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Interpretation of large diameter preliminary pile tests in Chalk for HS2

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ABSTRACT: Eleven instrumented large diameter preliminary test piles were installed and tested in Chalk Group (Grey Chalk Subgroup and White Chalk Subgroup) for the purpose of verifying the design and construction methodology of bored piles on contracts C1 and C2 (Main Works Civils Contracts) of HS2. The test piles were strategically constructed at the location of three major viaducts: eight piles for the Colne Valley Viaduct (C1), one pile at Wendover Dean Viaduct (C2) and two piles at Small Dean Viaduct (C2). The test piles are embedded in Seaford Chalk overlying Lewes Nodular Chalk, Holywell Nodular Chalk overlying Zig Zag Chalk, and New Pit Chalk overlying Holywell Nodular Chalk respectively, covering a wide range of chalk stratigraphical units and weathering conditions. Pile lengths ranged from 36 to 60 metres, constructed using bentonite (seven) or polymer (four) as support fluid. Except for the test pile at Wendover Dean Viaduct, which was top loaded with a reaction frame, all piles were tested using the Osterberg Cell bi-directional load test method. Field test results were interpreted and back-analysed to verify pile performance and investigate ultimate shaft friction and end bearing resistance and compared against the CIRIA C574 guidelines for designing piles in Chalk. The impact of support fluid type on the load bearing capacity of the piles was assessed, with particular consideration of the pile test results given to the potential reduction in shaft friction due to delayed construction times when using bentonite support fluid.

KEYWORDS: Test piles, Chalk, HS2, Osterberg Cell, O-cell, Bentonite, Polymer, Support fluid, CIRIA C574, Back-analysis

1 INTRODUCTION

The High Speed Two (HS2) project, currently under construction, represents a vital leap in the advancement of high-speed rail in the UK. The first phase of this ambitious project spans across the English Midlands, linking London and Birmingham.

The C1 contract of HS2, starts approximately 20km west of central London and runs through the Colne Valley and the Chiltern Hills, including the 3.4km long Colne Valley Viaduct (CVV) and the 15.75km long Chiltern Tunnels founded in Chalk. The package was awarded to Align JV, a joint venture of Bouygues Travaux Publics, VolkerFitzpatrick, Sir Robert McAlpine, and its design partners Jacobs Engineering and Ingerop-Rendel. The piles were installed by Keller and VSL, while Fugro Loadtest provided the Osterberg Cell (O-cell) and performed the pile testing.

The C2 contract, located north of C1, covers the stretch of 33.7km from North Chilterns Area to Twyford, has been awarded to the Eiffage, Kier, Ferrovial, Bam Joint Venture (EKFB) and encompasses the construction of eight viaducts to support the HS2 mainline, two of which, Wendover Dean Viaduct (WDV) and Small Dean Viaduct (SDV) on the southern end of the contract are founded in Chalk. Detailed design services for this contract are being facilitated by the Arcadis, Setec, COWI design joint venture (ASC). The installation of the piles was assigned to the Bauer Keller Joint Venture (BKJV), while Fugro Loadtest and Socotec performed the testing of the piles.

This paper delves into the procedure, interpretation, and back-analysis of eleven preliminary compression test piles, which were performed in various Chalk formations in the English Chiltern Hills. The primary objective of the pile load testing regime was not only to verify the pile design assumptions, such as shaft friction and base resistance, but also to confirm the performance of the piles under serviceability limit state. Furthermore, it sought to validate the efficacy of the chosen construction method to inform the pile design for other assets within the area, thereby paving the way for an efficient pile design for all viaducts and overbridges. A key aspect of this paper is the assessment of the impact of different support fluid types on the load-bearing capacity of the piles and comparing the results in accordance with the CIRIA C574 guidelines for designing bored cast in-situ piles in Chalk.

2 THE SITES

2.1 *Geological Setting*

Stratigraphically, the Chalk Group in England is divided into the Grey Chalk Subgroup and the White Chalk Subgroup. These subgroups are further subdivided into formations. The eight test piles for the C1 contract along the CVV were installed through superficial deposits into the Seaford Chalk Formation, which overlies the Lewes Nodular Chalk Formation. Both formations are part of the White Chalk Subgroup. The test pile at WDV, part of the C2 contract, was also installed in the White Chalk Subgroup, intercepting the New Pit Chalk Formation and the Holywell Nodular Chalk Formation. The two test piles at SDV were also installed through the Holywell Nodular Chalk Formation and into the Zig Zag Chalk Formation. Only Zig Zag Chalk Formation is part of the Grey Chalk Subgroup.

