

Ground Movement Analysis		 6 th Floor, 42-50 Kimpton Road, Luton, Bedfordshire, LU2 0FP
Project: Ark Project Union		
Title: Ground Movement Analysis of piles adjacent to the Thames Water Main		
Doc No.:		SFF Doc No.: 7039-GMA-01
Author: Giovanni Ferzola – 16/02/2021	Checked: Liam Connor – 17/02/2021	Client: JSM Group

1 INTRODUCTION

This analysis has been carried out in order to estimate the potential damage induced by the proposed works on a Thames Water (TW) asset located in vicinity of the Ark Project Union site. The assessment has been undertaken using Tomlinson on Woodward Pile Design and Construction Practice 3rd edition, Eurocode 7, BS 8004:2015 and Perko (2009).

ScrewFast have designed helical pile foundations. To understand these pile types and the design process please refer to ScrewFast's pile design and drawings (7039-Type4-HPD, 7039-01). The table below (Table 1) lists all the different pile types that SFF has designed for this project.

Table 1: Piles to be installed by ScrewFast Foundations

Structure	Pile Type	Pile Size (mm)	Pile length (m)	Maximum helix diameter (mm)
11kV Switchgear Building	1	140	6	550
11kV Switchgear Building	2	140	6.5	600
11kV Switchgear Building	3	170	7	600
11kV Switchgear Building	4	170	7.5	600
11kV Switchgear Building	5	194	9	800
11kV Switchgear Building	6	220	9	800
11kV Switchgear Building	7	244	10.5	800
11kV Switchgear Building	8	244	12.5	800
Transformer base and bund, disconnector base & Sealing end base	1a	140	6	550
Transformer base and bund, disconnector base & Sealing end base	3a	170	7	600
Transformer base and bund, disconnector base & Sealing end base	5a	194	9	800
Transformer base and bund, disconnector base & Sealing end base	7a	244	10.5	800
Transformer base and bund, disconnector base & Sealing end base	9	244	12.5	800

2 REVIEW OF GROUND INVESTIGATION AND ASSUMED GROUND CONDITIONS

A site wide ground investigation (GI), Hayes Phase 2 Ground Investigation Report, has been received. The data from this report has been used to develop a detailed ground model with specific reference to BH6 which is the most relevant BH to the existing site location.

The material described is a cohesive made ground, over layers of London Clay. Please find below the ground model assumed:

Table 2: Ground model summary and geotechnical parameters required for analysis.

GROUND CONDITIONS USING CHARACTERISTIC VALUES (f _k) FROM GROUND TEST RESULTS										
Strata description	Top of layer (mADD)	Bottom of layer (mADD)	Cumulative depth (mbgl)	Strata type for purposes of analysis	Unit weight γ (kN/m ³)	Average SPT 'N' value	c _{u,k} (kPa)	Φ' _k (°)	α (see note)	Shaft resistance R _{s,k} (kN)
Made Ground	29.90	25.40	4.50	Made Ground (Ignored in analysis)	16					0.00
Made ground	25.40	24.40	5.50	Cohesive	18	18	105		0.72	39.97
London Clay	24.40	23.40	6.50	Cohesive	20	17	100		0.74	39.13
London Clay	23.40	22.40	7.50	Cohesive	20	15	90		0.76	36.17
London Clay	22.40	21.40	8.50	Cohesive	20	17	100		0.74	39.13
London Clay	21.40	20.40	9.50	Cohesive	20	17	103		0.73	39.76
London Clay	20.40	19.40	10.50	Cohesive	20	18	105		0.72	39.97
London Clay	19.40	18.40	11.50	Cohesive	20	18	110		0.71	41.29
London Clay	18.40	16.90	13.00	Cohesive	20	22	130		0.66	68.05
NOTE: SPT 'N' value is used to calculate anticipated torque during installation; it is not used in pile capacity calculation					TOTAL SHAFT RESISTANCE R _{s,k} = ΣA _s q _{s,k} (kN)					343.46
NOTE: Refer to A.3.3.2 and A.3.3.3 if characteristic values of ground strength are derived from static load tests					NOTE: Factored ground conditions for M1 Material Factors are same as R _{s,k} value above since all factors ≥ 1.0					

3 STRUCTURE

The structures involved in the project are a 11kV switchgear building and a transformer base and bund, disconnector base and sealing end base. In particular, the structure closest to the sewer is the 11kV switchgear building. The loads applying on this structure are from a minimum of 80kN to a maximum of 410kN. The switchgear building stays on discrete columns, each column is supported by a steel helical pile with a direct steel to steel moment connection.

Ground movements will arise as a result of various mechanisms which are mobilised as part of the implementation of the proposed scheme. The works will involve the installation of the piles. The result is settlement extending over a given zone of influence surrounding the piles. The deflection of the single pile will stay under a 10mm limit as from the document 7039-Allpiles-ALP. The global lateral deflection will be affected by the top of the piles connection and will be smaller.

4 HELICAL PILES: DESCRIPTION AND DESIGN

The use of ScrewFast's helical piling systems have been approved for this project. ScrewFast's helical piles are designed using conventional pile design methodologies and soil mechanics in accordance with Eurocode 7, BS 8004:2015 and Perko (2009) only. Please note that there is not excavation required for the structure involved and no soil arisings result from helical pile installation.

Helical piles are a form of prefabricated deep foundation. As hinted in its name, helical piles are most notable for having spiralling blades along their shafts, as opposed to having a relatively uniform shape like driven piles and micropiles. Helical piles comprise a lead section, which comprises helix plates attached to a central steel shaft, and extension sections. Helical steel piles are typically suitable for use in loose to medium dense coarse soils and low to medium strength fine soils (BS 8004:2015). Helical piles are fitted into the ground in a similar way that a screw is inserted into wood, although on

a much larger scale. Using machine-mounted hydraulic or electrically powered drilling equipment, they are screwed directly into the ground.

