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Deep foundation at Designer Outlet Croatia project by using FDP and CFA piles

Fondations profondes au projet Designer Outlet Croatia en utilisant des pieux FDP et CFA

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ABSTRACT: The paper describes a case study of deep foundation project executed in Zagreb, Croatia: Designer Outlet Croatia. Two types of pile installation technologies were used for deep foundations of the structures: Full displacement piles (FDP) and Continues Flight Auger Piles (CFA). The diameter of FDP piles was 40 cm, whereas larger 60 cm diameter CFA piles were used as energy piles for thermal consumption. Geotechnical ground conditions are alluvial deposits near Sava River, consisting of three characteristic layers: top layer of medium stiff clay/silt crust, second layer or soft and loose deposits of clay and silt materials with quite large amount of peat inclusions; bottom layer of dense gravel with inclusions of peat/clay/sand. The geotechnical investigation works are described in the paper including investigation borings, laboratory and in-situ tests (SPT, CPTU). The interpretation of the soil parameters and pile capacity calculation is described in detail (alpha and betta pile capacity calculation method). The validation of the pile capacity calculation was performed according to static pile load tests of two FDP piles and two CFA piles. The performance and the results of pile tests are presented in the paper. The advantages and disadvantages of FDP and CFA installation technology are discussed taking into account time of pile execution, excavation material deposition and capability to penetrate into deep layers of dense gravel.

RÉSUMÉ: Cet article décrit l'analyse d'un cas de fondations profondes exécutées à Zagreb, en Croatie: le projet Designer Outlet Croatia. Deux types de technologies d'installation des pieux ont été utilisés pour les fondations des structures: les pieux de déplacement (FDP) et les pieux de tarière continue creuse (CFA). Le diamètre des pieux FDP était de 40 cm, alors que les pieux CFA de diamètre plus grand de 60 cm étaient utilisés comme pieux énergétiques pour la consommation thermique. Les conditions géotechniques du sol sont des dépôts alluviaux près de la rivière Sava, constitués de trois couches caractéristiques: couche supérieure de croûte de moyennement raide argile/silt, deuxième couche de dépôts tendres et meubles d'argile et silt avec une assez grande quantité d'inclusions de tourbe; couche inférieure de gravier dense avec des inclusions de tourbe/argile/sable. Les essais préalables sont décrits dans l'article, notamment les sondages d'investigation, les essais de laboratoire et les essais in situ (SPT, CPTU). L'interprétation des paramètres de sol et le calcul de la capacité portante des pieux sont décrits en détail (méthodes alpha et betta de calcul de la capacité portante des pieux). La validation du calcul de la capacité portante des pieux a été effectuée selon les essais de chargement statique des deux pieux FDP et des deux pieux CFA. La description d'exécution et les résultats des essais de chargement des pieux sont présentés dans l'article. Les avantages et les inconvénients des technologies d'installation FDP et CFA sont discutés en prenant en compte le temps d'exécution des pieux, le dépôt des déblais et l'aptitude de pénétration dans les couches profondes de gravier dense.

Keywords: deep foundation, CFA piles, FDP piles, pile load test, soft soil

1 INTRODUCTION

Designer Outlet Croatia is new shopping centre constructed in eastern part of city of Zagreb, the capital of Croatia. The total surface of the construction area is around 25.000 m^2 where different types of structures were built: reinforced concrete solid and mounted structures, small towers and huge commercial panels. Due to pure geotechnical conditions of the foundation soil, all structures were built on pile foundation. The schematic view of the mounted reinforced structure, foundation system and soil profile are shown on Figure 1. The typical raster of structure columns is 10 x 8 m to 15 x 8 m, the foundation system consists of foundation plate connected to reinforced concrete piles. The foundation plate was constructed as Nautilus system to minimize the weigh of structure and to ensure necessary rigidity of the plate. The system consists of plastic inclusions that are placed in the central part of the plate, which remain empty after pouring the concrete. The piles are constructed as Continues Flight Auger piles (CFA piles) with 60 cm in diameter, and Fully Displaced Piles (FDP piles) with 60 cm in diameter. Larger CFA piles were used as energy piles for thermal consumption. All piles are imbedded into the bearing stratum at depth of app. 16 m below the ground level. Paper describes the soil investigation works performed on the site, the results of laboratory and in-situ testing of sol, and derivation of corresponding soil parameters. The bearing capacity of piles is calculated according to modern approach by using *alpha* and *betta* method both for CFA piles and FDP piles taking into account the difference of installation technology, excavation and displacement of the soil (see Figure 2). To validate the design calculation, static pile load tests were performed on two FDP piles, and two CFA piles with different embedment depth into the bearing stratum. Detailed description of the static load test and interpretation is given in the paper, together with the comparison of design and measured results.