The local ground conditions at the test pile sites have been described in the following sections in accordance with Figure 3.1 of CIRIA C574 "Engineering in Chalk" (2002).

2.2 *Local Ground Conditions at Test Pile Sites*

2.2.1 *Colne Valley Viaduct (CVV)*

Two sites were chosen for preliminary pile load tests for the CVV. The first site (Location L1), located at the northern end of the viaduct, was selected due to its deeper weathering profile in the Chalk, representing a potential 'worst case' ground model for the viaduct. Beneath a 7m thick layer of superficial deposits, the Seaford Chalk Formation at this site was classified as grade Dm/Dc structureless chalk down to 18 metres below ground level (mBGL). This layer overlies a generally low-density (< 1.55 Mg/m³) grade B3 structured chalk extending to a depth of 51mBGL. Within the 33m thick layer of B3 chalk, a total of 6.5m of no core recovery was recorded. Below this, the chalk was typically described as medium (1.55–1.70 Mg/m³) or high density (> 1.70 Mg/m³) grade B3 chalk. At the second site (Location 2), located at the southern end of the viaduct, the ground conditions consist of 2.5m of superficial deposits over Seaford Chalk, generally classified as grade Dm/Dc down to a depth of 12mBGL. This is underlain by medium or high-density chalk grades B1 to B4, extending to a depth of 56mBGL. Below this depth, high-density grade A2 chalk, corresponding to the Lewes Nodular Chalk Formation, was encountered. Standard penetration tests (SPTs) refused above pile toe level at both test sites and therefore the SPT-N value at pile toe level would be in excess of 50. Given the proximity of the CVV test piles to the River Colne, groundwater table was considered at ground level.

2.2.2 *Small Dean Viaduct (SDV)*

Two preliminary test piles were installed and tested near the south abutment of the SDV. Beneath a 1.2m thick layer of superficial deposits, the ground was described as clay with flints down to 8mBGL corresponding to Holywell Nodular Chalk Formation. This is followed by Zig Zag Chalk Formation classified as structureless chalk grades Dc/Dm down to 15mBGL. Below this, medium to high-density structured chalk grade B5 extends to a depth of 22mBGL. High-density chalk grades B2 to B5 extend to 45mBGL, the end of the borehole. SPT-N values indicated refusal (>100 blows) at the depth corresponding to the pile toe. Groundwater at SDV was recorded at or below 17mBGL.

2.2.3 *Wendover Dean Viaduct (WDV)*

One preliminary test pile was installed and tested near Pier 5 of WDV, approximately in the middle of the overall span of the viaduct. The ground conditions at this pile consisted of approximately 1.3m thick layer of superficial deposits, overlying clay with flints down to 5.5mBGL, with a recorded core loss of 3.3m. This is followed by structureless chalk classified as grade Dc/Dm down to 8.7mBGL, corresponding to New Pit Chalk Formation. Below this, low to medium-density chalk grades B5/B4 extend to a depth of 18.1mBGL. Medium to high-density chalk grades B4 to B2 are present down to the bottom of the borehole at 55.2mBGL, noting the bottom 31m of the boreholes correspond to Holywell Nodular Chalk. SPT-N values indicated refusal (>100 blows) at the depth corresponding to the pile toe. Groundwater at SDV was recorded at or below 30mBGL.

3 TEST PILES

3.1 *Purpose of the tests piles*

The test piles were carried out to verify the shaft friction and base resistance, and confirm the performance of the piles under serviceability limit state, for an Eurocode 7 design approach (EN 1997-1:2004). Field test results were interpreted and back-analysed to verify pile performance and investigate ultimate shaft friction and end bearing resistance compared against the CIRIA C574 guidelines for designing piles in Chalk. Equivalent top load-settlement curves were approximated to confirm pile performance under serviceability limit state conditions.

