

Piled foundation aspects of the bridge slide at Newark Dyke

T.P. Suckling

Cementation Foundations Skanska, Rickmansworth, United Kingdom

J. Lee

Cementation Foundations Skanska, Doncaster, United Kingdom

Abstract

A complex design and construct piling project is described, which formed part of the works for the replacement and upgrade of the East Coast Mainline rail crossing over the River Trent at Newark. Complex pile group analyses using soil elasticity idealisations are described, along with the practical interpretation of the calculated forces taking into account the conflicting requirements of design to BS5400 and the difficult site logistics present on this site. A comparison between a conventional pile group analysis and the results from a new full continuum, multi-layered and non-linear soil model used in the program PGROUPN is presented. The practicalities of constructing CFA piles in a very restricted area in terms of access, headroom and space, and also adjacent to the river, are described.

Introduction

The River Trent crossing of the East Coast Mainline is located approximately 2km north of Newark town centre. The rail track approaches the river on an approximately 4m high embankment from both the south and the north. Up to the year 2000 the tracks were carried across the river by two separate bridges which rested on brick abutments. Around the base of each brick abutment was an apron of concrete retained by sheet piling, forming a level area of generally only 1.0m width.

The existing bridges were replaced during 1999 and 2000 by a single state of the art bow-string bridge. This was constructed adjacent to the existing bridges and then slid into position during a possession of the railway. The new bridge is to cater for an increased train speed of 140 mph, and is thought to be the first such structure within the United Kingdom. Foundations for the new bridge had to be constructed prior to the limited closure of the railway for the bridge slide, and comprised piles for four new bases, several launch tracks, temporary support within the river itself, and for several new overhead electric cable gantries. Foundation works for the four bases were constructed in areas of unrestricted headroom directly adjacent to the live railway and in areas of restricted headroom, limited access and working area beneath the live railway. For clarity, only the new base south-west of the river crossing will be described in detail in this paper.

This paper will firstly describe the design aspects of the piled foundation for the south-west pilecap. This non-symmetrical pile group is subject to a variety of load conditions for both the permanent condition and during the bridge slide. Pile loads calculated by structural methods are compared with those calculated using the PIGLET pile group program. For this paper, design structural pile loads are also compared to the recently developed pile group analysis program PGROUPN. The construction benefits on this site of not designing piles to the requirements of BS5400 Part 4 will also be discussed.

Secondly this paper will describe the challenging construction aspects of the piling works. The success of the Continuous Flight Auger (CFA) piles and the tripod piles will be compared, showing how the construction programme benefited from the use of a project inspired heavily modified low headroom Soilmec CM42 CFA piling rig for constructing the pile groups beneath the existing bridges.

Ground conditions

Reference to the British Geological Survey indicates the site to be underlain by Alluvium over river gravels, which in turn overlie the Mercia Mudstone. This was confirmed by the site investigation of two cable percussion boreholes, one each side of the river, as shown in Figure 1. Also provided were several static cone penetration tests. Reference to published literature confirmed the anticipated ground conditions.¹