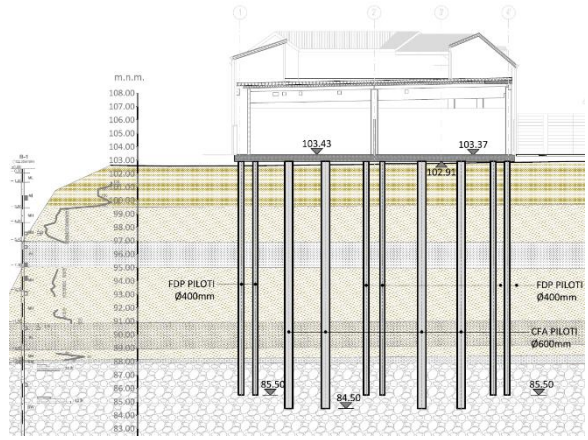

Figure 1. Schematic view of the mounted reinforced structure and foundation system: reinforced concrete slab with FDP and CFA piles

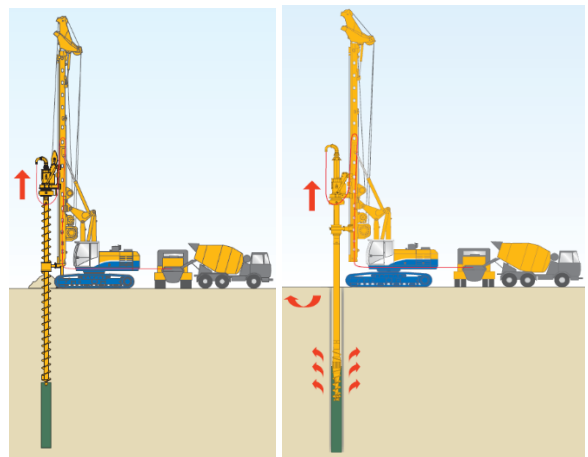

Figure 2. Comparison of different types of pile installation technology: CFA pile on the left and FDP pile on the right side

2 SOIL PROFILE

The soil profile generally consists of app. 3 m thick weathered crust, soft soil to the depth of app. 16 m below the ground level, and bearing stratum of dense gravel at the bottom. The underground water level is relatively high, app. 1.8 m below the ground level. The results of laboratory and in-situ geotechnical testing, including physical and mechanical properties of the soil, is presented on Figure 3. and Figure 4.

The soil investigation performed on the site included:

- Archive desk study which included systematic collection and validation of previous geotechnical, geological and hydrological investigations at the site and surrounding area.
- Investigation boring (five boreholes to the depth of 20 m during the of main design plus five additional boreholes to the depth of 25 m during the construction stage)
- Geotechnical soil classification of continuous core samples
- Pocket shear and penetrometer testing
- SPT in-situ testing in boreholes (2 m interval of measurements)
- CPTU in-situ testing (eight probes)
- Laboratory testing (soil classification, direct shear, uniaxial strength, UU triaxial test, oedometer test with permeability measurement)

The upper layer of weather crust consists of intermediate to high plasticity clay material to the depth of 2,6 to 4,1 m below the ground surface. The material is firm to stiff with average N_{SPT} value of 13 blows and cone penetration resistance q_t up-to 5 MPa.

The middle layer consists of soft silt and clay material, loose sand and peat. The N_{SPT} value is in the range of 3 to 6 blows, and cone penetration resistance q_t mostly between 0,5 to 2,0 MPa, with local increase in sand and peat lenses from 3,0 to 6,0 MPa. The natural water content of the peat material is grater then 100%. The uniaxial undrained shear strength is in the range 25 to 50 kPa and oedometer stiffness 1,0 to 3,0 MPa. The consistency index is in the range 0,75 to 1,0.

The third layer consists of well graded dense silty gravel with maximum particle size in the range of 3 to 5 cm. The N_{SPT} value is in the range 24 to more than 50 blows (average value 41 blows) and cone penetration resistance up-to 100 MPa (average value 41 MPa). The CPTU testing in this project was performed to the upper part of gravel layer, while the results of cone penetration trough the gravel layer was previously performed on the neighbour location.