The ultimate shaft resistance of these chalk piles was designed based on the recommendation of CIRIA C574 using the β design approach, i.e. $f_s = \beta\bar{\sigma}_v'$, where the β value was assumed to be 0.8 and $\bar{\sigma}_v'$ is the average vertical effective stress of the ground, excluding made ground or fill. An additional cautious estimate reduction factor of 20% was also allowed for installation effects of such piles in bentonite support fluid with construction duration of over 24hrs period. The ultimate base resistance of the pile was assumed to be $f_b = 200N$ where N is the standard penetration test blow count value.

3.2 *Description of the tests*

A total of seven out of the eleven test piles were constructed under a bentonite support fluid, including all the piles in C2 (SDV and WDV) and four of the test piles on C1 (CVV). An additional four piles were tested under polymer support fluid at the CVV test sites, in order to determine which support fluid was optimum in terms of both constructability and for load bearing capacity. Both support fluids were fully exchanged before concreting.

In terms of load testing, the Osterberg Cell (O-Cell) bidirectional load test method was employed for the bored piles at both the SDV and CVV. The O-cell testing method provides distinct advantages over conventional static load testing, primarily by eliminating the requirement for external reaction systems such as kentledge or reaction frames. The preliminary test piles undertaken using o-cells for the HS2 project were intended to load the full size test piles to failure in order to verify the pile capacities. This will require test load in excess of 30MN which is considered the limit of conventional top-loaded test pile setup without resolving to disproportionate large and expensive reaction frame system. Due to uncertainties and difficulties experienced in the test piles using o-cell at SDV, a decision was made to use a top-loaded test pile at WDV to verify the load-settlement or serviceability characteristic of the chalk pile which only required a test load of 28MN at WDV using a conventional reaction frame method.

For the test piles at the CVV the load was incrementally applied until the estimated ultimate load of the pile was reached. Each increment of load was held until the rate of O-Cell expansion was less than 0.25mm/hour, with a maximum three-hour hold period. Generally the rate of expansion reached this threshold in the earlier load steps but continued to creep in the final 2 to 3 load cycles. Upon reaching the maximum specified test load, the test was continued by increasing the load further in small increments to effectively ‘fail’ the pile, thereby providing the maximum amount of load versus displacement data. All load tests were terminated upon the O-Cells approaching maximum expansion (approximately 225mm) or the O-Cells being unable to sustain the required pressure. Table 2 includes the maximum O-Cell movement (upwards and downwards) for the last sustained load step and the maximum sustained total load. At SDV, the two test piles were loaded until the load was not stable during the full duration of the load increment (PTP01A) or the O-cells were approaching their maximum expansion limit of 225mm (PTP02). Each successive load increment was held constant by automatically adjusting the O-cell pressure until the settlement criteria were met or the maximum load hold duration was achieved. The maximum sustained bi-directional load was 21.03 MN and 17.66 MN for PTP01A and PTP02, respectively. It should be noted after the 3-hour hold at the last load increment, settlement rates in both piles still exceeded 0.25 mm/hr.

At the WDV site, a different approach was taken with the installation and testing of a sacrificial test pile using a conventional test pile method with a reaction frame, see Figure

1. The load was applied using two 15MN hydraulic actuators and monitored with two 7.5 MN strain gauge load cells with multiple levels of strain gauges to monitor load transfer during the test. The test followed the protocols described in the ICE SPERW Third Edition, and involved four loading-unloading cycles of 12, 18, 24, and 28MN, with the maximum load equivalent to 100% Design Verification Load (DVL) + 50% Representative Load (FREP). For the two hold periods where load was sustained more than 3h, the average settlement rate was around 0.03mm/hours. A summary of the eleven test piles is presented in Table 1.

Figure 1. Reaction frame for the top loaded test pile at WDV.

