	1e7	-
D	25	1e7	-
E	38	1e7	1e6

3 RESULTS

3.1 Single pile

Details on the single pile load test back-analysis procedure can be found in the literature (Tradigo et al. 2015b). As shown in Figure 5, a satisfactory agreement between numerical and experimental results is found in all cases, and especially in the green zone which corresponds to shaft mobilisation. It is worth noting that at higher loads a significant scatter appears even among piles with the same diameter. This behaviour was somehow expected and may be related to different degrees of cleaning of the pile base before the pouring of the concrete.

The best agreement is found for contract piles P3 and P4, which refer to 52 out of the 62 bored piles of which the foundation is made.

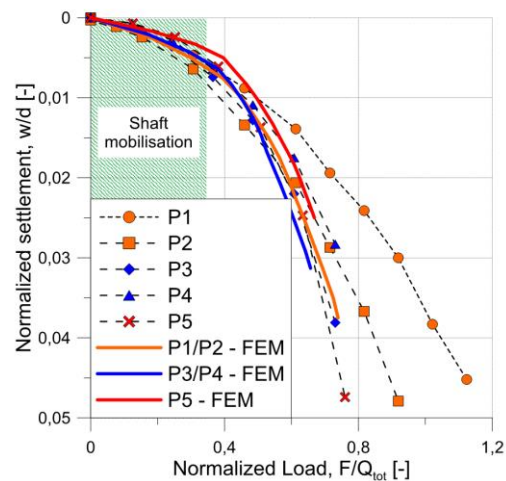


Figure 5. Normalised load-settlement curves from preliminary/contract pile load tests and finite element models.

3.2 Piled raft

3.2.1 Settlements

Figure 6 illustrates the FE settlement results obtained with the three different soil stiffness moduli profiles (Figure 4) compared with available monitoring data. In the Figure, the hatched area refers to the first construction period in which no monitoring activities were undertaken. Consequently, the first part of the measured load-settlement curve was extrapolated by maintaining the same curve tangent stiffness. This can be done under the realistic hypothesis of a nearly linear behaviour at low load levels, as it is also confirmed by numerical results.

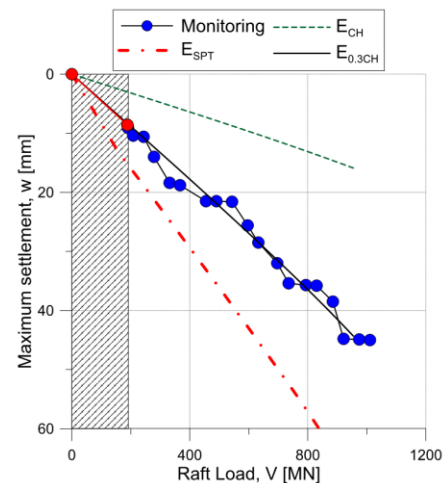


Figure 6. Comparison of foundation settlements from finite element models and monitoring data.

An excellent agreement is found in the $E_{0.3CH}$ case, while poor predictions are obtained in the other two cases. This was somehow expected because, in the presence of a fully coupled raft-pile mechanism, a variation of soil Young's modulus would affect not only the behaviour of the unpiled raft, but also the stiffness provided by every pile to the whole foundation system. The importance of performing pile load tests during the design phase is further testified by the good agreement found between monitored and computed results. In fact, it has been shown that the many uncertainties that may rise during the model calibration can be minimized by a proper calibration process, obtaining meaningful predictions. Indeed, this can never be an automated process and designers always need to be fully aware of the implications related to every modelling choice, as well as critically discuss numerical results.

Soil shear strain level at the final calculation stage are in good agreement with the design hypothesis of $\gamma \approx 0.1\%$ (Figure 7). On the other hand, higher and lower strains can be observed in correspondence of the raft edge and raft center, respectively. The stress state of the soil underneath the raft center is nearly in oedometric conditions, so that principal stresses almost coincide with vertical and horizontal stresses and low shear strain develop. As a consequence, soil in-situ operative stiffness is higher than the values employed in the numerical analyses. More advanced constitutive models with elasto-plastic hardening can be employed to account for such behaviour.

