Geotechnical assessment strategy for bridge maintenance – case study

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ABSTRACT: This paper presents a practical strategy used to conduct a geotechnical assessment, drawing principally on a maintenance work carried out recently for Rashwood Interchange which carries the M5 Motorway over the A38. The bridge, which was constructed in the early 1960s, had experienced long-term settlement attributed to historical brine pumping activities in the proximity of the bridge area. In planning for its maintenance work several issues challenged the geotechnical assessment, including the review of settlement history and mining instability in the area, the exploitation of as-built data records and the determination of foundation response to additional loading during the bridge repair. The paper presents how these complex challenges were approached, yet using simple procedures and common design tools. The procedures are also applicable to other infrastructure maintenance projects, particularly in transportation geotechnics.

1 INTRODUCTION

Infrastructure assets, currently in operation, will require to be partially, progressively or completely replaced in order to maintain their service (ICE, 2013) and meet the public expectations on health, safety and limited environmental impact. Bridges in the UK are essential assets and therefore they are routinely inspected and maintained. In planning for their maintenance, geotechnical risk is managed (HA, 2008) by conducting a suitable ground investigation and assessment to ensure the proposed repairing operation would not cause adverse effects on the existing bridge subs or superstructures. The geotechnical assessment of such structures may contain different approaches and procedures, adopted by geotechnical professionals, depending mainly on the type of structure, ground condition, and nature of work to be carried out (Hamza and Bellis, 2008).

Rashwood Interchange Bridge had been subjected to this type of maintenance by the Highways Agency (currently Highway England). The Interchange Bridge, shown in Figure 1, carries the M5 Motorway over the A38 at junction 5. Rashwood Interchange Bridge was designed by Hereford and Worcestershire County Council and constructed circa 1961.

The bridge consists of three continuously supported spans (as shown in Figure 2) formed from steel beams with an insitu reinforced concrete slab supported by piled bank-seats and slab wall piers on spread footing foundations. The abutment bank-seat and wing-walls are piled into Mercia Mudstone using bored reinforced concrete piles. The Structure is fixed at north abutment, free over piers and south abutment (roller bearings). Both bearing types contain shear pins to restrain lateral forces and maintain alignment. The bridge deck spans are comprised of composite 203mm thick reinforced concrete deck slabs with longitudinal fabricated steel I-Beams connected by shear studs. Each deck span consists of eleven steel I-beams at 1.45m centres continuous over three spans of skew length 14.48m, 21.65m and 14.48m at skew angle of approximately 41°. Each steel I-beam consists of two welded sections.

During routine inspections excessive vertical movement was noticed at the bridge deck ends. A number of Roller bearings were found to be experiencing uplift at the south abutment, with up to 2mm separation. Additional to the vertical movement it was noted that the steel components of the abutment bearings were corroded and in poor condition. The steel deck beams at the abutments were also heavily corroded and require paint remedial works to prevent further corrosion damage.

An assessment (carried out by Amey Plc) concluded that the bearing shelf had settled shortly after construction. In order to ascertain the levels of settlements that may have occurred at this site, a level survey was undertaken in October 2012. Levels were taken at the bearing pedestal of each pier, at the northern and southern abutments and the central pier and have been compared to as-built levels. The survey identified a maximum level difference between the as-built data and the recent survey data of some

200mm indicating a settlement range of approximately 100mm to 200mm between construction (circa 1961) and 2013. It is considered that the levels of settlement recorded may be a result of historic brine pumping and extraction within the Droitwich area.