Table 1. Summary of preliminary test piles

Pile Ref	Sup. Fluid	L/O-cell depth [m]	Dia [m]	T [hr]	D [days]
SDV-PTP01A	B	40.3 / 29.2	1.5	29	53
SDV-PTP02	B	36.4 / 28.4	2.0	34	32
WDV-TP01	B	44.5 / -	1.5	38	29
CVV/L1/LTP1	B	45 / 38	1.5	56	29
CVV/L1/LTP2[1]	B	60 / 45	1.5	68	33
CVV/L1/LTP3	P	45 / 38	1.5	41	26
CVV/L2/LTP4[2]	B	60 / 45	1.5	86	39
CVV/L2/LTP5	P	45 / 34	1.5	39	34
CVV/L2/LTP6	P	45 / 34	1.5	34	40
CVV/L2/LTP7	B	45 / 38	1.5	62[3]	49
CVV/L2/LTP8	P	45 / 34	1.5	36	34

L=pile length, Dia=pile diameter, B=bentonite, P=polymer, T=time from boring below casing to completion of concreting D= duration between pile construction and load test
[1] First test pile installed with delay and construction issues
[2] Pile test to investigate delayed in concrete placement
[3] Drilling time of first 4m not included.

3.3 *Test Pile Instrumentation*

For test piles with O-Cells, the hydraulic cells were cast into the piles at predetermined positions to balance the resistance of the upper and lower sections of the piles. Sister bar vibrating wire strain gauges were positioned at various levels in the pile to measure the load distribution along the pile shaft. These were spaced at 90° in sets of four to allow for load eccentricity, redundancy for installation damage or malfunctioning gauges. The expansion of the O-Cells and the displacement of pile head were measured using linear vibrating wire displacement transducers (LVDT) and telltale extensometers were used to measure compression of the pile above and below the O-Cells. These extensometers were also installed at different levels of the test piles at SDV test site to provide data for added redundancy measurements to verify the data of the strain gauges. Precise levelling was also carried out at the head of the pile using an automated digital survey level. A photo at CVV test site is shown in Figure 2 below:

Figure 2. Photo at CVV test site.

3.4 *Interpretation of the results*

3.4.1 *Colne Valley Viaduct (CVV)*

Equivalent top-loaded pile test curves were prepared for all the test piles, based on the method recommended by J.O. Osterberg (1998). These are presented in Figure 3 and 4 for the piles constructed under bentonite and polymer support fluid, respectively. This is a useful way of estimating the pile response should it have been loaded from the pile head, where the dashed lines are extrapolated downward movement.

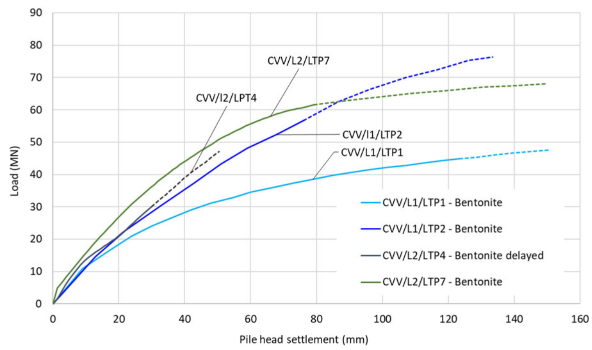


Figure 3. Equivalent top-load plots for piles constructed under bentonite at CVV.

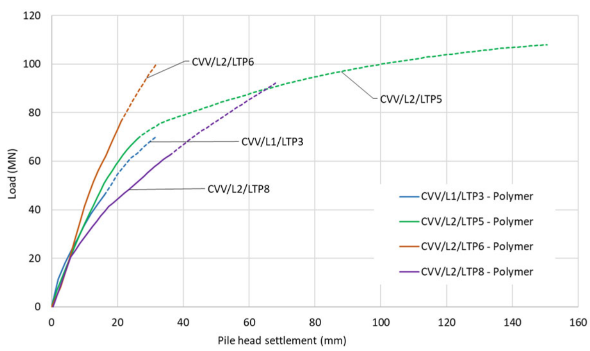


Figure 4. Equivalent top-load plots for piles constructed under polymer at CVV.

The ultimate capacity of the test piles has been determined by assessing the load on the equivalent top-load curve at a downward movement equal to 10% of the pile diameter, in this case 150mm. The interpreted ultimate capacities of the eight preliminary test piles for the CVV are presented in Table 2.

Table 2. Upwards and Downwards O-Cell moment and inferred ultimate capacity of the CVV test piles.