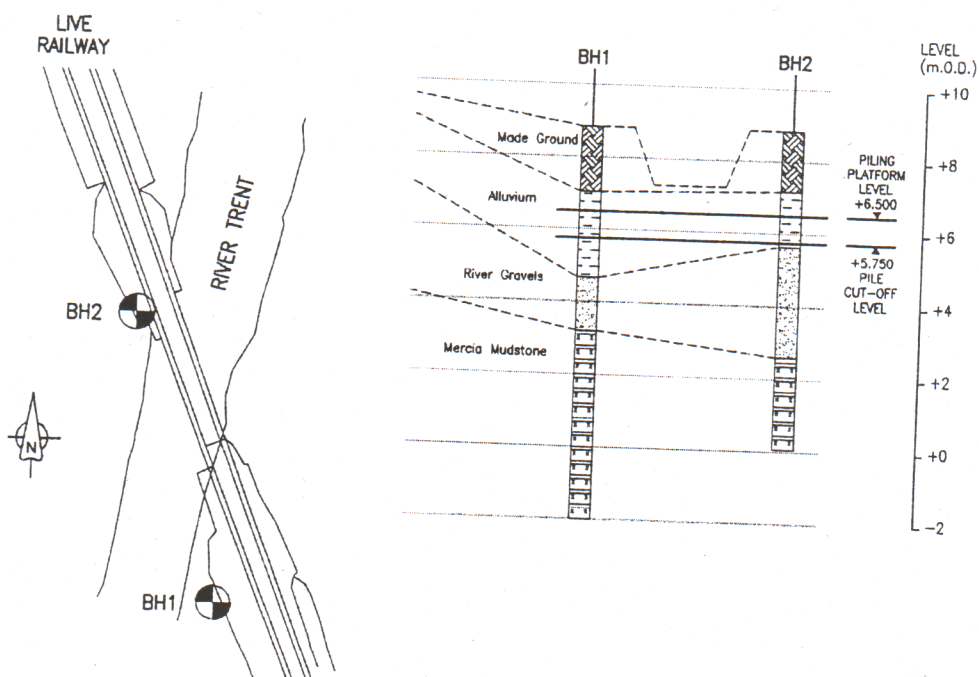


Fig.1 : Borehole location plan and sections

Mercia Mudstone here is predominantly composed of interbedded mudstone and calcareous siltstone. Layers of gypsum are also present. A significantly softened layer of weathered material was encountered to approximately 4m beneath the existing abutments. Standard Penetration Test 'N' values of 150 to 200 for 300mm penetration were recorded below this level.

The selection of geotechnical parameters for design was based upon standard relationships that can be found in normal textbooks and the recommendations of CIRIA Report No. 47 "Further work on the engineering properties of Keuper Marl" dated October 1973.

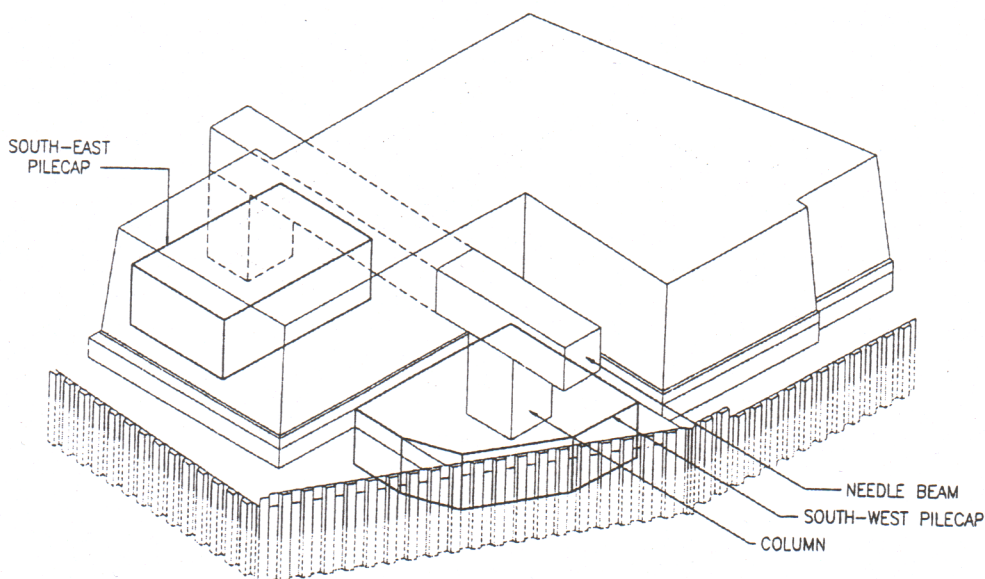


Fig.2 : New substructure at Newark Dyke on southern side of River Trent

Design aspects of the new structure

The design philosophy for the new bridge was required to be in accordance with BS5400. The specified design requirements for the new superstructure were necessary due to the increased weight and dynamic effects from trains running up to 140mph. All loadings from the new main superstructure are to be transferred to new substructures, but with the existing abutments preserved in order to retain the permanent way at the ends of the bridge.

The new substructure at each abutment comprises a needle beam passing transversely through the existing masonry beneath the tracks onto columns and piled foundations at either side, as shown in Figure 2. Construction of these works was arranged so that at no time was support removed from the live railway.

Steel piling was used to afford temporary protection to the pilecap excavations and also permanent scour protection against the river.

