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ASSESSMENT OF A BRIDGE PIER PILE FOUNDATION SUBJECTED TO BEARING REPLACEMENT

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SUMMARY: This paper reports on the geotechnical aspects of Highway maintenance work, which has been planned recently for A14 Orwell Bridge, Pier 5. The pier is (one of nineteen) supported by a group of 50 bored piles founded in Chalk. Because the bearings will be replaced for one side of the pier resulting in additional forces from a temporary supporting system erected on one side of the pile cap, a geotechnical assessment of the pier foundation has been required. The investigations has included the back analysis of a pile load test using PLAXIS to evaluate the engineering parameters of the Chalk and the pile bearing capacity calculations, and hence helped to reduce the uncertainty usually encountered in the estimation of skin friction in Chalk. To check for any potential for overstressing or excessive rotation, the load deformation response of the pile group under the new working load conditions was analysed using two different softwares: PIGLET and REPUTE. In general, the paper provides simple procedures to assess an existing pile group in Chalk. Keywords: bearing replacement, bridge pier piles, chalk, highway bridges, deep foundation, Orwell Bridge, piglet, piling in chalk, Plaxis software, Repute software.

INTRODUCTION

Bridges in the UK are routinely maintained by replacing their roller bearings when they have reached the end of their serviceable life. A14 Orwell Bridge has been subjected to this type of maintenance by the Highways Agency. The bridge, shown in Figure 1, carries the A14 over the River Orwell just south of Ipswich in Suffolk, England. The construction of the bridge commenced in October 1979 and was opened to road traffic in 1982. The bridge is 1.3 km long, 24 m wide, 18 span, continuous post-tensioned concrete twin box-section structure and supported on 19 piers¹. Defects were initially observed on roller bearings during an inspection in 1992, resulting in them being replaced. A remote monitoring system was set up in 2003 to detect any excessive movement and bearing deterioration at the bridge. Further maintenance work is being planned at Pier 5; the pier (indicated by arrow in Figure 1-(i)) comprises two columns (north and south), which share piled foundation. Bearing replacement will be required only at the north column of

Pier 5. A temporary steel frame is to be erected on the pile cap to support the bridge loads of the North deck while replacing the bearings². Although the additional load is estimated to be less than 15% of the total existing loads applied on the foundation, the distribution being on one side of the pile cap may significantly increase the existing stresses under this side.

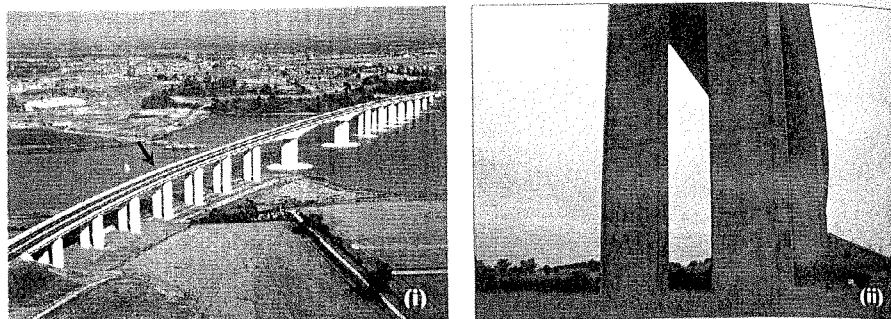


Fig. 1: (i) Overview of Orwell Bridge, Pier 5 is indicated by an arrow. (ii) Photo taken at Pier 5, where the height of the pier from top of pile cap is 34.5 m.

To ensure the safety and serviceability of the foundation under the new temporary supporting system (i.e. during the proposed bearing replacement), a geotechnical assessment is required to predict any potential over-stressing or excessive settlement under the piles. In the initial stage of the assessment, the existing geotechnical information was collected from the as-built records¹ and reviewed on the basis of the knowledge and experience reported in literature.