Detailed results of laboratory and in-situ testing are shown on Figure 3. and 4. It can be seen that the results correlate quite well and fall within the narrow range. This gives the great confidence to correlations used to derive the mechanical parameters of the soil layers.

The following correlations were used to calculate constrained modulus M_v and undrained strength c_u according to N_{SPT} testing (after Clayton 1995):

$$\mathrm{M}_v[MPa] = f_1 N_{SPT} \tag{1}$$

$$\mathrm{c}_v[kPa] = f_2 N_{SPT} \tag{2}$$

where N_{SPT} is the number of blows according to SPT test; f_1 is the correlation coefficient depending on soil bearing capacity mobilisation (f_1 = 0,5 for soft soil clayly material; f_1 = 1,2 for sand); f_2 is correlation coefficient depending on plasticity index of soil (f_2 = 6,0 was used).

Following correlations were used to derive the soil parameters from CPTU testing (Lunne et al.1997):

$$\mathrm{M}_v = \propto_c (q_t - \sigma_{v0}) \tag{3}$$

$$\mathrm{c}_v = (q_t - \sigma_{v0})/N_{kt} \tag{4}$$

where q_t is normalised cone resistance, σ_{v0} is total vertical stress in the soil, α_c and N_k are the correlation coefficients (α_c = 3,0 and N_k = 22 were used for all soils).

Following corelation was used to calculate relative density of the gravel layer (Skempton 1986):

$$ID = \sqrt{(N_1)_{60}/(0{,}28(\sigma'_v) + 27)} \tag{5}$$

where $(N_1)_{60}$ is the normalized N_{SPT} value to the in situ state of stress, σ'_v is the vertical effective stress at the position of testing.

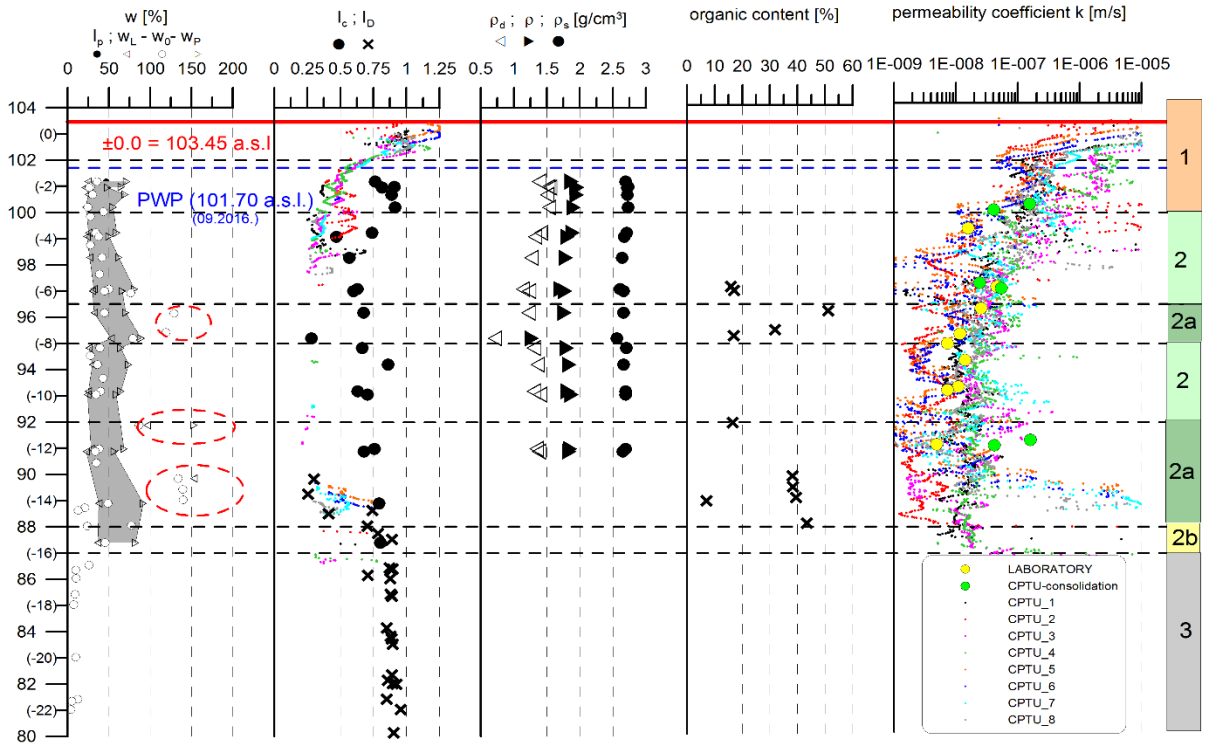

Figure 3. Results of laboratory testing with physical characteristics of soil material