Design aspects of the south-west pilecap

The south-west pilecap is presented as an example for the pile design philosophy for this project, and the pile layout is shown in Figure 3. The pile layout was produced resulting from iterative calculations considering the available pilecap geometry, applied loadings and pile capacity.

Three load combinations were specified for the loads applied to the pilecaps, as follows;

- ◆ Load combination 1 – dead load + live load + braking / nosing load
- ◆ Load combination 2 – dead load + live load + braking / nosing load + wind load
- ◆ Load combination 3 – temporary loads during bridge slide

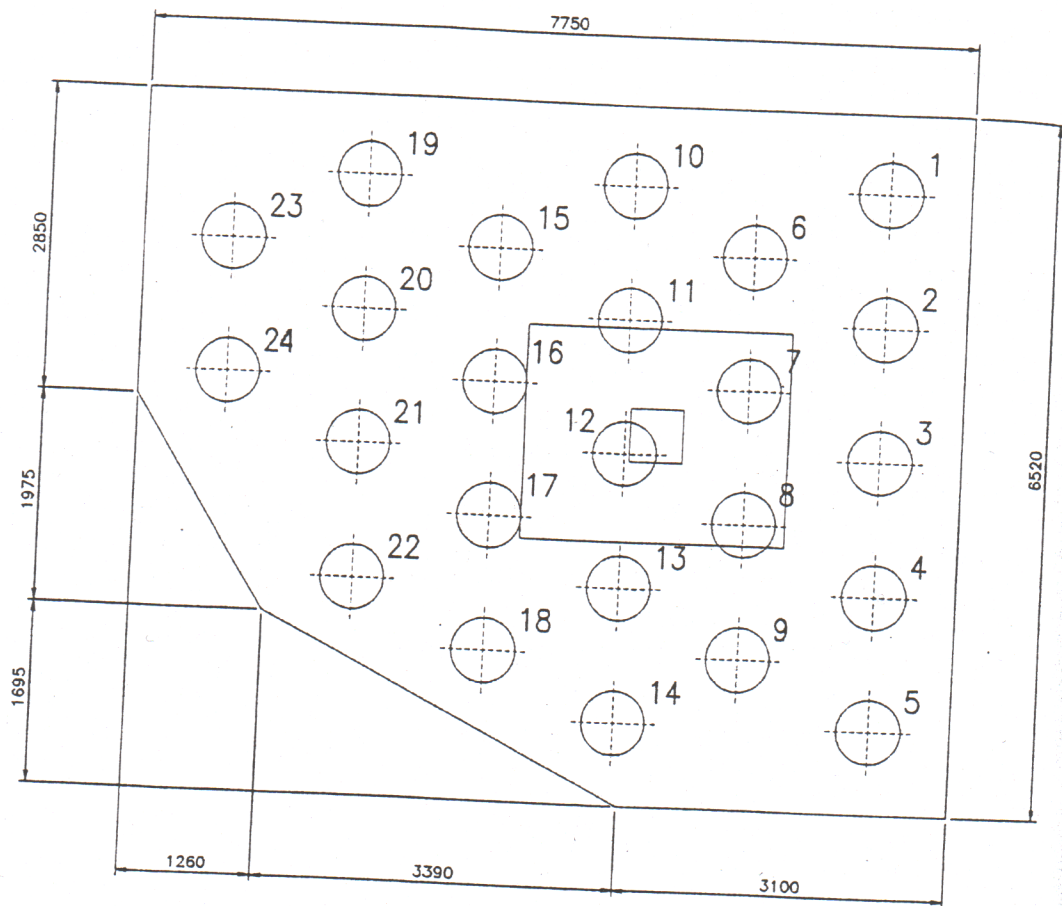


Fig.3 : South-west pilecap

The loads that were specified as applying to this pile group are given in Table 1.