The ground investigations conducted for the bridge have shown that the site comprises various superficial deposits underlain by an extensive layer of Upper Chalk. The properties of these materials, used for the existing pile design, were evaluated against a historical pile load test, which was back analysed using PLAXIS-2D³. The analysis of the pile load test also provided a tool to evaluate the bearing capacity estimated for a single pile and particularly in reducing the uncertainty in estimating the skin friction of the chalk.

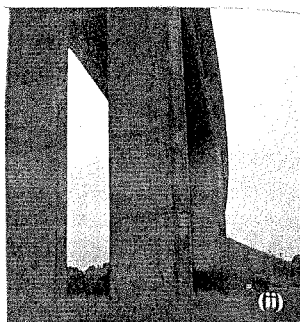
To calculate the deformations and load distribution among the piles in the group, under the 3-dimensional working load conditions, the load deformation response of the pile group was analysed. The pile group was modelled first using PIGLET software⁴ adopting a linear elastic model for the ground. To ensure that the elastic assumption was reasonable at the load and deformation levels, a further check using REPUTE software⁵ was performed.

METHODOLOGY

The method, carried out to assess the effects on the foundation system under the temporary loading system (produced by the proposed bridge maintenance e.g. steel work), included a number of steps, which can be summarised as follow:

- Desk study exercise to collect and review the existing geological and geotechnical data for the site.
- Identify the pile group arrangement at Pier 5 from as-built records¹
- Evaluate the material properties used for the assessment by conducting back analysis on a historical pile load test using finite element method in PLAXIS.

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(ii) Photo taken at Pier 5, where

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- Estimate the bearing capacity and settlement of the piles
- Compare the outputs from above with that given in the 'as built record' and identify any significant difference
- Conduct structural/ matrix analysis of the pile group subjected to the new loading system. This part of the assessment was achieved using two different commercially available softwares. PIGLET was used to check the elastic response of the system. Then, the effect of plasticity of the ground materials on the analysis was checked by REPUTE software.
- The axial forces in piles obtained from above were compared with the allowable bearing capacity of the piles. It is believed that by this step any critical condition that may be induced by the proposed temporary supports on the foundation can be identified.

EXISTING INFORMATION

Geology of the site

The subsoil information used to establish the ground model was obtained from various sources, including:

- Ground investigation data used for the construction of Ipswich Bypass and Ipswich Southern Bypass Orwell Bridge reported in 1976 and 1979 by Sir William Halcrow & Partner ^{6,7}
- As-built construction drawings and documents (1997–1984) ¹
- Recent site investigation conducted for Pier 9 remediation ⁸
- In addition to the British Geological Survey 1:50,000 Series, Sheet 207 Solid and Drift for Ipswich ⁹ and British Regional Geology – East Anglia and adjoining areas ¹⁰

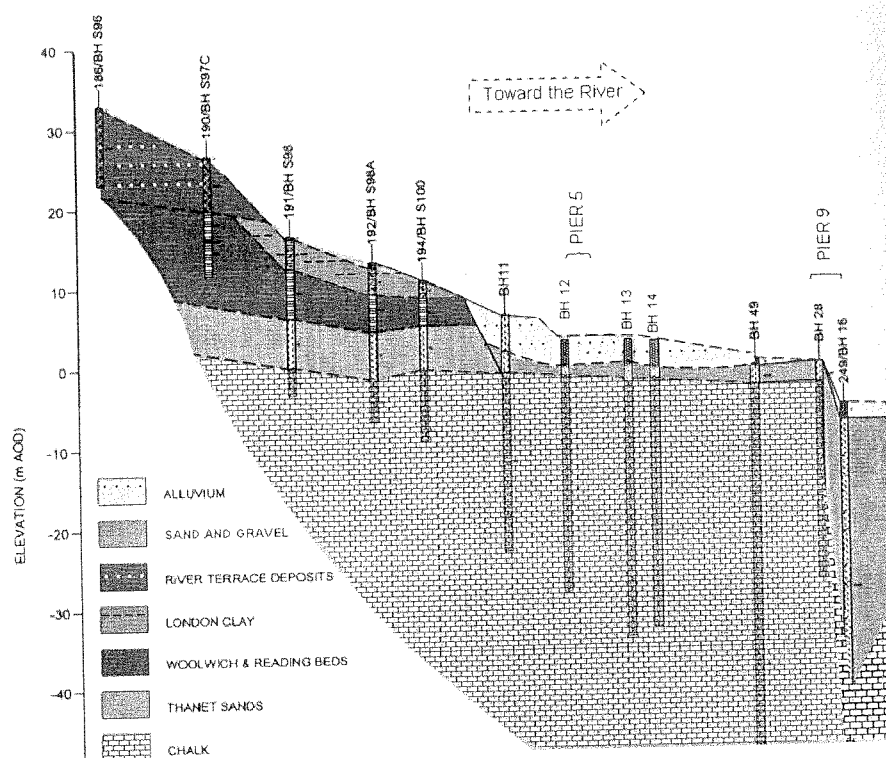
The information provided by the existing ground investigation was found generally adequate to establish the subsurface conditions for the purpose of the proposed assessment. A walkover survey and a number of trial pits ¹¹ were conducted to confirm the pile cap size and provide a schedule of ground parameters that may be required to design a temporary access road.