Helical piles are classed as low displacement piles in the codes, therefore parameters and partial factors relevant to this pile type are used in the design. Low displacement piles are piles that displace very little of the material they are driven into. The soil around a driven pile is disturbed during installation and with a pile group the volume of soil affected may be considerable. Mitigation of this effect can be obtained using low displacement piles (BS8004:2015).

The helical piles are particularly beneficial for this kind of structure. In fact, the installation process is relatively quick, the equipment used to install helical piles is not only easy-to-manage but also keeps noise, disturbance, vibration, and pollution to a minimum level. ScrewFast's helical piles have been specifically chosen on this substation job to reduce vibration and disturbance to the operating substation. One other and is a common reason for adopting the helical piling technique. Some examples of sensitive projects include pile installation within millimeters from services including water and gas mains and tree roots due to the lack of disturbance this technique creates. One particular project was at a hospital where helical piles were installed adjacent an operating magnetic resonance imaging (MRI) machine where vibration could affect the results of the equipment. During the monitoring of pile installation vibration and ground movement it was found that hand digging created larger vibrations than the pile installation highlight the advantage of this technique and the pile installation was satisfactory to the hospital's needs.

The bearing capacity of each helical pile is considered as the sum of the capacities of each helical plate along the shaft. Helical plate spacing of greater than 1.6 times helix diameter means that bearing capacity is always more critical than shaft and can therefore be ignored in the design. The capacity of each plate is calculated by considering it as an end-bearing face, with the helix plate area being net of the shaft diameter. Each helix plate therefore acts as an individual base plate supported directly by the ground above and below it for tensile and compressive loads respectively. The spacing of the helix plates and piles is designed to ensure that the bulbs of loaded soil as described by Boussinesq's theory do not overlap so that the pile would not suffer a shaft-type failure. The most likely failure mechanism is therefore the failure of an individual helix plate, which is checked during design.

The software, Oasys Alp, was used to calculate the horizontal pile capacities. The horizontal displacement (deflection) and bending are calculated under the combination of the shear force and bending moment. In this instance the generated P-Y curve method is used in Oasys Alp provided in Appendix B. The P-Y curves are generated from the same data set as those used in the vertical pile model. Lateral loads are supported by the passive resistance of the soil and fixed pile head with partial restraint to rotation.

5 SETTLEMENT OF HELICAL PILES

Equation 4.38 on page 195 of Tomlinson on Woodward Pile Design and Construction Practice 3rd edition is used to predict total settlement of stiff Over Consolidated clay such as London Clay.

Engineers (Section 11.4) using the maintained loading procedure. More generally the pile settlements can be calculated if the load carried by shaft friction and the load transferred to the base at the working load can be reliably estimated. The pile head settlement is then given by the sum of the elastic shortening of the shaft and the compression of the soil beneath the base as follows:

$$\rho = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4} \cdot \frac{W_b}{A_b} \cdot \frac{B(1 - \nu^2)I_p}{E_b} \quad (4.38)$$

Thomlinson also refers to. Equation 5.14 (p260) below similar to equation 4.38 but without the first term i.e. the elastic shortening of the pile shaft.

Another method of estimating the *total* settlement of a structure on an *over-consolidated* clay is to use equation 5.14, making the substitution of a deformation modulus obtained for loading under drained conditions. This modulus is designated by the term E_v' , which is substituted for E_u in the equation. It is approximately equal to $1/m_v$. The equation implies a homogeneous and elastic material and thus it is not strictly valid when used to calculate consolidation settlements. However, when applied to over-consolidated clays for which the settlements are relatively small, the method has been found by experience to give reasonably reliable predictions. Success in using the method depends on the collection of sufficient data

The table 3 below summarises the parameter used for bearing capacity and settlement calculations for the piles closest to the sewer. The closest pile type with highest load is pile type 4, therefore the pile is likely to induce the greatest ground movement, making it the most critical condition to check.

Table 3: Parameters used for bearing capacity and settlement calculations.

Depth (m)	N	Plate Area (m ²)	C _u (kPa)	Q _{ult} (kPa)	E _u (0.5*N) (kPa)	E' (0.3*N) (kPa)
4.25	18	0.26	105	945	9000	5400
5.25	18	0.26	105	945	9000	5400
6.25	17	0.26	100	900	8500	5100
7.25	15	0.26	90	810	7500	4500

5.1 DERIVATION OF PARAMETERS CIRIA R143 SPT, METHOD AND USE

C_u is determined from empirical correlations (Stroud and Butler (1975)). The C_u/N ratio is therefore = 5. The C_u values are assumed from direct measurements of C_u from hand vane tests. E_u and E' are the undrained and drained elastic moduli which can be determined with empirical correlations with SPT-N values (Butler 1975). The correlations are shown in page 86 to 87, as E_u/N= 1.0 (MPa) and E'/N = 0.6 (MPa). Over 200 tests without a single settlement failure and high levels of predictability of design settlement to tested settlement, realised by ScrewFast over years, has allowed for additional factors to improve the predictability of settlement with ScrewFast's helical piles

5.2 ELASTIC THEORY FOR DETERMINING TOTAL SETTLEMENT

In CIRIA SP27 Settlement of structures on clay soils, Table 4 from page 20, defines the components of total settlements based on drained and undrained conditions. It also discusses key points from the CIRIA SP27 below relating to total settlements, ρ_t of Over Consolidated Clays using drained parameters with elastic deformation theory.