Load Combination	Axial Load (kN)	Horizontal Load in x direction (+/- kN)	Bending Moment in x to z direction (+/- kNm)	Horizontal Load in y direction (+/- kN)	Bending Moment in y to z direction (+/- kNm)	Torque (kNm)
1	19892	200	1278	1290	8243	0
2	19940	450	2876	1425	9106	0
3	13240	1184	7566	984	8288	0

Table 1 : Applied pile group loads

The authors' company routinely uses the computer program PIGLET for the analysis of such pile groups, a program written by Prof. Randolph. This program has been successfully tested against other similar programs such as PGROUP and DEFPIG. PIGLET analyses produce approximate solutions for single piles and then use interaction expressions for groups of piles. These types of program, which model the soil response in terms of elasticity, are notorious for over predicting outer pile loads or loads at sharp changes in pile geometry.

The PIGLET program requires input values of soil shear modulus, G . The following relationships were employed for this project;

- ♦ For fine grained soils $2G_h = G_v = 175 \times C_u$
- ♦ For coarse grained soils $2G_h = G_v = 1000 \times N$

Where G_v = shear modulus in the vertical direction in kN m^{-2}
 G_h = shear modulus in the horizontal direction in kN m^{-2}
 C_u = undrained shear strength in kN m^{-2}
 N = standard penetration test value

A design philosophy for pile axial loading in these types of analyses is to provide a conventional BS8004 structural and geotechnical factor of safety on the average pile load in each row of piles. However, the analysis also has to ensure that a structural and geotechnical factor of safety of at least unity is provided on the average maximum pile load in each row as well. For the Newark Dyke project, with its unusual dynamic load effects, it was decided to provide a factor of safety of 2 on the average load and 1.5 on the maximum load in each row. The philosophy of BS8004, which allows up to 25% overstress due to wind loading is applied. The PIGLET calculated pile loads are shown in Tables 2 to 4.

Load combination 1 (Maximum pile load calculated by structural methods was 1541kN.)

Row No.	Pile Nos	Maximum Pile Load (kN)	Average Maximum Pile Load (kN)
1	1 to 5	2545	1905
2	6 to 9	939	727
3	10 to 14	1253	741
4	15 to 18	996	608
5	19 to 22	1183	869
6	23 to 24	1391	1308
Whole group	1 to 24	2545	1028

Row No.	Pile Nos	Maximum Bending Moment (kNm)	Average Maximum Bending Moment (kNm)
1	1 to 5	164	135
2	6 to 9	80	73
3	10 to 14	133	91
4	15 to 18	105	78
5	19 to 22	134	105
6	23 to 24	134	127
Whole group	1 to 24	164	100

Table 2 : PIGLET results

Load combination 2 (Maximum pile load calculated by structural methods was 1642kN.)

Row No.	Pile Nos	Maximum Pile Load (kN)	Average Maximum Pile Load (kN)
1	1 to 5	2709	1997
2	6 to 9	996	763
3	10 to 14	1360	790
4	15 to 18	1086	648
5	19 to 22	1288	935
6	23 to 24	1484	1386
Whole group	1 to 24	2709	1087

Row No.	Pile Nos	Maximum Bending Moment (kNm)	Average Maximum Bending Moment (kNm)
1	1 to 5	186	152
2	6 to 9	91	80
3	10 to 14	149	101
4	15 to 18	116	85
5	19 to 22	151	117
6	23 to 24	150	141
Whole group	1 to 24	100	111

Table 3 : PIGLET results

Load combination 3 (Maximum pile load calculated by structural methods was 1325kN.)

Row No.	Pile Nos	Maximum Pile Load (kN)	Average Maximum Pile Load (kN)
1	1 to 5	2294	1593
2	6 to 9	844	616
3	10 to 14	1262	686
4	15 to 18	1007	558
5	19 to 22	1173	819
6	23 to 24	1248	1127
Whole group	1 to 24	2294	901

Row No.	Pile Nos	Maximum Bending Moment (kNm)	Average Maximum Bending Moment (kNm)
1	1 to 5	186	142
2	6 to 9	88	69
3	10 to 14	127	83
4	15 to 18	96	68
5	19 to 22	131	93
6	23 to 24	133	123
Whole group	1 to 24	186	95

Table 4 : PIGLET results

For example, considering row No. 1 the following ultimate axial capacity, Q_{ult} , is required for these five piles;