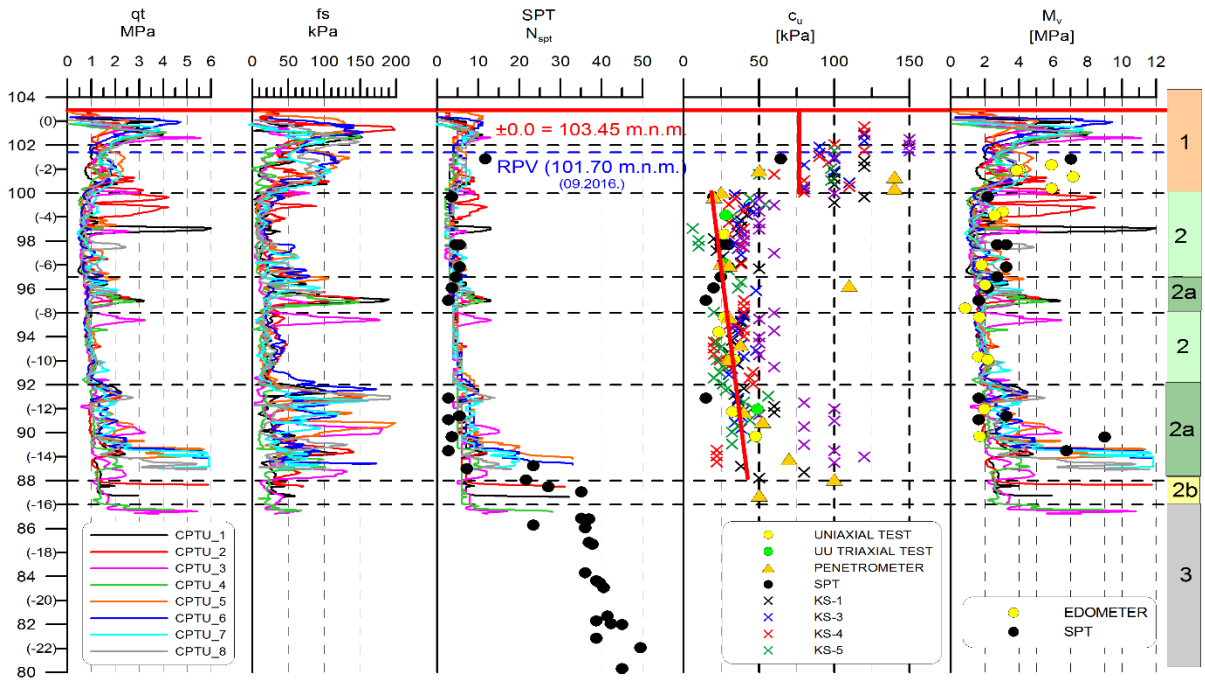

Figure 4. Results of in-situ and laboratory testing with mechanical characteristics of soil material

3 BERING CAPACITY OF PILES

Bearing capacity of piles were calculated according to *alpha* and *betta* method both for CFA and FDP piles. The difference between the calculations is due to stress change in soil during the installation of the piles. During the CFA pile installation the soil is excavated so there is slight release of horizontal stress in the ground while during the FDP pile installation the soil is pushed aside by the probe causing the increase of both side friction and base resistance.