Table 4: Definition of settlement components. CIRIA SP27 page 20

Table 1 *Definition of settlement components*

	Settlement	Stress/strain parameters
Drained conditions	$\rho_t = \text{Total settlement (which might more correctly be denoted by the symbol } \rho')$	E', v', m_v, σ' Effective stress parameters
Undrained or immediate conditions	$\rho_i = \text{Immediate ('elastic') settlement}$	E_u, v_u, σ Total stress parameters
Difference of above two categories: Consolidation	$\rho_c = \text{Consolidation component of settlement}$ $\rho_c = \rho_t - \rho_i$	Difference between total and effective stress is the porewater pressure $\sigma - \sigma' = u$

CIRIA SP27 page 54

$$\begin{array}{l}
 \rho_i = 0.5 \text{ to } 0.6 \rho_{od} \\
 \rho_c = 0.5 \text{ to } 0.4 \rho_{od} \\
 \rho_t = \rho_{od}
 \end{array}
 \left. \vphantom{\begin{array}{l} \rho_i \\ \rho_c \\ \rho_t \end{array}} \right\} \text{ for stiff clays}$$

$$\begin{array}{l}
 \rho_i = 0.1 \rho_{od} \\
 \rho_c = \rho_{od} \\
 \rho_t = 1.1 \rho_{od}
 \end{array}
 \left. \vphantom{\begin{array}{l} \rho_i \\ \rho_c \\ \rho_t \end{array}} \right\} \text{ for soft clays}$$

As shown in Table 1 of CIRIA SP27 the total settlement = immediate (undrained) settlement + consolidation (drained) settlement. The calculations below indicate that consolidation settlement $\rho_c = 0.4 * \rho_t$.

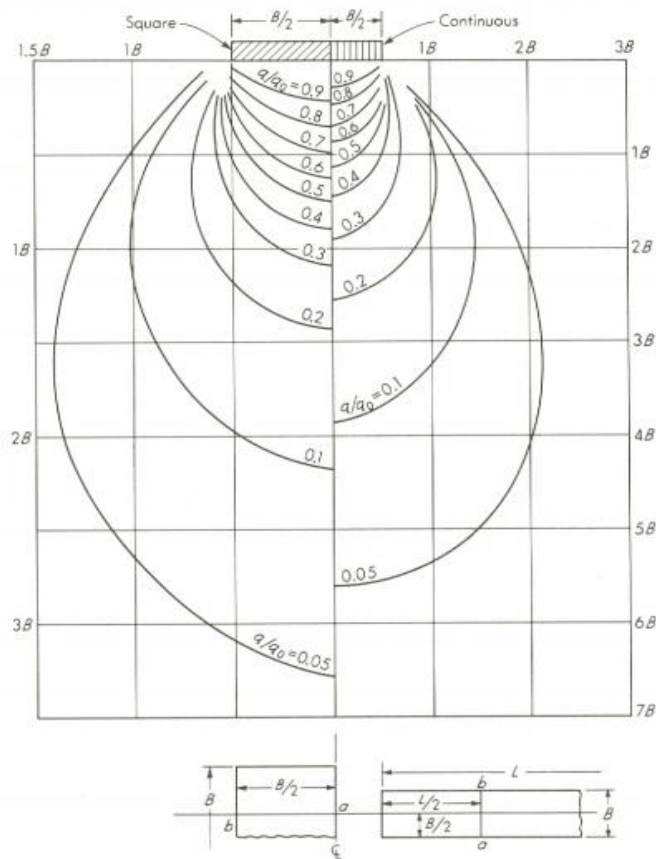
In the Appendix A the load and the settlement of each helix is calculated. Please find below a table 5 summarising the results.

Table 5: Summary of helix settlement calculations.

Pile type	Helix size (mm)	Helix depth (m)	N	Load per plate (kN)	Maximum drained settlement per plate (mm)	Maximum undrained settlement per plate (mm)
4	600	4.25	18	12.7	1.93	1.16
4	600	5.25	18	62.7	9.57	5.74
4	600	6.25	17	59.1	9.55	5.73
4	600	7.25	15	55.5	10.17	6.102

Considering Boussinesq's theory of settlement and consolidation (1885), each helix is considered as a circular footing. In the below Figure 1 it is possible to see the pressure isobars based on that theory.

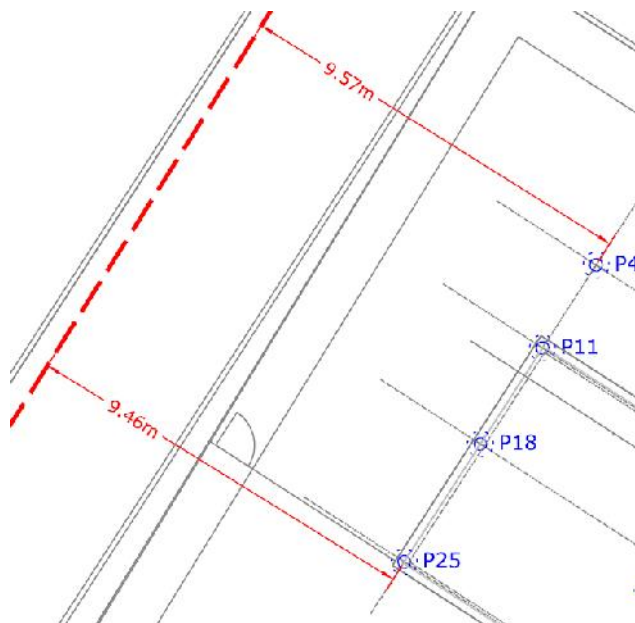
Figure 1: pressure isobars based on the Boussinesq (1885) equation for a footing



Calculating pressure bulb diameters and depth we see from the Figure 1 that the stresses reduce as the bulb expands. Even considering the absolute extremity, with B (helix diameter) of 0.6m, it can be seen that the soil deformation or movement decreases steadily up to a maximum limit of 5% ground movement at $3B$ or in this case 1.8m laterally and vertically. The worst case: Max helix settlement at $3B$ distance (1.8m) can be show to be 10.17mm settlement from table $\times 0.05 = 0.5\text{mm}$ at 1.8m from the helix centre depth and width.