- ◆ Load combination 1 – $Q_{ult} = 1.5 \times 2545 = 3818\text{kN}$ or $Q_{ult} = 2 \times 1905 = 3810\text{kN}$
- ◆ Load combination 2 – $Q_{ult} = (1.5 \times 2709)/1.25 = 3251\text{kN}$ or $Q_{ult} = (2 \times 1997)/1.25 = 3195\text{kN}$
- ◆ Load combination 3 – $Q_{ult} = 1.5 \times 2294 = 3441\text{kN}$ or $Q_{ult} = 2 \times 1593 = 3186\text{kN}$

Thus pile Nos 1 to 5 were constructed with a minimum ultimate capacity of 3818kN. Interestingly this ultimate load is almost 2.5 times the load of 1541kN calculated for load combination 1 by structural methods. The other rows of piles were treated in a similar manner to that described above with regard to axial capacity. Pile bending moments will be discussed in the next section of this paper.

A preliminary pile test was undertaken prior to the construction of the main piling works. A nominal 600mm tripod pile (600mm casing, 550mm tool) was installed in the summer of 1999 to investigate the practicalities of pile installation in these particular ground conditions and also to check design soil parameters. The pile was constructed 11.7m long, of which 5m was through mudstone and the bottom 1m was into siltstone. Recorded pile head settlements were 2.2mm at 2000kN and 5.2mm at 3500kN. This test confirmed that pile construction using the tripod technique was feasible, although very slow.

Pile reinforcement design

A design philosophy for pile bending in these types of analyses is to provide a load factor in accordance with BS8110 or BS5400, as appropriate, on the average pile moment in each row of piles (here BS5400 was required). In addition the analysis also has to ensure that a load factor of at least unity is provided on the maximum pile moment in each row. For the Newark Dyke project, with its unusual dynamic load effects, it was decided to provide a load factor of (1.1 x 1.5) on the average moment and a load factor of 1.5 on the maximum moment in each row for the calculation of the ultimate limit state. All reinforcement design was undertaken using in-house software, which considers the code requirements at both the ultimate limit state and at the serviceability limit state.

Again as an example consider row No. 1. The following serviceability and ultimate bending capacities, BM_{safe} and BM_{ult} , are required for these five piles from Tables 2 to 4;

- ◆ Load combination 1 – $BM_{safe} = 1.0 \times 135 = 135\text{kNm}$
- ◆ Load combination 2 – $BM_{safe} = 1.0 \times 152 = 152\text{kNm}$
- ◆ Load combination 3 – $BM_{safe} = 1.0 \times 142 = 142\text{kNm}$
- ◆ Load combination 1 – $BM_{ult} = 1.1 \times 1.5 \times 135 = 223\text{ kNm}$ and $BM_{ult} = 1.5 \times 164 = 246\text{kNm}$
- ◆ Load combination 2 – $BM_{ult} = 1.1 \times 1.5 \times 152 = 251\text{ kNm}$ and $BM_{ult} = 1.5 \times 186 = 279\text{kNm}$
- ◆ Load combination 3 – $BM_{ult} = 1.1 \times 1.2 \times 142 = 187\text{ kNm}$ and $BM_{ult} = 1.5 \times 186 = 279\text{kNm}$

Thus pile Nos 1 to 5 were to be constructed with a serviceability bending capacity of 152kNm and an ultimate bending capacity of 279kNm, combined with the appropriate pile axial loads.

Reinforcement within the piles was not checked for the serviceability concrete crack control requirements for any load combination in accordance with BS5400 Part 4. Such concrete crack control calculations were undertaken for load combination 1 only, using the less stringent BS8110. TRL Report 144 recommends that for fully embedded piles reinforcement should always be kept to a minimum.² This is because additional reinforcement in piles for concrete crack control does little to improve durability and can indeed have the opposite effect by inhibiting the formation of dense pile concrete. This design philosophy was very important for this project, especially for the piles constructed in the restricted headroom areas. Here the provision of lapped cages with a high density of reinforcement would potentially result in zones of poorly compacted pile concrete, clearly an undesirable result.

Pile shear reinforcement was designed to BS5400 in a similar manner to that described above for bending.