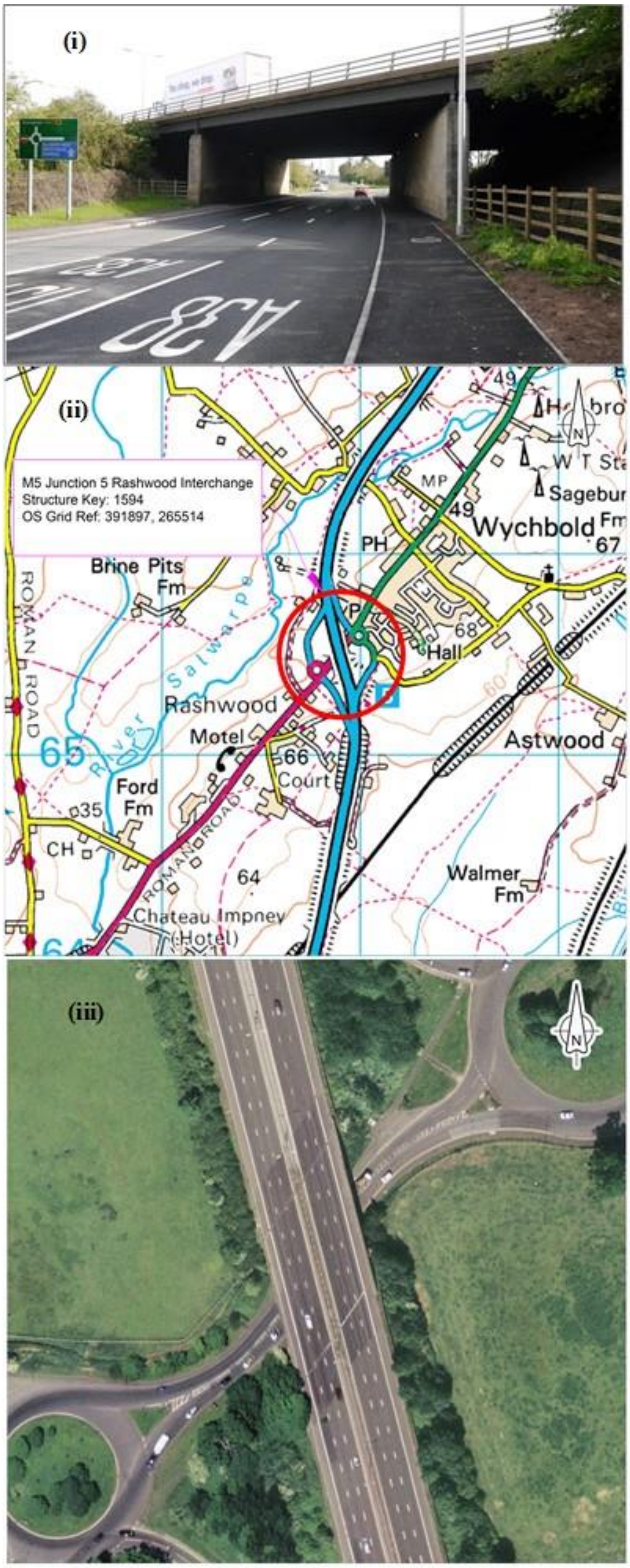

Figure 1. (i) Photo of Rashwood Interchange Bridge (ii) Map showing the location of bridge (iii) Aerial view of the bridge.

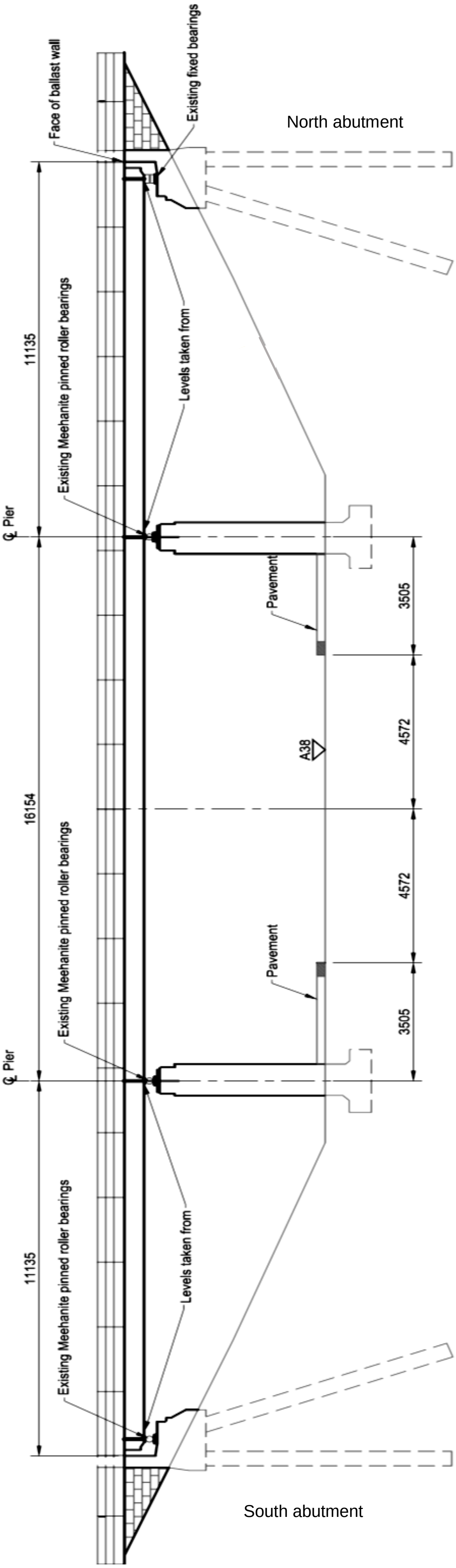

Figure 2. Cross section and elevation of the bridge.