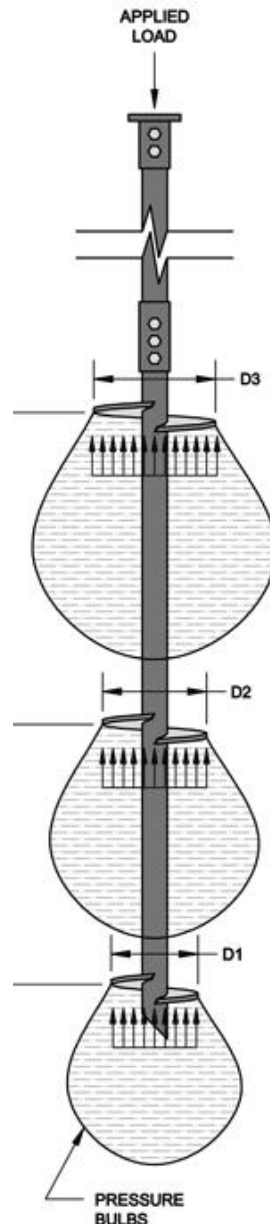
In Figure 2 the sewer locations are shown.

Figure 2: sewer locations



According to Figure 3 below, assuming a conservative 45 degree zone of influence from each helix and assuming the worst case plate settlement (10mm) from the helix, the 45deg zone of influence is more than 9m far from the water pipe. Additionally, at 45 degrees all the helices zone of influence passes under the water main which is understood to be 12m below ground level. This is far greater than 3B out lined above at more than 13B and the theory indicates that there will be no measurable ground movement result from the loading on any of the helices.

Figure 3: helices and bulbs of influence.



6 CONCLUSION

As the distance of influence from the water main is greater than the settlement at the point of 5% ground movement at three times the helix diameter and given Boussinesq theory of dissipating stress there is no risk of any stress, strain or ground movement resulting from the piles on this the identified water main.

7 REFERENCES

-) BS EN 1997-1: 2004 Eurocode 7: Geotechnical design (EC7)
-) Tomlinson, M. & Woodward, J., 2008. Pile Design and Construction Practice. 5th Edition. Taylor & Francis;
-) BS 8004:2015 Code of practice for foundations;
-) PERKO, H.A., 2009, Helical piles – a practical guide to design and installation., Hobokon, New Jersey: John-Wiley, 2009, ISBN 978-0-470-40479-9;
-) CIRIA SP 27 Settlement of structures on clay soils;
-) Boussinesq's theory of settlement and consolidation (1885);
-) Stroud, M. A. .and Butler, F.G. 1975. "The standard penetration test and the engineering properties of glacial materials" Proceedings of the symposium on Engineering Properties of glacial materials, Midlands, U.K.
-) CIRIA R143. The standard penetration test (SPT): methods and use.

APPENDIX A

SETTLEMENT CALCULATIONS

JFF 7039 - ARK UNION

16/02/2021

Settlement calculations of helical piles in clay

$$SWL = 190 \text{ kN (Type 4)}$$

Load distribution per plate: $SWL \times (\text{plate resistance} / \text{total resistance})$

(Plate resistance / total resistance) = R

Load distribution per plate W

$$R_1 = 35,2 / 527,5 \text{ (kN/kN)}$$

$$W_1 = SWL \times R_1 = 12,7$$

$$R_2 = 174,2 / 527,5$$

$$W_2 = SWL \times R_2 = 62,7$$

$$R_3 = 164,1 / 527,5$$

$$W_3 = SWL \times R_3 = 59,1$$

$$R_4 = 154,1 / 527,5$$

$$W_4 = SWL \times R_4 = 55,5$$

$$\text{Check} = 190 \text{ kN}$$

Pile head settlement is the sum of the elastic shortening of pile shaft and compression of the soil beneath base.

Ref: Tomlinson and Wordsworth, Pile design and construction practice 5th Ed

$$P = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4} \frac{W_b}{A_b} \frac{B(1-\nu^2)}{E_b} \cdot I_p$$

$$W_s = 0$$

$$W_b = 190 \text{ kN}$$

$$A_s = 4403 \text{ mm}^2 \text{ X-section area of steel}$$

$$L = 7500 \text{ mm Pile length}$$

$$E_p = 210 \text{ kN/mm}^2 \text{ elastic modulus of steel}$$

$$\text{Elastic shortening } P_{es} = \frac{(0 + 2 \cdot 190) \cdot 7500}{2 \cdot 4403 \cdot 210} = 1,54 \text{ mm (First term)}$$

Compression of soil (sand tremie)

$$P = \frac{\pi}{4} \cdot \frac{W_i}{A_{bi}} \cdot \frac{B_i (1 - \nu_i^2) l_p}{E_{bi}}$$

W_i = Load on plate

B_i = Diameter of plate

ν = Poisson ratio, taken as 0,3

I_p = Influence factor, taken as 0,5 (Toulemous and Winkworth)

E_{bi} = Deformation modulus under i^{th} plate DRAINED

A_{bi} = Area of plate

$$P_1 = \frac{\pi}{4} \cdot \frac{117}{0,26} \cdot \frac{0,6 (1 - 0,3^2) \cdot 0,5}{9,4} = 1,93 \text{ mm}$$

$$P_2 = \frac{\pi}{4} \cdot \frac{627}{0,26} \cdot \frac{0,6 (1 - 0,3^2) \cdot 0,5}{5,4} = 9,57 \text{ mm}$$

$$P_3 = \frac{\pi}{4} \cdot \frac{59,1}{0,26} \cdot \frac{0,6 (1 - 0,3^2) \cdot 0,5}{5,12} = 9,55 \text{ mm}$$

$$P_4 = \frac{\pi}{4} \cdot \frac{55,5}{0,26} \cdot \frac{0,6 (1 - 0,3^2) \cdot 0,5}{4,5} = 10,17 \text{ mm}$$