Post construction comparison with PGROUPN

PGROUPN is a new advanced computer program for the analysis of pile groups subjected to any combination of vertical loads, horizontal loads and moments. Currently torque moments cannot be considered. PGROUPN is the result of a research programme led by Dr. Francesco Basile, now of Halcrow Group, with contributions from Dr. Ken Fleming of Cementation and Prof. Simon Wheeler of the University of Glasgow.³ Key features of this analysis method are a full continuum solution, multilayered soil profiles and soil non-linearity effects. The PGROUPN analysis provides a full boundary element solution of the soil continuum overcoming the interaction factor approximations of the PIGLET method. The inclusion of soil non-linearity effects reduces the load concentration in the outer piles and results in a more realistic and uniform load distribution between all the piles.

Table 5 compares the PIGLET and the PGROUPN results for pile Nos 1 to 5, i.e. for row No. 1, in the south-west pilecap, subject to load combination 1.⁴

Pile No.	PIGLET Pile Load (kN)	PGROUPN Pile Load (kN)
1	2545	2130
2	1589	1540
3	1446	1430
4	1542	1530
5	2403	2120

Table 5 : PIGLET / PGROUPN comparison

As described previously the piles in this row have been designed for an ultimate axial load of 3818kN. The PGROUPN results indicate an increase in design factors of safety to 2.2 on the average pile load and 1.8 on the maximum pile load, compared to 2.0 and 1.5 originally desired from the PIGLET results. The use of this analytical technique can be expected to produce economies for future similar projects.

Construction aspects of the land based piling

The very restricted nature of the piling area for the south-west pilecap can be seen by reference to Figures 4 and 5.



Fig.4 : The south-west pilecap prior to piling (view from river)

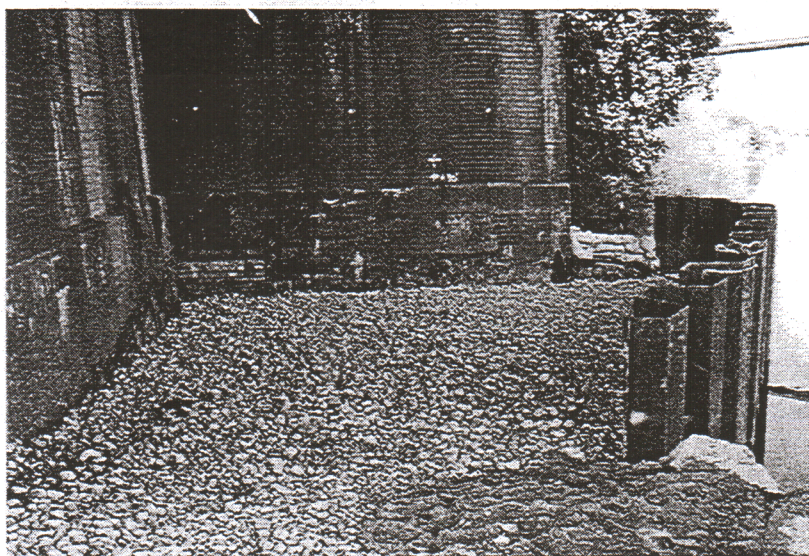


Fig.5 : The south-west pilecap prior to piling (view from under bridge)

Due to the restricted nature of the site it was envisaged at tender stage that the piles would be constructed using the tripod method, which is a cable percussive technique. This is a slow and labour intensive method of piling and only one pile per rig per day would be achieved. However, this technique is normally the only option in such restricted piling areas. A preliminary pile test was

constructed using the tripod method and due to the difficult rock boring the cutting shoe of the tool was changed regularly in order to maintain progress.