From the available data of the area, it appears that the geology of the broader area has the arrangement of a typical river valley as shown in Figure 2, where a geological cross section has been plotted from a number of historical boreholes on the west side of the river. The geological sequence is the same on both sides of the valley and consists of a sedimentary succession of River Terrace Deposits, London Clay, Lambeth Group and Thanet Sands ¹². The whole sequence has been deposited on Upper Chalk and eroded from the centre of the valley where the river currently flows and now comprises Alluvium overlying glacial granular soil underlain by the Chalk.

In the immediate vicinity of Pier 5, the ground investigation data has shown that 0.3 m of Topsoil is underlain by about 1.5 m of granular Made Ground, which has been used to bring the soil to 0.4 m above the pile cap level. Below the Made Ground, the natural geological sequence comprises Alluvium deposits, Glacial Sands and Gravels overlying the Upper Chalk. The ground summary at Pier 5 including the description of the geological units, their average thicknesses are given in Table 1.

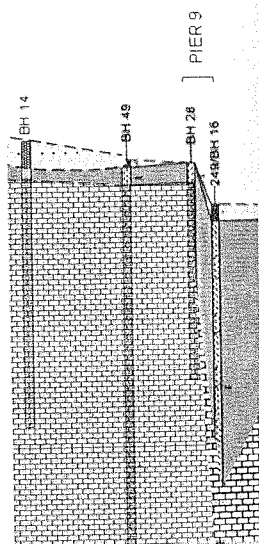
Table 1. Geological profile at Pier 5 of Orwell Bridge

Unit	Description	Thickness of the Unit (m)
Topsoil	Soft to firm fine grained sandy gravelly clay with occasional soft organic material	0.3
Made Ground	Orange brown gravelly, silty sand with occasional pockets of stiff sandy clay	1.2
Alluvium	Soft silty sandy clay	1.60
Sands and Gravels	Generally medium dense sand or silty gravelly Sand with occasional pockets of stiff clay.	0.5
Chalk	Grade V (top 15m), and Grade IV/ III	30+

Fig. 2: Geological section of the west side of the Bridge constructed from historical boreholes³.

The Chalk is described as highly weathered Grade V at the top 15 metres and becoming Grade IV/III at deeper locations. According to the classification of Chalk proposed by Ward, Burland and Gallois (1968)¹³, Grade V is used to describe 'Structureless remoulded chalk containing lumps of intact Chalk', while Grade III is used for 'Rubbly to blocky unweathered Chalk'. Further description of Grades and Classes of Chalks is

	Thickness of the Unit (m)
l	0.3
f	1.2
	1.60
	0.5
	30+

historical boreholes⁵.

presented in CIRIA report C574¹⁴. Although the Chalk formation is known in other regions to be subjected to dissolution features, there is no evidence at the surface of such features within the study area, nor from the ground investigation.