Max settlement: 10,17 mm DRAINED

Elastic shortening = 1,54 mm

Total settlement: 11,71 mm

2nd term (Undrawn Compression of the soil)

$$p_1 = \frac{\pi}{4} \cdot \frac{12,7}{0,26} \cdot \frac{0,6(1-0,3^2) \cdot 0,5}{9} = 1,16 \text{ mm}$$

$$p_2 = \frac{\pi}{4} \cdot \frac{62,7}{0,26} \cdot \frac{0,6(1-0,3^2) \cdot 0,5}{9} = 5,74 \text{ mm}$$

$$p_3 = \frac{\pi}{4} \cdot \frac{59,1}{0,26} \cdot \frac{0,6(1-0,3^2) \cdot 0,5}{8,5} = 5,73 \text{ mm}$$

$$p_4 = \frac{\pi}{4} \cdot \frac{55,5}{0,26} \cdot \frac{0,6(1-0,3^2) \cdot 0,5}{7,5} = 6,102 \text{ mm}$$

Max settlement = 6,102 mm UNDRAWN

Using (IRIA SP27 (Table 1, 920) technologies:

$$\rightarrow p_c = (p_4 - p_1) = 10,17 \text{ mm} - 6,102 \text{ mm} = 4,068 \text{ mm}$$

APPENDIX B

PILE DESIGN

SFF JOB No:	7039
JOB NAME:	Ark Project Union
CLIENT:	JSM Group

PILE DESIGN REF:	7039-Type4-HPD-00
LOAD REF:	P1152_ARK_DWG_2003_C and 20-112-03A
SFF DRAWING REF:	
CLIENT GA DRAWING REF:	P1152_ARK_DWG_2003_C

SFF REV No:	00
DATE:	
REVISION DESCRIPTION:	

LEVELS	
G.I. REF NUMBER:	Hayes Phase 2 G I Report Final - BH6
G.I. DATUM (mAOD)	29.90
TOP OF PILE (mAOD)	29.90
GROUND LEVEL (G.L.) (mAOD)	29.90
GROUNDWATER DEPTH (mAOD) (See note)	29.90
NOTE: Groundwater depth: 2.4.6.1(6)P: When dealing with ground-water pressures for limit states with severe consequences (generally ultimate limit states), design values shall represent the most unfavourable values that could occur during the design lifetime of the structure. For limit states with less severe consequences (generally serviceability limit states), design values shall be the most unfavourable values which could occur in normal circumstances. 2.4.6.1(11): Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design ground-water table should be taken as the maximum possible level, which may be the ground surface.	

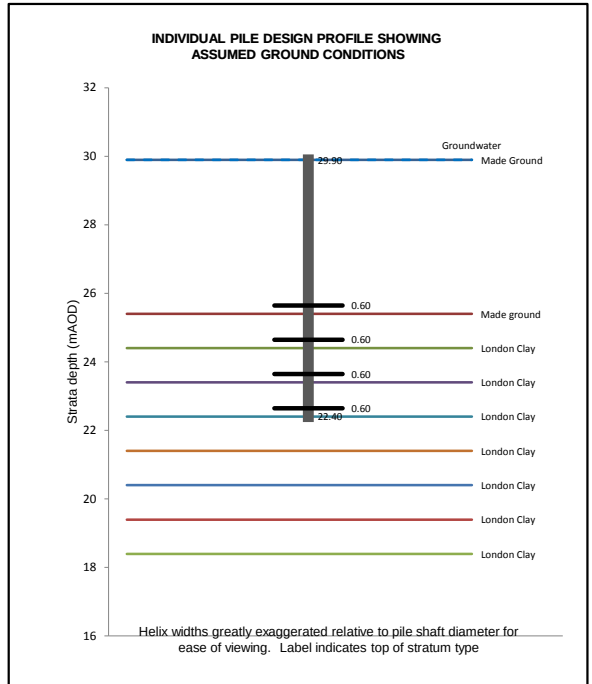
PILE GROUP FOUNDATION DESIGN	
RAKE ANGLE FROM VERTICAL (°)	0
ALTER PILE DESIGN TO ACCOUNT FOR RAKING	NO
MINIMUM PILE SPACING C/C (m)	1.2
PILES PER STRUCTURE	4
TOPOGRAPHY AT SITE	Level
MAXIMUM PILE PROJECTION ABOVE GROUND LEVEL (m)	0.00
BRACING REQUIRED	NO
PILE POSITION SELECTED FROM GROUP FOR DESIGN:	N/A: Level site; all piles in group have identical topographical conditions

PILE DETAILS				
Pile diameter at helix depth(s) (mm)	Wall thickness (mm)	Length (m)	Top (mAOD)	Bottom (mAOD)
168.3	10.00	7.50	29.90	22.40
Surface Area A_s (m ² per m)			0.53	
Helix plate thickness			12mm	

SITE NAME:	Ark Project Union
STRUCTURE TYPE:	Type4
CLIENT REF NUMBER:	

DESIGNER		COMMENTS
NAME:	Giovanni Ferzola	
DATE:	09.12.2020	

REVIEWED & CHECKED		
NAME:	Liam Connor	COMPANY (IF NOT SFF):
DATE:	10.12.2020	
COMMENTS:		