The maintenance work planned and carried out for the scheme consisted of jacking the bridge deck at the south west abutment and inserting steel shim plates at the affected bearings. Temporary works were required to support the bridge deck and facilitate the bridge deck lifting. The temporary works included access scaffolding to the abutments and temporary jacking arrangement.

To ensure the safety and serviceability of the foundation under the new temporary supporting system (i.e. during the proposed jacking) as well as under the new support arrangement (i.e. steel shim plates), a geotechnical assessment was required to predict any potential over-stressing or excessive settlement of the existing pile foundation, particularly at south abutment where the jacking was planned. In addition, historical information was reviewed as part of the geotechnical assessment to confirm that the settlements identified by the level survey are the result of brine subsidence in this location. With rising pressures on control of safety, quality, costs and time, the geotechnical assessment was faced by several technical challenges as presented in this paper. The project was successfully delivered on site in May 2015 and completed to the satisfaction of the client.

2 STRATEGY USED IN THE GEOTECHNICAL ASSESSMENT

The strategy carried out to assess the effects on the foundation system under the temporary loading (exerted by the bridge maintenance e.g. steel work and jacking operation), 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:
  - Review of mining reports and all other relevant information (such as the settlements identified by the level survey) in order to assess the likely risk of future settlements at the bridge location.
  - Review of available geotechnical data and determine ground model parameters including strength and stiffness, to be used in the analysis.
- Identify the pile group arrangement from as-built records.
- Structural analysis of pile –soil system to evaluate stress distribution on the piles during jacking process and after completion to determine the internal forces i.e. axial/ lateral forces and bending moments.
- Determine the ultimate capacity of the piles at the south abutment of M5 Junction 5 Rashwood Bridge and identify any risk of negative skin friction or pull-out forces.

It is believed that by following these steps any critical condition that may be induced by the temporary supports (during the maintenance work) on the existing foundation can be identified. More details about these procedures are provided in the following sections.

3 REVIEW OF EXISTING INFORMATION

3.1 *Geology of the Site*

The subsoil information used to establish the ground model was obtained from various sources, including the Geological mapping available on the Highways Agency Geotechnical Data Management System (HA GDMS), together with the BGS Geological Map Sheet 182 for Droitwich (BGS, 1976).

The geological plans indicate that the site is underlain by solid geology comprising mudstone of the Triassic Mercia Mudstone which is formerly known as Keuper Marl (Benton et al., 2002). Although not indicated on the geological plan, it is considered likely that made ground in the form of engineered fill for the approach road embankments, will overlay the solid geology at this location. This was confirmed by existing ground investigation factual and interpretative reports available for the site (M5 Motorway) and its vicinity (HAGDMS, 1982, 2002). In one of these reports, a borehole identified possible fill material to depths of between 1.35m and 1.5m below ground level (bgl) directly underlain by Mercia Mudstone Group, described as firm to stiff reddish brown and light grey friable silty clay with highly weathered mudstone lithorelicts (HAGDMS, 1982).

Boreholes undertaken as part of another investigation in 2002 (HAGDMS, 2002) were located approximately 40m to the north of the northern abutment at existing carriageway level. These boreholes identified the presence of made ground, considered to represent ‘possible embankment fill’ material, to depths of between 2.6m bgl and 4.0m bgl. Mercia Mudstone, described as stiff red brown occasionally mottled green grey slightly gravelly clay, was encountered directly beneath the ‘possible embankment fill’. The composition of the ‘possible embankment fill’ was noted to be of varying composition. Red brown clayey sands and gravels were noted, with the gravel comprising mudstone and quartz together with stiff to very stiff red brown sandy gravelly clay. It is considered that the ‘possible embankment fill’ is consistent with reworked Mercia Mudstone material.

3.2 *History of mining (brine pumping) in the area*

The Ove Arup Partnership Review of Mining Instability in Great Britain report was reviewed as part of this assessment. It notes that subsidence from brine pumping has affected the Droitwich area. Specific documents referred to in the Mining Instability report (Ove Arup, 1990) have been reviewed. Based on the this information, it can be postulated that the Rashwood Interchange is within Zone B (passive brine run) described as areas that are known or anticipated from precise levelling and/or surface features to be experiencing minor ground movements at worst with little or no differential factor.