No groundwater was encountered at shallow depths in the recent trial pits conducted in the vicinity of the scheme area, but the Chalk is considered as a Major Aquifer¹⁵ and the presence of groundwater in the Chalk has been confirmed in the piling record of the bridge foundations¹. Therefore, for the purpose of this assessment, the phreatic surface was considered at the top of the Chalk.

Ground condition and material properties

Chalk can be considered the main geological unit providing the bearing stratum required for the deep foundation system. As-built records^{6,7} have shown an extensive investigation conducted on this stratum. The investigation comprised various common field and laboratory tests to determine the material properties. A summary of the test results is presented in Table 2. Moreover, Cone Penetration Testing has been recently conducted for remedial works⁸ at Pier 9 adding further confidence in the Chalk properties. Based on these ground investigations, interpretation has been carried out to obtain the design parameters, of which a summary is presented in Table 3. Details on this interpretation have been discussed elsewhere^{6,7,12}.

The engineering parameters recommended in 1976 by Sir William Halcrow & Partners⁶ are the lowest value of stiffness, which has been adjusted later by the same Author⁷ who suggested higher values after conducting supplementary site investigation and general trial pile tests in the scheme area. The highest values¹² has been based on CPT (conducted by the river), where the effective Young's modulus E has been determined from relationships suggested by Meigh¹⁶ and Lord¹⁴.

Table 2. Field and laboratory testing results conducted on the Chalk^{6,7}.

Properties			Number of tests	Range (average)
SPT (results shown from borehole adjacent to Pier 5)	Elevation	0 to -15 m AOD	10	3-18 (9)
		-15 to -20	3	15-25 (20)
		-20 to -23	2	25-33 (29)
		-23 to -30	5	25-40 (33)
Triaxial tests on intact samples	Drained condition	Effective Shear strength, c' , (kPa)	10 on Soft Chalk* 5 on Hard Chalk*	60* 320*
		Effective friction angle, ϕ' (Deg)		33* 39*
	Undrained	C_u , (kPa)	20 on Soft Chalk	60-260
Drained Triaxial tests on remoulded samples		Effective friction angle, ϕ' (Deg) ($c'=0$)	8	39
Deformation Modulus, E' , MPa		In-situ Pressuremeter test	13	10-350
		Laboratory test	37	10-190

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Table 3. Summary of the engineering parameters of the Chalk^{6,7,12}.

Reference	Bulk Unit Weight γ (kN/m ³)	Cu (kPa)	ϕ' (°)	c' (kPa)	v'	E' (MPa)
Ref. 12	20	-	33	0	-	50 (to 2m brh**) 100 (2m to 13m) 175 (below 13m)
Ref. 7	18.5 - 20	60*-260	33*- 39	60* - 320	0.25	40 (to 10m) 80 (10m to 20m) 160 (below 20m)
Ref. 6	18.5 - 20	125*-800	39	320	0.30	17.5 (to 10m) 43 (10m to 20m) 121 (below 20m)

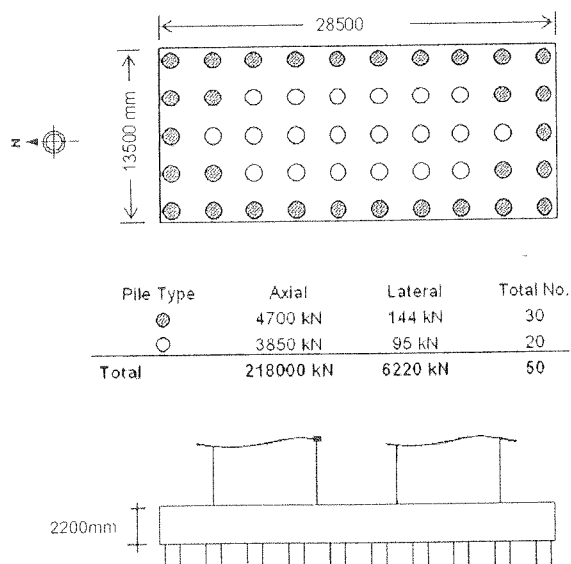
* Increases with depth ** brh: below rock head

Arrangement of the foundation system

As-built construction drawings¹ have shown that the foundation system at Pier 5 contains 50 bored piles (10 × 5), with 3 m centre to centre spacing and 1.05 m in diameter. The pile cap is 28.5 m long and 13.5 m wide as shown in Figure 3. Two types of piles were identified including:

- Type B: 20 pile distributed around the edge of the pile group, and
- Type C: 30 pile in the internal area.