The bearing capacity for CFA piles was calculated according to following equations:

$$q_s = \alpha c_u; \text{ for cohesive soils} \quad (6)$$

$$q_s = \beta \sigma'_{v0}; \text{ for noncohesive soils} \quad (7)$$

where q_s is shaft resistance, c_u is undrained shear strength of soil, σ'_{v0} is vertical effective stress in the ground, α is empirical coefficient of shear resistance for cohesive soils which depends on c_u and reference pressure p_a (Rees at al. 2006):

$$\alpha = 0{,}55 \quad (8)$$

for $c_u/p_a \leq 1{,}5$

$$\alpha = 0{,}55 - 0{,}1(c_u/p_a - 1{,}5) \quad (9)$$

for $1{,}5 \leq c_u/p_a \leq 2{,}5$

β is empirical coefficient of shear resistance for noncohesive soils depending on the depth z:

<u>For sand</u>

$$\beta = 1{,}5 - 0{,}25z^{1/2} \quad (10)$$

for $1{,}5 \leq c_u/p_a \leq 2{,}5$

$$\beta = N_{60}/15\left(1{,}5 - 0{,}25z^{1/2}\right) \quad (11)$$

for $N_{60} \leq 15$

<u>For gravel</u>

$$\beta = 2{,}0 - 0{,}15z^{3/4} \quad (12)$$

q_b is base resistance:

$$q_b = 0{,}06N_{60}(d/10b) \quad (13)$$

for $d/b \leq 10$

$$q_b = 0{,}06N_{60} \quad (14)$$

for $d/b > 10$

Maximum alowable walue for CFA piles is qb = 3.0 MPa

The bearing capacity for FDP piles was calculated according to following equation:

$$\alpha = 0{,}5(c_u/\sigma'_{v0})^{-1/2} \quad (15)$$

for $c_u/\sigma'_{v0} \leq 1{,}0$

$$\alpha = 0{,}5(c_u/\sigma'_{v0})^{-1/4} \quad (16)$$

for $c_u/\sigma'_{v0} > 1{,}0$

$$\beta = Ktg\delta \quad (17)$$

where K – horizontal stress in the ground, δ is friction angle between soil and pile (Table 1).

$$q_b = N_q \sigma'_{v0}; \text{ for noncohesive soils} \quad (18)$$

where N_q is empirical coefficient (Table 1).

Table 1. Bearing capacity coefficient N_q and δ

N_{SPT}	0-4	4-10	10-30	30-50	>50
N_q	8	12	20	40	50
δ[°]	15	20	25	30	35

Table 2. Bearing capacity for CFA and FDP piles

resistance	unit	CFA	FDP
diameter	cm	60	40
$q_{s(1)}$	kPa	-	-
$q_{s(2a,2b)}$	kPa	17	27
$q_{s(2c)}$	kPa	80	80
$q_{s(3)}$	kPa	100	100
$q_{b(3)}$	kPa	3.000	7.000
R_k(d=2,0 m)	kN	1.871	**1.671**
R_k(d=3,0 m)	kN	**1.966**	1.797
R_k(d=4,0 m)	kN	2.154	1.923
R_k(d=5,0 m)	kN	**2.334**	**2.048**

d – embeddment depth into the gravel layer

$q_{s(i)}$ – is the skin resistance in *i* layer of soil

It is evident from the pile bearing capacity calculation that FDP piles are more efficient (almost the same calculated bearing capacity for 40 cm FDP pile and 60 cm CFA pile). The CFA piles were used only for energy piles because there is enough space inside the reinforcement casing to install the pipes for thermal consumption.

4 PILE INSTALATION

CFA piles are constructed in following sequence:

- Drilling the soil by continuous flight auger to the bottom of the pile.
- Continuous excavation of the soil material
- Extracting the drilling equipment by simultaneous pouring of the concrete trough the centre of the probe
- Insertion of the reinforcement casing

FDP piles are constructed in following sequence:

- Pushing the 'drilling' probe to the bottom of the pile
- The soil material is pushed aside
- Extracting the probe by simultaneous pouring of the concrete trough the probe
- Insertion of the reinforcement casing

The main advantages of FDP technology are following. The installation of the pile is very quick. In this project 18 m piles were installed in average sequence of 30 min per pile and minimum 25 minutes per pile (positioning of the equipment = 5 min.; drilling 16 m in the soft soil = 5 min; drilling 2 to 3 m in-to the dense gravel = 5 min; concreting = 5 min; reinforcement installation = 5 min). The soil is pushed aside during the installation which increase the horizontal stress in the ground resulting with higher shaft resistance. There is minimum extraction of the soil leading to minimum cleaning of the working plateau comparing to CFA technology. During the installation of each pile the process is monitored by drilling equipment (see Figure 5) and measuring: the depth of probe in time, concrete pressure, penetration resistance (*alfa value*) the penetration rate (see Figure 6. and 7). Those parameters are curtail to validate the quality of pile installation: total length of pile, bearing stratum depth, continuity of concreting (pile integrity). For this project it was concluded that bearing stratum is reached for alfa > 2000 (Figure 7).