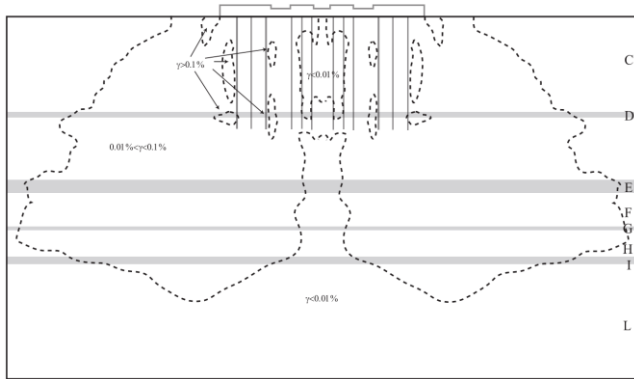


Figure 7. Final shear strains in the soil domain, V=950 MN.

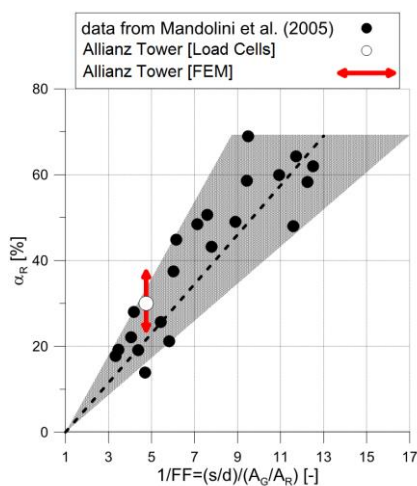


Figure 8. Raft and load sharing ratios (s =pile spacing; FF =filling factor, A_G/A_R =ratio between pile group area and raft area).

3.2.3 Load sharing

Numerical results (not presented here for brevity but reported in Tradigo 2015) show a variation of α_R and α_G at increasing loads. In particular, the raft contribution decreases due to the progressive pile resistance mobilisation. Quite significant variations occur also at service loads when pile resistances are not fully mobilized. After construction, while in absolute terms the corner and the larger diameter piles attract higher loads, a quite uniform load distribution can be noted in terms of pile resistance mobilization. In Figure 8, a good agreement was found between literature results and the present case. The original figure by Mandolini et al. (2005) is updated in Figure 8 in order to show that if $1/FF \rightarrow 1$ ($A_G/A_R = s/d = 1$) the load coming from the superstructure may be resisted by piles only. It is worth noting that, as also indicated by the wide hatched area in Figure 8, this trend should be interpreted with care and is not to be regarded as a general rule. In fact, if on the one hand it represents in a synthetic picture a number of real case histories and provides a useful and interesting insight into PR behaviour, only a few geometrical parameters are taken into

consideration so that a number of factors which may affect the response of a PR are not included. Among other aspects one may note that, as demonstrated by the Allianz Tower case study, load sharing is not a constant value and may significantly vary during the loading process.

4 CONCLUSION

In the present paper, the design of the Allianz Tower in Milan has been successfully validated on the basis of the monitoring data available at the end of construction.

The present study confirmed the crucial role played by model calibration against pile load tests in predicting the behaviour of the foundation. In fact, pile load tests allowed to choose all the significant model parameters. It is worth noting that in this respect pile load tests can give a much better estimate of soil properties compared to site investigations, limiting unnecessary conservative assumptions. In the present case, the use of SPT correlations to estimate soil Young's modulus would have led to significantly overestimating the raft settlements. Similarly, while soil-pile interface properties typically estimated as a reduction of the soil characteristic mechanical properties, back analysis resulted in $\Phi_{int} > \Phi_{soil}$ in the presence of granular soil layers.

A very good agreement was found between the FE model (calibrated against pile load tests before monitoring data were made available) and monitoring analyses in terms of both average foundation settlements and raft-pile load sharing. In particular, load-sharing values corroborated the observations available in the literature, which have been also critically discussed and presented in a renewed manner.

The sharing of monitoring results of a high-rise building in the Milan area is particularly valuable given the lack of published case histories.

5 ACKNOWLEDGEMENTS

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