The two pile types have identical geometries and both are 24 m long and reach -23 m AOD, but different allowable working load and steel reinforcement. The difference in pile capacity may be attributed to group efficiency, where an outer pile with fewer neighbouring piles will perform better¹⁷.

Fig. 3: Schematic diagram showing the pile group arrangement at Pier 5¹

a)	ν'	E' (MPa)
	-	50 (to 2m brh**) 100 (2m to 13m) 175 (below 13m)
20	0.25	40 (to 10m) 80 (10m to 20m) 160 (below 20m)
	0.30	17.5 (to 10m) 43 (10m to 20m) 121 (below 20m)

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Pile load test

A static load test was conducted on an internal working pile of Pier 5 during the construction¹ (1980), where the axial load was applied to the top of the test pile with the use of hydraulic jacks. The reaction force was provided by two adjacent anchor tension piles. The test was carried out for three days, where the applied load increased progressively over this period to a maximum value of 5787 kN $\approx$ 150% of the working design load for a single internal pile (3850 kN). The loading produced a maximum settlement of only 3.6 mm at the pile head level, with the load maintained for 12 hours.

The pile test is a valuable tool to understand the settlement characteristics and performance of the pile when under load. Therefore, the test was back analysed to evaluate the material properties and the estimated bearing capacity.

EVALUATION OF MATERIAL PROPERTIES AND PILE CAPACITY

Evaluation of material properties

Before conducting the pile group analysis, it was important to evaluate the properties of the Chalk. For this purpose, the historical pile load test was back analysed by finite element analysis using PLAXIS. In particular, 2-D axisymmetric model of a single pile embedded in the geological sequence explained above was constructed and based on Mohr-Coulomb constitutive model.

Using the range of strength and stiffness parameters suggested in Table 3, the model predicted (in average) a settlement of the pile head approximately 3.3 times the value reported in the actual test. The model was initially analyzed for drained condition because the permeability of the Chalk is relatively large compared with the actual loading time in the test (3 days). The permeability of Chalk in the field is normally high enough to be considered free-draining¹⁴, however, at critical state the conceptual models for the shaft resistance in Chalk explained by Burland¹⁸, has suggested that a partial drained condition develops due to a thin layer of remoulded Chalk formed around the shaft during boring.

The sensitivity of the model to the undrained condition was investigated; the analysis showed a better result, with a smaller gap between the actual and the predicted settlement, but still overestimated by 2 times, indicating that the Chalk parameters used in the model (shown in Table 3) are conservative, particularly the stiffness values considered for the Chalk at and below the pile toe level, where Grade III and IV has been found. Compared to Matthews scale¹⁹, these grades fall into Class B- a medium density- with secant modulus usually ranging from 1500 to 2000 MN/m² for stresses up to 500 kPa; after this stress level the stiffness decreases²⁰ to 50-80 MN/m². This proposed stiffness is up to 6 times larger than the values used for the Chalk adjacent to the pile toe level (i.e. 20 m below the Chalk head) in the initial PLAXIS model. By incorporating the new higher range of stiffness, the same settlement values reported in the pile test were obtained.