Figure 5. FDP displacement probe and monitoring

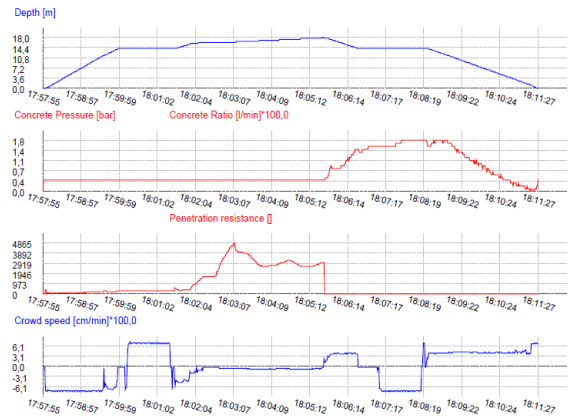

Figure 6. Parameters measured for FDP in time

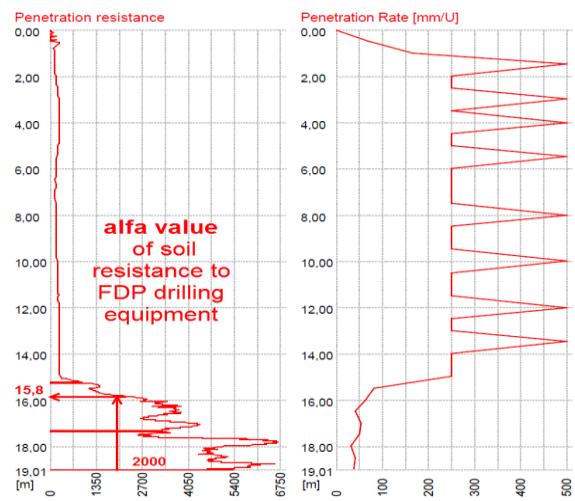

Figure 7. Penetration resistance and rate per depth

5 STATIC PILE LOAD TEST

Static pile load tests were performed on four test piles:

- FDP pile, 2 m into the gravel layer
- FDP pile, 4 m into the gravel layer
- CFA pile, 3 m into the gravel layer
- CFA pile, 5 m into the gravel layer

The testing was performed according to Suggested method ISSMGE, Subcommittee on Field and Laboratory Testing: ‘’Axial Pile Loading Test – Part 1: Static Loading’’. Four tension piles were used for each test pile to retain the tension forces of the system (Figure 8). The driving force was applied by 4800 kN load cell. The displacement of the pile was measured at four points symmetrically around the pile head with 0,01 mm accuracy. The load was applied in 250 kN increments both for loading and unloading. During the loading stage, each load was retained minimum 30 min, and 3 hours at the pile working load. The settlement and creep of pile was measured during the testing. Target test load was 2250 kN for FDP piles and 2750 kN for CFA piles. Additionally strain gauge sensors were installed in the pile at three characteristic points: at the top of the pile, at the bottom of soft soil layer, and near the bottom of the pile (Figure 10). The sensors were used to measure the axial force in the pile which enables to extract shaft resistance and base resistance from total applied load (Figure 12). The sensor at the top of the pile was used as reference measurement for calibration of axial pile stiffness. The results of calibration are shown on Figure 11. for FDP piles.

The results of static pile load tests are shown on Figure 10. It can be seen that the prediction of the FDP pile settlement curve is ideal for pile embedment 2 m into the bearing stratum and quite well for pile embedment of 4 m. On the other hand, the load settlement calculation for the CFA is underestimated both in stiffness and ultimate bearing capacity. The main reason for this difference can be the fact that surface crust was not included into the calculation, and the bearing capacity of pile in gravel layer is much bigger than predicted. Back calculation shows that the skin friction of gravel layer should be app. 60% bigger than used in calculation (q_s = 160 kPa), and base resistance 100% bigger (q_b = 6.000 kPa) to get relatively good correlation of measured and calculated load-settlement curves (Figure 9).