Pile Ref CVV/	O-cell movement (mm) Up	O-cell movement (mm) Down	Max. applied load[1] [MN]	Ultimate Capacities[2] [MN]
L1/LTP1	106	120	46.1	47.6
L1/LTP2	168	53	70.6	80.0
L1/LTP3	9	217	72.7	110.0
L2/LTP4	118	33	48.4	70.0
L2/LTP5	68	24	89.4	108.0
L2/LTP6	25	14	80.1	200.0
L2/LTP7	153	74	71.5	68.0
L2/LTP8	125	46	85.3	110.0

[1] O-Cells maximum expansion approximately 225mm or last sustained total load
[2] Assessed from the equivalent top-down curve at a movement equal to 10% of the pile diameter.

3.4.2 *Small Dean Viaduct (SDV)*

Eight to nine levels of four strain gauges, three to four extensometers and displacement transducers were installed in the test piles to allow detailed measurement of load shed in the pile shafts and eventually to the pile toes at different magnitude of applied load during the tests. Details of these test piles are presented in Table 3.

Table 3. Details of SDV preliminary test piles

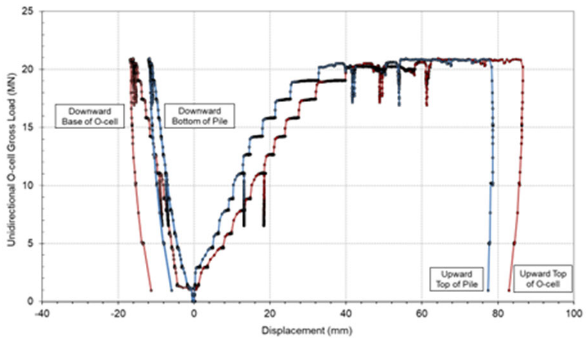
Pile	Design resistance [MN] Shaft	Design resistance [MN] Base	Measured resistance [MN] Shaft	Measured resistance [MN] Base
PTP1A	27.7	35	29	6[1]
PTP2	32.2	30	24	6[1]

[1] Mobilised end bearing resistance

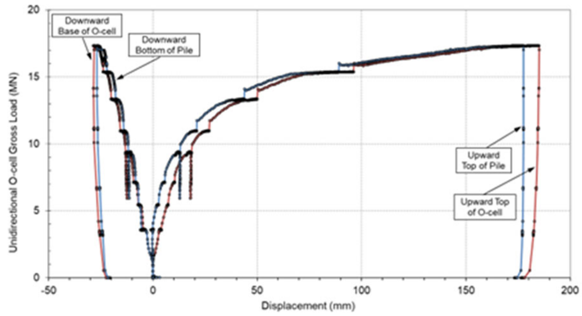
Figure 5 shows the load-displacement profiles of the two test piles with the interpreted shaft and base components of the piles summarised below.

While undertaking interpretation of the test piles data, it was apparent that the stiffness of the 2000mm test pile was significantly lower than the smaller 1500mm test pile during the tests. Further coring of the pile concrete revealed segregation in the concrete in the upper shaft close to the O-cell although the strength is only marginally lower than the statistical determined characteristic value of 32MPa.

For the pile shaft resistance, the two preliminary test piles at SDV produced conflicting β values with the average measured value of the smaller 1500mm diameter test pile of 0.7 while the corresponding β value for the larger test pile was approximately 0.4. Based on the performance of the 1500mm diameter test pile, the 20% reduction allowed for the pile design is justified. The lower than expected shaft resistance performance of the larger test pile was a concern and this was rectified by a confirmatory test pile at the WDV, which is located about 1.5km from the SDV viaduct test pile site.



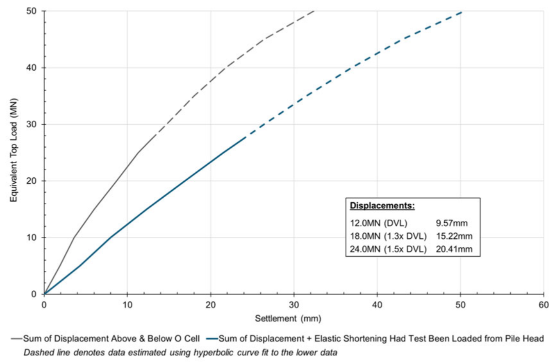
a) PTP1A (1500mm)