Evaluation of pile axial bearing capacity at Pier 5

The estimation of bearing capacity of bored piles in Chalk is usually based on the SPT N value²¹. For a bored pile with a base area A_b , the ultimate end bearing Q_b may be estimated from:

$$Q_b = 240 N A_b \text{ kN} \quad (\text{for } N < 30) \quad (1)$$

While there has been a general agreement about the calculation of the end bearing of bored piles in Chalk based on Standard Penetration Tests (SPT), the estimation of skin friction (or shaft resistance, Q_s) have had more debate. Two methods of determining the ultimate skin friction in Chalk have been proposed: the first method has been discussed by Hobbs and Healy²¹ based on empirical relationships with SPT blow-count N . However, re-analysis of the case histories used for the first method has indicated that the shaft resistance of bored piles should not be related solely to the SPT N value²⁰. On contrary, the second method has indicated that the skin friction of piles in Chalk is controlled by frictional behaviour and the average effective overburden pressure along the pile shaft^{22,23,24}.

In the current assessment, the ultimate skin friction, Q_s , was calculated using both methods. Thus, a smaller value was obtained using the empirical correlation with SPT. According to the calculations the total ultimate capacity of a single pile ($Q_b + Q_s$) varies from 10,900 and 13,500 kN. Indeed, the difference was primarily caused by the uncertainty of estimating the skin friction, Q_s , where the lower bound value was based on SPT. However, this method has many shortcomings²⁰ including that the test has not been able to distinguish between a medium density Chalk at Mundford and a low density Chalk at Norwich for both of which N is typically 8 to 15. This has been attributed to the nature of the Chalk, where discontinuities in higher density or Grade of Chalk may ease the spoon penetration.

To reduce the uncertainty in the bearing capacity estimation, the PLAXIS model validated by the historical pile load test was rerun to find out the load required to achieve maximum pile head settlement of 10 mm¹⁴, where the base resistance is likely to be mobilised i.e. when shaft resistance has been or is close to being fully mobilised. The result showed that the pile in the model required an axial force of about 14,000 kN to achieve the proposed criteria, which is close enough to the upper bound value estimated by the second method suggesting that this method may provide more economic pile design.

The working load, P_w , may be estimated from the ultimate pile capacities Q_s and Q_b divided by factor of safeties: $F_b = 10$ and $F_s = 1.5$ as suggested by CIRIA Report PG6²¹ for bored cast in place piles with large diameter:

$$P_w = Q_b/F_b + Q_s/F_s \quad (2)$$

The large factor applied to the ultimate end bearing has been chosen to restrict settlement and to guard against risk from solution features immediately beneath a highly stressed pile base. Accordingly, the estimated working load was 5600 kN, which is 20% larger than its allowable capacity (4700 kN) given in the as-built construction records (see Figure 3) implying that the existing pile design is conservative. Nevertheless, the bearing capacity and the engineering parameters (i.e. in Table 3⁷), which have been identified from as-built, are reasonable for the initial assessment of the pile group.

STRUCTURAL ANALYSIS OF PILE GROUP

Bridge load combinations

Details about trestles legs to be used for the steel work and the load breakdown for the geotechnical analysis have been provided by the bridge engineers. This information is presented in Figure 4 and Table 4, where four critical load cases were identified and considered in the assessment herein.

Table 4. Load combinations used for the initial assessment of the pile group

Load case	Fz (vertical), kN	Fx, kN	Fy, kN	Mx, kN.m	My, kN.m
1	82385	685	0	83006	16780
2	81760	0	954	111561	5648
3	82074	343	954	111142	11214
4	72178	1753	1753	126532	65847

In addition to the effect of the eccentricity of the loads applied by the trestle legs (shown in Figure 4) with an additional 800 mm thick concrete slab placed on the north side of the pile cap, the bending moments also include other effects such as wind and jacking forces.