3.3 *Review of walkover survey and settlement records*

Structural inspections undertaken in 2012 identified excessive vertical movements at the bridge deck ends. A 'bouncing' movement of the bridge deck was evident when heavy goods vehicles passed over the expansion joint between the abutment and bridge deck. Initially it was considered that settlement of engineered fill material, possibly as a result of shrinkage, behind the abutments may have led to a lip forming between the deck and embankment fill. However; during a site walkover there was no visual evidence of a dip in the carriageway surfacing behind the bridge abutments, and information on the surfacing indicates similar road construction thicknesses at both the north and south abutments. This suggests that settlement of the abutment backfill is not the cause of the apparent uplift.

The structural inspections noted 2mm of apparent uplift of the bridge deck in relation to the roller bearings. These findings would suggest settlement of a structural component of the bridge in relation to the bridge deck. Historic assessments have concluded that minor settlements of the abutment bank-seats could account for the apparent 'uplift', which would suggest that settlement of the bank-seat piled foundations had occurred with no settlement of the pad foundations of the central piers. This led to the consideration that larger scale brine subsidence may have been the cause of the apparent settlements.

A level survey of the bridge deck beams has indicated a settlement range of approximately 100mm to 200mm between construction (circa 1961) and 2013. A total settlement since 1982 of between 23mm and 39mm has been recorded at the survey pins located closest to the bridge, at a maximum mean annual settlement of 1.34mm and differential settlement of 0.31mm (WCC, 2012). This would suggest that the majority of the identified settlements occurred during the period following motorway construction in 1961 and the commencement of detailed level monitoring in 1982. The available detailed levelling records do not date this far back and therefore do not include the full settlement profile of the structure.

Evidence from other sources do however note that settlements in the order of 380mm have occurred over a length of approximately 180m of the M5 just south of the Rashwood Interchange (JPB, 1971) between 1961 and 1971. It should be noted that this area is within Zone A, the active brine run area, whilst Rashwood Interchange is considered to be located in Zone B, the passive run area (JPB, 2012). This does provide some surety that the settlement identified at Rashwood can be attributed to brine subsidence in this area. It also indicates that the majority of the identified settlements occurred prior to 1982, and most probably during the 10 year period between motorway construction (1961) and cessation of brine pumping in 1971.

It is noted that a historic brine well is located adjacent to the north abutment. This could not be confirmed; however it is possible that its proximity may have accelerated differential settlement at the north abutment during the time period of active brine extraction, however there is no information directly linking this to brine abstraction and it may therefore just be a domestic well.

The available information suggests that the majority of significant settlements associated with the brine extraction within the area have ceased and subsidence occurring at present is steady and at low levels. Total settlements in the region of 1mm per year and minimal differential settlements have been recorded. It is considered likely that this low level of settlement will continue for the foreseeable future and are considered representative of settlements associated with natural salt solution processes. Available information suggests that the potential for brine extraction to re-commence in the area is very low, considering that salt and brine resources in Worcestershire are not considered likely to be workable or commercially attractive in the future (WCC, 2012).

3.4 *Geotechnical recommendation for structural repair work*

On the basis of the geotechnical assessment of the historical mining activities in the area and settlement records, it was recommended to incorporate an allowance within the structural repairs for ongoing total settlements of approximately 3 mm per year and potential differential settlement in the order of 1.5mm per year.

The potential for larger settlements to occur at this location should not be discounted due to the unpredictable nature of brine solution in the area. Large scale settlements may be sudden and significant; however the likelihood of this occurring is considered to be very low. For comparative purposes it can be considered that the risk of occurrence is no greater than is present in other areas of the network where the risk from historic coal or limestone mining exists.

It may be prudent to consider the installation of levelling stations with the instigation of ongoing annual settlement monitoring. The incorporation of jacks within the structural repair design may mitigate the potential for any future significant settlements or ongoing minor settlements. The level of jacking required at the bridge location could be defined by the levels of settlement recorded at the levelling station.

4 GROUND CONDITION AND MATERIAL PROPERTIES

4.1 *Ground condition*

Mercia Mudstone was found to be the main geological unit providing the bearing stratum required for the bridge pile foundation. As-built records have shown several investigation conducted on this stratum. The Mercia Mudstone was generally a soft becoming very stiff reddish brown cay of weathering grades IV to I. The investigation comprised various common field and laboratory tests to determine the material properties. Based on these ground investigations, interpretation was carried out to obtain the design parameters, of which a summary is presented in Table 1. Further details on this interpretation are provided in the next sections.