GROUND CONDITIONS USING CHARACTERISTIC VALUES (γ) FROM GROUND TEST RESULTS										
Strata description	Top of layer (mAOD)	Bottom of layer (mAOD)	Cumulative depth (mbgl)	Strata type for purposes of analysis	Unit weight γ (kN/m ³)	Average SPT 'N' value	$c_{u,k}$ (kPa)	Φ'_k (°)	α (see note)	Shaft resistance $R_{s,k}$ (kN)
Made Ground	29.90	25.40	4.50	Made Ground (ignored in analysis)	16					0.00
Made ground	25.40	24.40	5.50	Cohesive	18	18	105		0.72	39.97
London Clay	24.40	23.40	6.50	Cohesive	20	17	100		0.74	39.13
London Clay	23.40	22.40	7.50	Cohesive	20	15	90		0.76	36.17
London Clay	22.40	21.40	8.50	Cohesive	20	17	100		0.74	39.13
London Clay	21.40	20.40	9.50	Cohesive	20	17	103		0.73	39.76
London Clay	20.40	19.40	10.50	Cohesive	20	18	105		0.72	39.97
London Clay	19.40	18.40	11.50	Cohesive	20	18	110		0.71	41.29
London Clay	18.40	16.90	13.00	Cohesive	20	22	130		0.66	68.05
NOTE: SPT 'N' value is used to calculate anticipated torque during installation; it is not used in pile capacity calculation					TOTAL SHAFT RESISTANCE $R_{s,k} = \sum A_s q_{s,k}$ (kN)					343.46
NOTE: Refer to A.3.3.2 and A.3.3.3 if characteristic values of ground strength are derived from static load tests					NOTE: Factored ground conditions for M1 Material Factors are same as $R_{s,k}$ value above since all factors = 1.0					

PARTIAL MATERIAL FACTORS FOR SOIL PARAMETERS (γ_M) (TABLE A.NA.4)					
		M1 SET	M2 SET	Reciprocal of M2 Set	Most onerous factor for M2
Angle of shearing resistance	γ_ϕ	1.00	1.25	0.80	0.80
Effective cohesion	γ_c	1.00	1.25	0.80	0.80
Undrained shear strength	γ_{cu}	1.00	1.40	0.71	0.71
Unconfined strength	γ_{qu}	1.00	1.40	0.71	0.71

NOTE: The value of the partial factor should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value (Note to Table A.NA.4). The characteristic values may have to be adjusted to upper values for the M2 situation as per 7.3.2.1(2)

FACTORED GROUND CONDITIONS FOR M2 MATERIAL FACTORS SET										
Strata description	Top of layer (mAOD)	Bottom of layer (mAOD)	Cumulative depth (mbgl)	Strata type for purposes of analysis	Unit weight γ (kN/m ³)	Average 'N' value	$c_{u,d}$ (kPa)	Φ'_d (°)	α (see note)	Shaft resistance $R_{s,d}$ (kN)
Made Ground	29.90	25.40	4.50	Made Ground (ignored in analysis)	16					0.0
Made ground	25.40	24.40	5.50	Cohesive	18	18	75.0		0.7	28.6
London Clay	24.40	23.40	6.50	Cohesive	20	17	71.4		0.7	27.9
London Clay	23.40	22.40	7.50	Cohesive	20	15	64.3		0.8	25.8
London Clay	22.40	21.40	8.50	Cohesive	20	17	71.4		0.7	27.9
London Clay	21.40	20.40	9.50	Cohesive	20	17	73.6		0.7	28.4
London Clay	20.40	19.40	10.50	Cohesive	20	18	75.0		0.7	28.6
London Clay	19.40	18.40	11.50	Cohesive	20	18	78.6		0.7	29.5
London Clay	18.40	16.90	13.00	Cohesive	20	22	92.9		0.7	48.6
TOTAL SHAFT RESISTANCE $R_{s,d} = \sum A_s q_{s,d}$ (kN)										245.3

BASE RESISTANCE FOR M1 MATERIAL FACTORS SET											
Helix Plate Diameter (m)	Plate Area A _b (m²)	Helix Depth (m below top of pile)	Helix Depth z _h (mAOD)	Resistance in compression (kN)			Resistance in tension (kN)			Plate base resistance in compression r _{b,k} (kN)	Plate base resistance in tension r _{b,k} (kN)
				at z _h	at z _h -0.25m	at z _h -0.75m	at z _h	at z _h +0.25m	at z _h +0.75m		
0.60	0.26	4.25	25.65	0.0	0.0	246.2	0.0	0.0	0.0	49.2	0.0
0.60	0.26	5.25	24.65	246.2	246.2	234.4	246.2	246.2	0.0	243.8	196.9
0.60	0.26	6.25	23.65	234.4	234.4	211.0	234.4	234.4	246.2	229.8	236.8
0.60	0.26	7.25	22.65	211.0	211.0	234.4	211.0	211.0	234.4	215.7	215.7
NOTE: A _b = A _{helix} - A _{shaft} . For compression r _{b,k} = (0.5.f _{b,k} at z _h)+(0.3.f _{b,k} at z _h -0.25m)+(0.2.f _{b,k} at z _h -0.75) For tension r _{b,k} = (0.5.f _{b,k} at z _h)+(0.3.f _{b,k} at z _h +0.25m)+(0.2.f _{b,k} at z _h +0.75)							BASE RESISTANCE Σr _{b,k} = R _{b,k} (kN):			738.5	649.4

BASE RESISTANCE FOR M2 MATERIAL FACTORS SET											
Helix Plate Diameter (m)	Plate Area A _b (m²)	Helix Depth (m below top of pile)	Helix Depth z _h (mAOD)	Resistance in compression (kN)			Resistance in tension (kN)			Plate base resistance in compression r _{b,k} (kN)	Plate base resistance in tension r _{b,k} (kN)
				at z _h	at z _h -0.25m	at z _h -0.75m	at z _h	at z _h +0.25m	at z _h +0.75m		
0.60	0.26	4.25	25.65	0.0	0.0	175.8	0.0	0.0	0.0	35.2	0.0
0.60	0.26	5.25	24.65	175.8	175.8	167.5	175.8	175.8	0.0	174.2	140.7
0.60	0.26	6.25	23.65	167.5	167.5	150.7	167.5	167.5	175.8	164.1	169.1
0.60	0.26	7.25	22.65	150.7	150.7	167.5	150.7	150.7	167.5	154.1	154.1
NOTE: A _b = A _{helix} - A _{shaft} . For compression r _{b,k} = (0.5.f _{b,k} at z _h)+(0.3.f _{b,k} at z _h -0.25m)+(0.2.f _{b,k} at z _h -0.75) For tension r _{b,k} = (0.5.f _{b,k} at z _h)+(0.3.f _{b,k} at z _h +0.25m)+(0.2.f _{b,k} at z _h +0.75)							BASE RESISTANCE Σr _{b,k} = R _{b,k} (kN):			527.5	463.9

NOTE: CALCULATION OF PILE RESISTANCE:

Pile capacity (resistance) is calculated as the sum of the helix resistances, assuming each acts like an end-bearing face. The helix plates are spaced far enough apart along the shaft to ensure that the effect of interference between the bulbs of loaded soil can be ignored in accordance with Boussinesq's theory. The skin friction over the embedded length of the pile shaft is considered negligible and is ignored for the purposes of pile capacity.