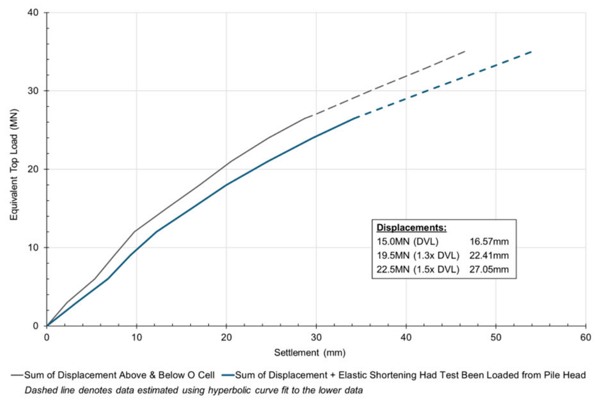
b) PTP2 (2000mm)

Figure 5. Load-displacement of O-cell tests at SDV

Their equivalent top load-settlement behaviours are shown in Figure 6 below:



a) PTP1A (1500mm)



b) PTP2 (2000mm)

Figure 6. Equivalent top loaded load-displacement of the tests at SDV

The two test piles at SDV did not achieve full mobilization of end-bearing resistance. To further evaluate the performance of the larger 2000 mm diameter pile, the O-cell test data was back-analysed prior to conducting a top-down load test to failure, confirming the pile's total capacity (Vitorino & Yeow, 2024). This assessment had shown the ultimate capacity of the test pile was greater than the 32.2MN despite the lower shaft capacity measured during the tests. As the pile needs to rely higher component of the capacity from the base, settlement of the pile is likely to be slightly higher which requires the design of the viaduct to ensure serviceability limit state criteria is verified.

3.4.3 *Wendover Dean Viaduct (WDV)*

The 1500mm test pile at WDV was tested using a conventional test pile with a reaction frame to provide a maximum test load of 28MN. It was located approximately 1.5km from the SDV test pile site with a purpose to verify the low β value measured in the 2000mm diameter test pile at SDV. At 100%DVL of 12MN, the pile settled 6mm while at SLS verification load of 100%DVL + 50%F_{rep} the pile settled slightly over 10mm. These settlement values are well within the 10mm for 100%DVL and 15mm for 100%DVL + 50%F_{rep}. At the maximum test load of 28MN, the test pile only settled 26mm.

Due to the malfunction of multiple sets of strain gauges, the shaft resistance was back analysed using several other methods, including Chin and CEMSET methods and the β value was found to be 0.6. The average vertical effective stress determined for the WDV back analysis was 335kPa.

3.4.4 *Comparison of all Test Piles*

Figure 7 presents the normalized pile head load–settlement curves for all test piles. The vertical axis shows the load ratio (applied load divided by ultimate pile capacity), and the horizontal axis shows the displacement ratio (pile head settlement divided by pile diameter). Three distinct trends in pile behavior are evident. Assuming a design load corresponding to a load ratio of about 0.35, the SDV and WDV test piles exhibit the stiffest response, with a maximum displacement ratio of approximately 0.75%. For polymer-supported piles at the CVV site, this value is about 1.25%, while bentonite-supported piles at the same site reach roughly 2%. At 150% of the design load (load ratio ≈ 0.525), the maximum displacement ratios increase to approximately 1.1% for SDV/WDV piles, 1.75% for polymer-supported piles, and 2.5% for bentonite-supported piles.

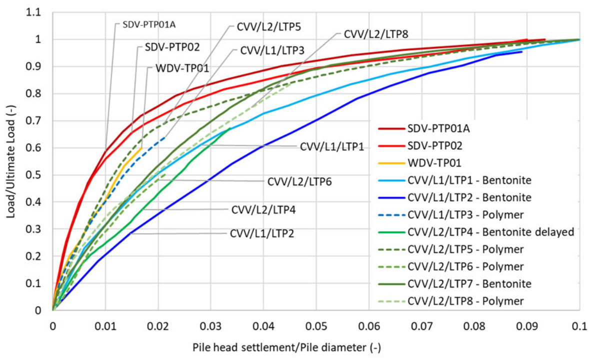


Figure 7. Normalised top load displacement curve for all piles.