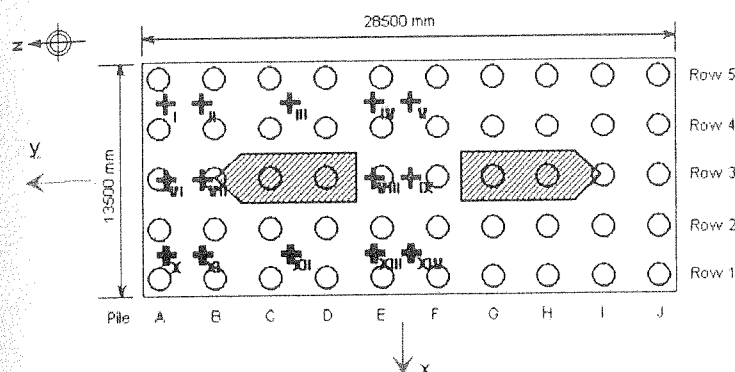


Fig. 4: Plan view of the pile cap showing pile notations and the proposed location of trestle legs (denoted +) of the steel work.

The pile group was initially analysed using PIGLET⁴. The program is Excel based software which is an approximate closed form solution allowing analysis of the elastic response of pile groups under 3D working load conditions. In the analysis, the soil is modelled as a linear elastic material, with stiffness varying linearly with depth. The solution provides stiffness and flexibility matrices for the pile cap, axial, lateral and moment loading at the head of each pile, and profiles of bending moment and lateral deflection down selected piles.

The predicted forces and deflection are dependant on the stiffness of the piles and soil and also on the fixity of the pile head and the flexibility of pile cap. In this analysis, the Young's modulus of the concrete of the piles was taken equal to 15 GPa (suggested by the Bridge engineers based on structural assessment), the piles were assumed fixed to a rigid pile cap, and the loads were applied on the centre of the pile cap. Based on the parameters recommended in Table 3, the axial shear stiffness of the strata was plotted with depth and successfully fitted to a liner profile, starting from a value of 0 MPa at the ground surface and increasing linearly by approximately 2.5 MPa per metre depth.

The deflection and rotation of the pile cap predicted by the software was insignificant. However, the axial load was noted to be relatively large. Figure 5(i) shows the maximum axial loads predicted in all piles by PIGLET, where clearly the piles at the north side are subjected to larger value as a result of the load combinations (shown in Table 4) produced by the proposed supporting system, with greatest load of 4900 kN in the corner pile A5, which has exceeded the as-built allowable axial pile capacity, P_w , by 5% (where $P_w = 4750$ kN).

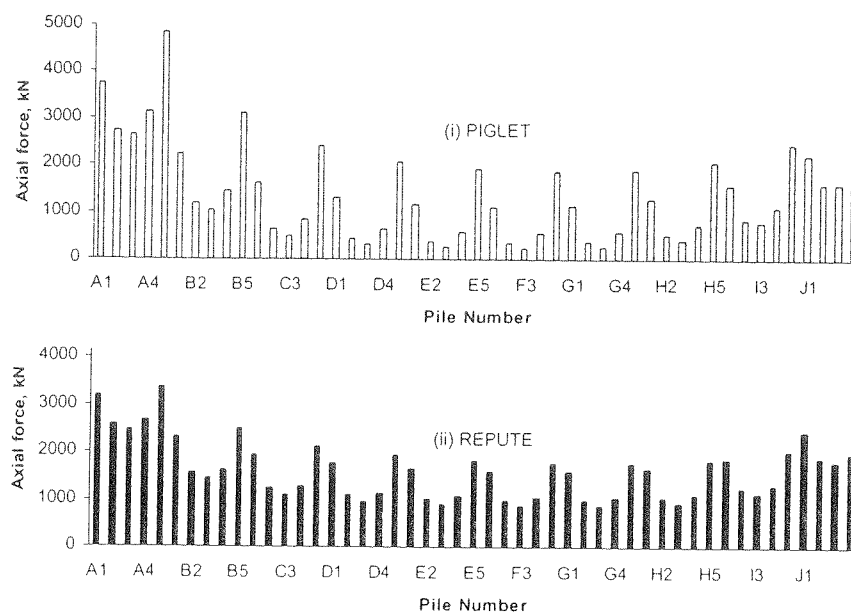


Fig. 5: Maximum axial loads in all piles obtained by: (i) Piglet and (ii) Repute. The pile locations and numbers are shown in Fig. 4. (Due to the large number of piles, the diagrams here do not show all the pile numbers)

Discussion and further analysis

To evaluate the PIGLET analysis results, further assessment was carried out by predicting the response of the pile group by considering a perfect plastic behaviour for the Chalk in addition to the linear elasticity. This was carried out using REPUTE software, which is based on Boundary Element Method⁵, and has the option of using shear strength criteria of Mohr-Coulomb. In particular, the effective friction angle of the Chalk was considered in addition to the initial elastic modulus, as given in Table 3.