All input values are based on measured, site specific values where such information is available. Otherwise the input data is estimated using values taken from published literature.

Minimum helix spacing (m):	1
Distance between lowest helix and end of pile (m)	0.25

PARTIAL RESISTANCE FACTORS (γ_R) FOR DRIVEN PILES FOR THE GEO LIMIT STATE (TABLE A.NA.6)						
	R1	R4 without..	R4 with..	DO YOU HAVE "EXPLICIT VERIFICATION OF SLS" FOR R4?	NO	APPLICABLE R4 FACTORS
SHAFT IN COMPRESSION	$\gamma_{s,c}$	1.0	1.5	1.3	NOTE: The lower values in R4 may be adopted (a) if serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the construction piles to loads not less than 1.5 times the representative load for which they are designed, or (b) if settlement is explicitly predicted by a means no less reliable than in (a), or (c) if settlement at the serviceability limit state is of no concern. (Note A to Table A.NA.6)	1.5
SHAFT IN TENSION	$\gamma_{s,t}$	1.0	2.0	1.7		2.0
BASE	γ_b	1.0	1.7	1.5		1.7
BASE IN TENSION	$\gamma_{b,t}$	1.3	2.2	2.0	NOTE: EC7 does not have a $\gamma_{R,t}$ factor since the base resistance does not contribute to tensile resistance in 'normal' pile design. The $\gamma_{R,t}$ factor is therefore a ScrewFast addition to account for the fact that the soil will provide less resistance to a tensile load on the helix plates than to the same load in compression.	2.2
MODEL FACTOR	γ_{Rd}	1.4	NOTE: The value of the model factor should be 1.4, except that it may be reduced to 1.2 if the resistance is verified by a maintained load test taken to the calculated, unfactored ultimate resistance. (A.3.3.2)			

PILE DESIGN ACTIONS (kN)									
CHARACTERISTIC DESIGN ACTIONS (kN) Permanent = G _k , Variable = Q _k			FAVOURABILITY i.e. promotes stability (favourable) or instability (unfavourable)	A1 SET			A2 SET		
				PARTIAL FACTOR γ _G or γ _Q	FACTORED ACTIONS G _k ·γ _G & Q _k ·γ _Q	DESIGN ACTIONS F _{ci,d} = G _k ·γ _G + Q _k ·γ _Q	PARTIAL FACTOR γ _G or γ _Q	FACTORED ACTIONS G _k ·γ _G & Q _k ·γ _Q	DESIGN ACTIONS F _{ci,d} = G _k ·γ _G + Q _k ·γ _Q
COMPRESSION:	PERMANENT	120.0	UNFAVOURABLE	1.35	162.0	267.0	1.00	120.0	211.0
	VARIABLE	70.0	UNFAVOURABLE	1.50	105.0		1.30	91.0	
TENSION:	PERMANENT		UNFAVOURABLE	1.35	0.0	90.0	1.00	0.0	78.0
	VARIABLE	60.0	UNFAVOURABLE	1.50	90.0		1.30	78.0	
NOTE: Refer to drawing referenced above for pile design load derivation and pile key plan. Partial factors from BS EN 1990:2002 as referenced in Table A.NA.3. Refer to A.NA.3 if actions relate to water level as factors may not be appropriate.									

DESIGN APPROACH 1, COMBINATION 1: A1 + M1 + R1				
	A1 F _d = G _k ·γ _G + Q _k ·γ _Q	M1 R _{b,k}	R1 R _{ci,d} = R _{b,k} ·k ^{1/3} ·l ^{1/3} /r _{sd}	DESIGN CHECK F _d ≤ R _d
COMPRESSION	267.0	738.5	527.5	Difference = 260.5 DESIGN OK
TENSION	90.0	649.4	356.8	Difference = 266.8 DESIGN OK

The critical design case is Design Approach 1, Combination 2
A2 + M2 + R4

DESIGN APPROACH 1, COMBINATION 2: A2 + (M1 or M2) + R4							
	F _d = G _k ·γ _G + Q _k ·γ _Q	R _{b,k}		R _d = R _{b,k} ·k ^{1/3} ·l ^{1/3} /r _{sd}		DESIGN CHECK FOR M1 F _d ≤ R _d	DESIGN CHECK FOR M2 F _d ≤ R _d
	A2	M1	M2	R4 for M1	R4 for M2		
COMPRESSION	211.0	738.5	527.5	310.3	221.6	Difference = 99.3 DESIGN OK	Difference = 10.6 DESIGN OK
TENSION	78.0	649.4	463.9	209.9	149.9	Difference = 131.9 DESIGN OK	Difference = 71.9 DESIGN OK
NOTE: The M2 situation covers unfavourable actions on piles e.g. negative skin friction. 7.3.2.1(2) states that: "for these situations, the design values of the strength and stiffness of the moving ground should usually be upper values".							