Based on the geological setting and construction details of these test piles (see Table 1), the following are factors contributing to the increase in measured stiffness:

- Consistently higher chalk density and higher uniaxial compressive strength of the intact chalk of SDV and WDV test sites compared to the CVV test sites;
- Shorter construction times, that could have led to less development of filter cake for the test piles at SDV and WDV;
- The effects of remoulding of the chalk during drilling and the re-cementing of the remoulded chalk with time;
- The use of polymer support fluid led to improved shaft capacity compared to the longer construction duration bentonite piles, where a potential build-up of bentonite cake on the shaft of the CVV test piles may have inhibited pile performance.

4 COMPARISON WITH CIRIA C574

4.1 *Shaft friction*

To compare the β values derived from the test piles with the CIRIA C574 bored pile design recommendations, the average shaft resistance versus vertical effective stress for the test piles has been superimposed onto Figure 8.3 of C574 and is presented here as Figure 8. It should be noted that the β values in CIRIA C574 represent a lower-bound estimate, and the guidance recommends pile load testing to optimise pile design and validate assumptions. When considering the calculation of shaft friction as $f_s = \beta \bar{\sigma}_v'$, the following was surmised in relation to the β value:

- piles constructed under polymer support fluid demonstrated β values of ≥1.0;
- pile constructed under bentonite support fluid show data consistent with the CIRIA document, if the PTP02-SDV test result is not considered;
- the 20% reduction in shaft resistance adopted for the HS2 project (β = 0.64) proved adequate for piles constructed under bentonite support fluid.

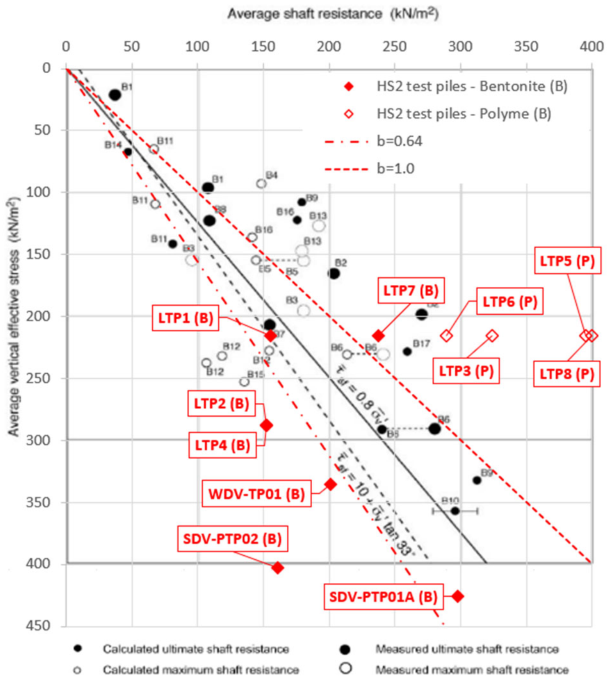


Figure 8. Average shaft friction of test piles superimposed on C574's data (© *CIRIA 2002*)

4.2 *End bearing resistance*

Mobilised end bearing resistance on the CVV test piles varied from 4.4MPa (LTP6) to 19.3MPa (LTP2). Most of these values do not represent an 'ultimate' end bearing resistance due to insufficient downward movement at the toe of the piles. It is considered likely that the actual ultimate end bearing resistance is closer to the upper end of this range, given that the lower values correspond to piles where limited downward movement was recorded. For the test piles at SDV and WDV, the measured mobilised bearing resistance is between 1.3 and 6MPa. For both these sites the end bearing resistance was not fully mobilised.

4.3 *Impact of support fluid*

The performance of the test piles constructed using polymer support fluid at the CVV test site clearly demonstrated higher pile shaft capacity and stiffer load-settlement response than the test piles constructed under bentonite at the same test site, as presented in Figure 9.