Figure 8. Static pile load test setup

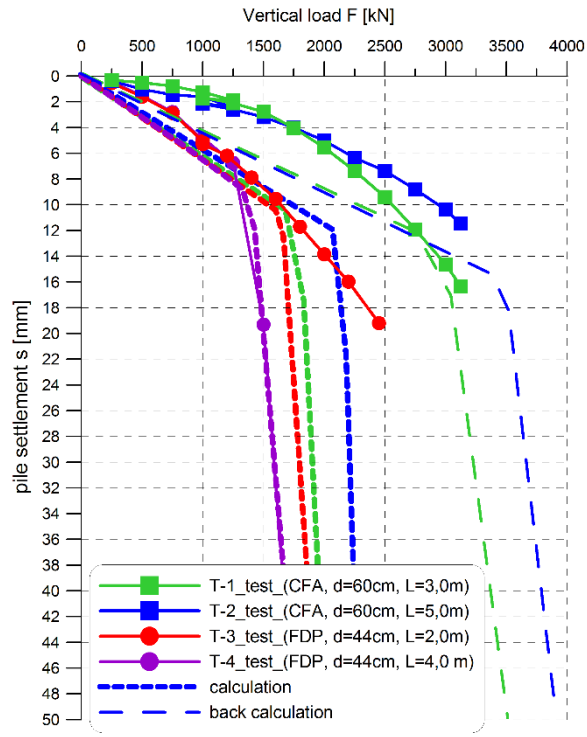

Figure 9. Static pile load test results (calculation is presented with dashed line)

Figure 10. Strain gauge sensor installation

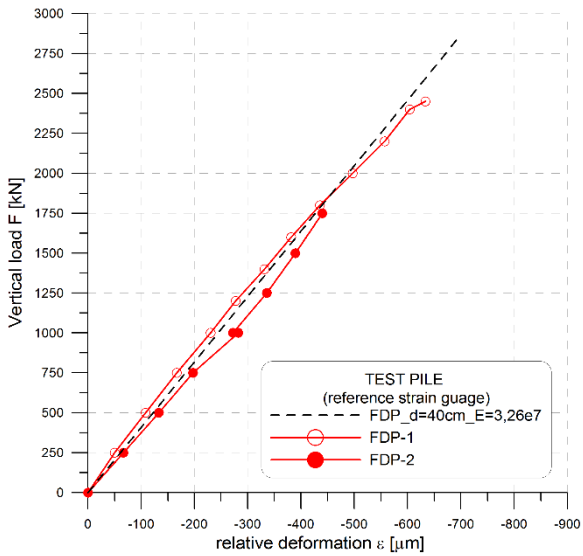

Figure 11. Calibration of pile axial stiffness

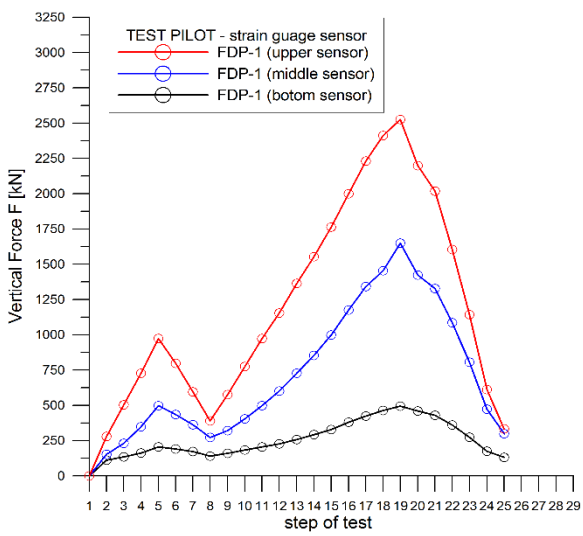

Figure 12. Skin and Base resistance of FDP test pile

6 CONCLUSIONS

Deep foundation for Designer Outlet Centre in Zagreb, Croatia was successfully constructed by using CFA and FDP reinforced concrete piles. The performance of FDP piles was almost ideal as calculated in main design. The execution of the piles was very quick and with high rate of quality control which lead to optimum and safe design. The performance of CFA pile was underestimated for app. 30%. It is mostly due to underestimation of bearing capacity in gravel layer. One of the main reason could be due to correlations for noncohesive soils which are mostly derived for sandy material, and also due to quite scatter of the N_{SPT} results which are often the case in gravel soil. According to Eurocode 7 principles the design must be performed according to characteristic value of soil parameter which should be derived as 95% fractal value. On the other hand during the pile load test the shaft friction is sum of real skin friction along the pile shaft which is equal to average skin friction along shaft area.

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