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Notes

General Data

Number of increments = 1
Increment applied loads only

Convergence Control

Maximum number of iterations = 300
Maximum displacement error [mm] = 0.0010000
Maximum pressure error [kN/m²] = 0.10000
Damping coefficient = 1.0000
Maximum incremental deflection [m] = 2.0000

Soil Data

Soil model : Generated P-Y curves for static loads

No.	Level [m]	Type	Unit wt [kN/m ³]	E50	C _u top [kN/m ²]	dC _u /dz [kN/m ² /m]	K ₀	K ₁ [kN/m ² /m]	Ø [°]	q _{ur} [kPa]	~r	k _{rm}	E _{ir} [kPa]
1	0.050000	Soft Clay	17.000	0.020000	40.000	0.0	-	-	-	-	-	-	-
2	-4.500000	Soft Clay	17.000	0.010000	105.00	0.0	-	-	-	-	-	-	-
3	-5.500000	Stiff Clay	18.000	0.010000	100.00	0.0	-	-	-	-	-	-	-
4	-6.500000	Stiff Clay	18.000	0.010000	90.000	0.0	-	-	-	-	-	-	-
5	-7.500000	Stiff Clay	18.000	0.010000	100.00	0.0	-	-	-	-	-	-	-
6	-8.500000	Stiff Clay	19.000	0.010000	103.00	0.0	-	-	-	-	-	-	-

Sections

Name	Input Type	Description	Material	Class	Effective Width [m]	EI [kNm ²]
B2M (192.9 X 9.6)	Wizard Generated	STD%CHS%192.9%9.6	Steel	S355*	0.19290	4656.3
B2M (218.3 X 9.6)	Wizard Generated	STD%CHS%218.3%9.6	Steel	S355*	0.21830	6868.3
140x10	Wizard Generated	STD%CHS%138%9.1	Steel	S355*	0.13800	1538.3
170x10	Wizard Generated	STD%CHS%168%9.1	Steel	S355*	0.16800	2876.9

Pile Properties

Level [m]	Section
0.0	170x10
-2.0000	140x10

Pile base at -6.000000 m

Applied Loads and Displacements

No.	Level [m]	Force [kN]	Moment [kNm]	Displacement [mm]
1	0.0	25.000	0.0	0.0

Restraints

No.	Node	Lateral Stiffness	Rotational Stiffness
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1 1 [kN/m] [kNm/rad]
0.0 100000.

Geometry and Initial state

Node	Level	Soil	EI	Effective Width	Water Pressure	Soil Disp
	[m]		[kNm ²]	[m]	[kN/m ²]	[mm]
1	0.0	1	2876.9	0.16800	0.0	0.0
2	-1.0000	1	2876.9	0.16800	0.0	0.0
3	-2.0000	1	1538.3	0.13800	5.3000	0.0
4	-3.0000	1	1538.3	0.13800	15.300	0.0
5	-4.0000	1	1538.3	0.13800	25.300	0.0
6	-5.0000	2	1538.3	0.13800	35.300	0.0
7	-6.0000	3	4656.3	0.19290	45.300	0.0

CALCULATED Soil P-y curves

Node	P1	Y1	P2	Y2	P3	Y3
	[kN/m]	[mm]	[kN/m]	[mm]	[kN/m]	[mm]
1	0.0	0.0	7.5656	1.6800	12.937	8.4000
2	0.0	0.0	12.578	1.6800	21.508	8.4000
3	0.0	0.0	14.527	1.3800	24.840	6.9000
4	0.0	0.0	14.527	1.3800	24.840	6.9000
5	0.0	0.0	14.527	1.3800	24.840	6.9000
6	0.0	0.0	38.132	0.69000	65.205	3.4500
7	0.0	0.0	50.764	0.96450	86.805	4.8225

Node	P4	Y4	P5	Y5	P6	Y6
	[kN/m]	[mm]	[kN/m]	[mm]	[kN/m]	[mm]
1	18.629	25.200	25.874	67.200	25.874	420.00
2	30.972	25.200	43.016	67.200	43.016	420.00
3	35.770	20.700	49.680	55.200	49.680	345.00
4	35.770	20.700	49.680	55.200	49.680	345.00
5	35.770	20.700	49.680	55.200	49.680	345.00
6	93.895	10.350	130.41	27.600	130.41	345.00
7	125.00	14.467	173.61	38.580	173.61	482.25

Output for load increment 1

Iteration	Max at	Disp	Pressure
	Inc node	error	error
		[mm]	[kN/m ²]
19	5.96 1	0.0007	0.02

Node	Level	Defl	Rotation	Soil	Pressure	Bending	Shear
	[m]	[mm]	[rad]		[kN/m ²]	[kNm]	[kN]
1	0.0	-5.9641	-186.65E-6	1	-65.416	0.0	0.0 P
1	0.0					-18.665	-25.000
2	-1.0000	-3.6633	-0.0032849	1	-90.557	0.83967	-11.898 P
3	-2.0000	-0.77247	-0.0022484	1	-58.923	5.1311	-0.22580
4	-3.0000	0.22655	-166.10E-6	1	17.281	1.2913	2.6475
5	-4.0000	0.13436	192.35E-6	1	10.249	-0.16381	0.74789
6	-5.0000	0.0048596	61.767E-6	2	1.9461	-0.20451	-0.093579
7	-6.0000	-0.0084907	-10.858E-6	3	-2.3167	0.023346	-0.0044162

- The letter "P" next to a result indicates that the effective earth pressure is greater than 0.99 times the passive limit, but within the convergence pressure limit.

EXTREME values so far:-

Deflections		Rotations		Moments		Shears	
Min	Max	Min	Max	Min	Max	Min	Max
[mm]	[mm]	[rad]	[rad]	[kNm]	[kNm]	[kN]	[kN]
-5.9641	0.22655	-0.0032849	192.35E-6	-18.665	5.1311	-25.000	2.6475

RESTRAINT FORCES

No.	Node	Lateral force	Moment
		[kN]	[kNm]
1	1	0.0	18.665