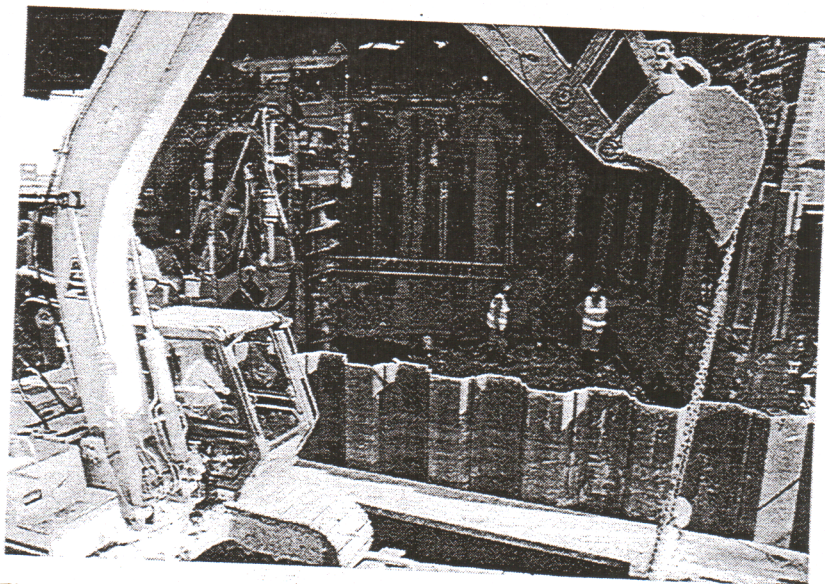


Fig.6 : CFA piling with modified rig beneath live railway bridges (view from barge)

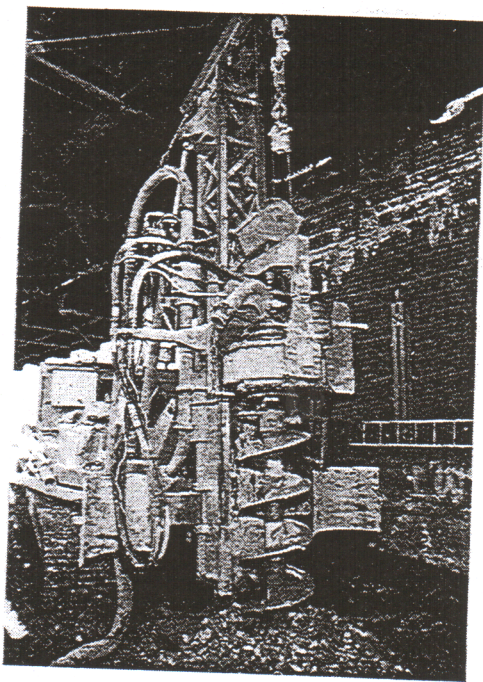


Fig.7 : CFA piling with modified rig beneath live railway bridges

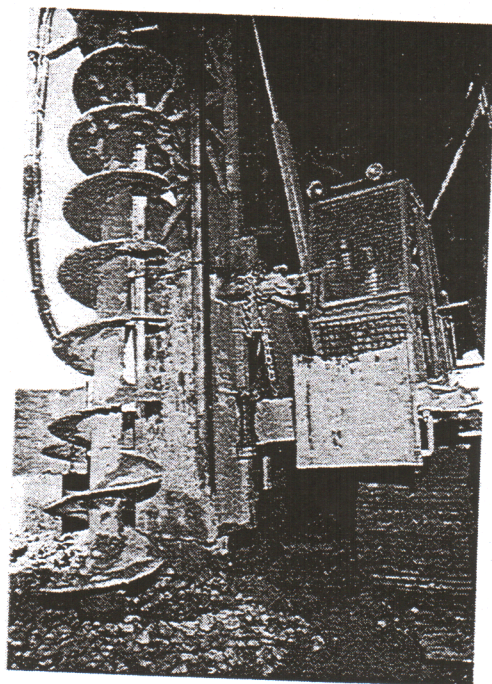


Fig.8 : CFA piling with modified rig beneath live railway bridge

The project was unfortunately delayed at the start, but the date of the slide of the finished bridge structure was fixed due to the programmed limited closure of the railway over a bank holiday. The project team thus needed to reduce the overall construction time and consequently the use of a modified CFA piling rig was investigated. The modified Soilmec CM42 is shown in Figures 6, 7 and 8. This rig had a 6m shortened mast and utilised modified augers of 2 and 2.5m lengths (the standard short auger length is 3m).

Some work had to be carried out to allow the use of the modified piling rig, including the lowering of the piling platform by some 0.5m to provide the required headroom and the temporary removal of a public walkway beneath the existing bridges. Spoil removal was undertaken using an excavator located on a barge on the river.

The Mercia Mudstone is notoriously variable and deciding the appropriate founding level for CFA piles can be problematic. For this project CFA piles were constructed to the design length or to practical refusal, being defined as 2 minutes of boring with less than 100mm of rock penetration. There were no problems with flighting of the upper softer soils.