The result of the axial loads obtained from REPUTE for each pile is graphically presented in Figure 5(ii), indicating that the maximum vertical load is about 3500 kN, which is almost 30% less than the allowable bearing capacity used for the existing design.

The diagram of the bending moment was also predicted and presented in Figure 6. Comparing the results obtained from both softwares, the linear response predicted by PIGLET has resulted in 53% larger vertical loads at the corner piles; but 44% less maximum bending moment. Nevertheless, the maximum value (predicted by REPUTE) was found to be less than the bending moment capacity of the pile.

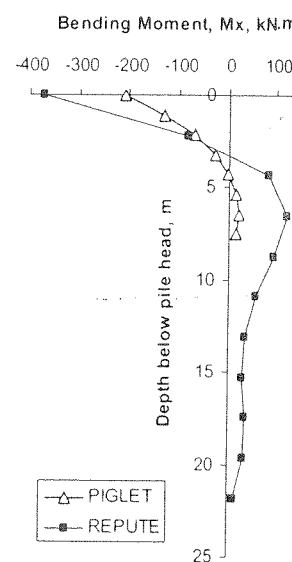


Fig. 6: Diagram of bending moment predicted by PIGLET and REPUTE. (By default, PIGLET values stop when they become insignificant).

The difference in the results obtained from both softwares may be attributed to the plasticity of the materials used in the analysis and also to the method and assumptions embedded in the softwares.

CONCLUSION AND RECOMENDATIONS

An initial geotechnical assessment of a group of bored piles in Chalk has been presented in this paper discussing the stages implemented to ensure that safe working loads are applied during the bearing replacement of the bridge pier and no excessive deflection or rotation will result from the new temporary load distribution.

To establish the ground conditions and determine the engineering parameters of the strata, the existing geotechnical data was collected from as built records, and then evaluated against a historical static load test using PLAXIS software.

The bearing capacity of a single pile was calculated considering two different methods in obtaining the ultimate skin friction, where the larger value based on the effective overburden pressure has been found to be in agreement with the results of the pile load test analysis.

The elastic response of the pile group was analysed using PIGLET software, which has predicted a significant increase in the vertical loads on the edge piles at the North side (where the bridge maintenance work will occur). However, this proved to be an over-estimation compared with the result obtained from REPUTE software where a perfect plastic behaviour of the Chalk was considered.

The analysis of the pile group can be improved by using analytical tools that consider the non-linear mechanical behaviour of the Chalk i.e. adopting strain and stress dependent stiffness. Nevertheless, the analysis carried out was sufficient to satisfy the designers that the proposed method of bearing replacement would not cause an unexpected behaviour to the existing bridge sub or superstructures.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the support of the Highways Agency.

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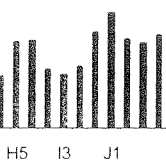
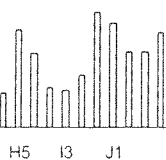


Figure 1. Pile locations and numbers (show all the pile numbers).

Figure 2. Bending Moment, M_x , kN.m

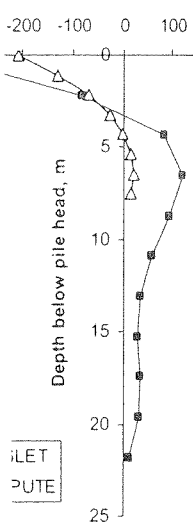


Figure 3. Comparison of bending moment PIGLET and REPUTE. PIGLET values stop when insignificant).

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