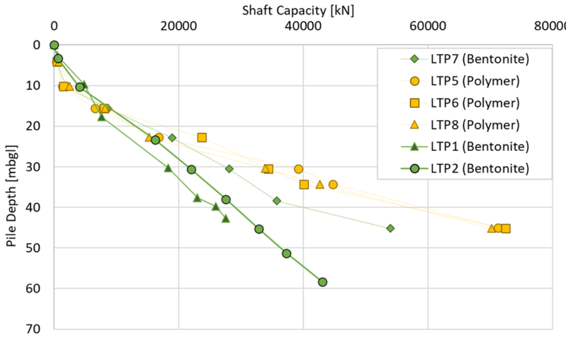
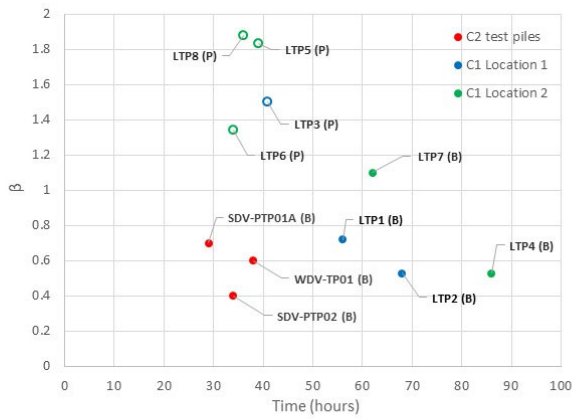


Figure 9. Mobilised shaft resistance over depth along pile shaft for test piles at CVV installed under bentonite and polymer support fluids

This improved performance may be a result of the quicker construction duration for piles constructed under polymer, which does not require desanding as bentonite does. These piles were also constructed after the first bentonite test piles and therefore the efficiencies and lessons learned from the first bentonite piles (such as reducing the number of sections of reinforcing cage) could be adopted for the construction of the polymer test piles. The test piles installed under polymer also demonstrate higher shaft resistance but a softer load-settlement response when compared to the test piles at SDV and WDV (all of similar length construction duration). The softer response when compared to the SDV and WDV piles is considered to be due to the lower density and strength of the chalk at the location of the CVV, as discussed in 3.4.4, not necessarily a function of the support fluid used for construction. An average mobilised skin friction of 353kN/m^2 was achieved in the piles constructed using polymer, versus an average mobilised skin friction of 174kN/m^2 in the piles constructed using bentonite at the CVV test sites. The results would justify using a β value of 1 or more for the design of piles constructed under polymer, suggesting that the guidance in C574 may not be directly applicable for piles using this support fluid.

4.4 *Impact of delayed construction times*

The two test piles constructed at SDV using bentonite as a support fluid, PTP01a and PTP02, were constructed in accordance with the ASC Specification for Preliminary Test Piles, so the time between commencing excavation below casing and concreting did not exceed 36 hours. The test pile constructed at WDV had a concreting time of 38 hours, which marginally exceeded the specified maximum of 36 hours. At the CVV test sites, test piles installed under polymer support construction time was not too dissimilar to the SDV and WDV test piles. The test piles at the CVV test sites constructed under bentonite support fluid took a minimum of 56 hours under normal construction circumstances without intentionally extending the construction duration. These long durations were largely due to the time required to connect instrumentation and hydraulic cables as sections of reinforcement cage were dropped into the pile. The effects of construction time on the performance of the shaft resistance is presented in Figure 10.



Note 1 – LTP7 was installed in three shifts and the time presented corresponds to drilling from 20mBGL until completion of concrete (temporary casing 16m long).

Figure 10. Variation of measured β values with construction time from boring below the temporary casing to completion of concreting

5 CONCLUSIONS AND DESIGN RECOMMENDATIONS

Eleven preliminary load test piles were installed and tested in the chalk at the CVV, SDV and WDV test pile sites for the HS2 project. These tests were undertaken primarily for the purpose of validating the design assumptions for axially loaded bored piles supporting the viaducts in this section of the highspeed railway. The following significant observations have been made from review of the results of these test piles:

- The average shaft resistance values measured are within the spread of case history database presented in CIRIA C574 for bored cast in-situ piles (see Figure 8);
- Bored piles constructed using a polymer support fluid exhibited higher shaft resistance with β values exceeding 1.0;
- Overall shorter construction time is achieved with the use of polymer support fluid;
- The design of bored piles installed under bentonite support fluid should allow a reduction in the β value. A 20% reduction is found to be reasonable for piles concreted up to 68hr from commencing boring below the bottom of casing to completion of concreting;
- The stiffness or load-settlement response of bored pile is affected by the type of chalk with pile installed in higher density chalk exhibiting stiffer response.

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