During concreting of CFA piles a positive head of concrete must be maintained within the ground. This was achieved by trapping the auger with a joint 1m above piling platform level, before removal of the auger section above the joint. The trapping was essential to prevent the auger from dropping under its own weight into the wet pile concrete. This concreting process is analogous to the tremie technique used in the concreting of large diameter rotary bored piles.

Every pile beneath the existing bridges was constructed using the CFA method. For the pile bases adjacent to the bridges the CFA method was used for all piles except for those closest to the railway line and the overhead conductors. Here the tripod method was kept.

The switch from the tripod piling method to the CFA method was crucial to the successful and timely completion of this project. An average of five CFA piles were constructed per day compared to the envisaged one per day for each of the two tripod rigs originally intended. This change saved six weeks on the overall piling programme, and the costs were also reduced.

The performance of the CFA piles will be superior to that of the tripod piles, with increased shaft and end bearing capacity. However, differential settlements between CFA and tripod piles within the same pile group will be of little or no consequence for this project due to the small pile movements at the working condition. No CFA pile working tests were carried out, except for integrity testing.

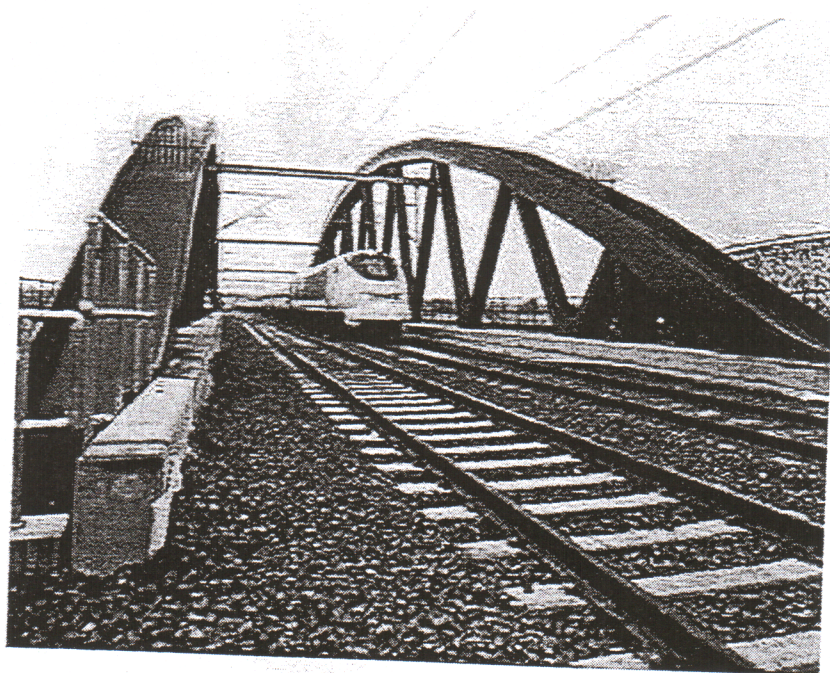


Fig.9 : The completed bridge structure

Conclusions

The bridge slide was successfully undertaken by Skanska Construction in August 2000 and construction of the new permanent structure is now complete, as shown in Figure 9. This project was also the winner of the "prestige award scheme 2000" of the East Midlands Association of the Institution of Civil Engineers.

Pile design was undertaken with due consideration of the type of structure involved, and the need to balance both technical and practical project specific requirements to produce the desired solution.

The analysis of pile groups necessarily needs to take due consideration of the limitations of the design assumptions employed. The use of new analytical techniques such as PGROUPN will assist the engineer in this regard.

The construction of piles in areas of limited access, headroom and/or space is described. The implications of such construction on design must be considered prior to starting on site.

The authors wish to thank Dr. Francesco Basile for permission to publish results he obtained using PGROUPN, and Railtrack PLC for support in publication of this paper.

References

1. E. JEFFERS: "Steel H bearing piles driven into Mercia Mudstone", Ground Engineering, June 1995
2. J.P. TYSON: "Design of reinforcement in piles", TRL Report 144, 1995
3. F. BASILE: "Non-linear analysis of pile groups", Proc. Institution of Civil Engineers, Geotechnical Engineering, April 1999
4. F. BASILE: Personal communications, 2001