Table 1. Summary of the characteristic geotechnical parameters used in the assessment

Strata	Embankment Fill (Granular Class 1A)	Made Ground/ Fill materials (Clay)	Mercia Mudstone Group
Thickness, (m)	Approx. 4	1.35-1.5	>10
Unit Weight, γ (kN/m^3)	19	17-19	18-21
Shear strength	ϕ'=35^o	C_u=50 kPa	C_u= 70+48Z*
Young's Modulus, E (MPa)	E'_{av}=15-20 (effective average)	E_u=10-20 (undrained)	E_u=30 + 24Z* (undrained)
Axial Shear stiffness, G (MPa)	5.3-7.7	4.3-6.7	10+8Z*
Lateral Shear stiffness, G (MPa)	2.1-3.8	1.9-3.4	5+4Z*

* Z is depth in metre below the top of stratum

4.2 *Strength and stiffness*

Strength and stiffness properties are required for the calculation of pile capacity and group analysis. This section describe how these properties were derived and the values used in the analysis.

The shear strength was assessed for the cohesive fill/ clay and Mercia Mudstone based on laboratory and insitu Standard Penetration Tests (SPT's) where undrained cohesion (C_u) was determined using Stroud and Butlers (1975) estimation as indicated in Equation 1:

$$C_u = F_1\, N_{60} \quad (1)$$

where N_{60} = corrected SPT 'N' value and F_1 = a factor ranges between 5 and 6 for Mercia Mudstone as recommended by NHSE (1990).

The estimates of shear strength C_u against AOD (Above Ordnance Datum) were based on data obtained from four different boreholes in the vicinity of the bridge.

For Mercia Mudstone, Young's Modulus was estimated using empirical correlation with SPT and undrained shear strength C_u (Peck, 1974; Tomlinson, 2001). Vertical Shear Modulus (G_V) was then derived from Young's Modulus using the relationship below from elastic theory:

$$G_V = E/2(1 + \nu) \quad (2)$$

where ν = Poisson's Ratio, which may vary between 0.3 and 0.5 depending on material and loading condition. The lateral Shear Modulus (G_L) might be estimated using the correlation recommended by Randolph and Wroth (1978):

$$G_L = 0.5\, G_V \quad (3)$$

5 EVALUATION OF PILE CAPACITY

5.1 *Evaluation of pile axial bearing capacity*

The as-built geotechnical interpretative report of the bridge contains an outline estimation of the pile capacity and this stated that the piles should be taken into the Mercia Mudstone by approximately 6.5m and creating a socket of 1m in the grade I/ II Mercia Mudstone. Summary of the recommendation provided in the as-built report is given in Table 2.

The as-built design load for 0.43m diameter pile installed 6.5m into Mercia Mudstone was 60 Ton (circa 598) incorporating design safety factors of 1.5 shaft and 3.0 for the base. The above design recommendation has been partially based on the estimate of skin friction adopted by Chandler and Davis for different weathering grades of Mercia Mudstone (CIRIA Report 47, currently not available).

Table 2. As-built pile design recommendation

Properties	Value
Bored pile diameter, D	0.43m
Minimum Shaft length in Mercia Mudstone	6.5m including 1.00m socketed within grade I / II
Recommended Shaft skin friction	150-180 kN/m^2 for grade III / IV Mercia Mudstone
	250-280 kN/m^2 for grade III / II Mercia Mudstone

As part of the geotechnical assessment it was necessary to evaluate the pile bearing capacity to confirm the as-built estimation and examine the effect of new loading condition. The bored piles used to support the bridge are the most common type of non-displacement piles and many design methods have been well established in literature (Poulos and Davis, 1980). The ultimate pile load carrying capacity was calculated in this study applying undrained analysis method which is conventionally used for bored piles in clays/weak cohesive rocks. This method represents static analytical approach where the load carrying capacity of the pile consists of two components: shaft tangential resistance Q_s and base compressive resistance Q_b. Thus, the ultimate load-carrying capacity of a pile is given by Equation 2.

$$Q_u = Q_b + Q_s \quad (2)$$

Using the undrained shear strength parameters obtained from the testing associated with the boreholes (summarized in Table 1) it is possible to determine the shaft resistance Q_s of a pile using Norwest Holst Soil Engineering-NHSE (NHSE, 1990) for various Mercia Mudstone grades by applying Equation 3:

$$Q_s = p\, \Sigma\Delta L\ \alpha\, C_u \quad (3)$$

where p = pile perimeter; $\Sigma\Delta L$= pile length; C_u: undrained cohesion of soil around the pile shaft; and α = empirical adhesion factor adopted as 0.4, 0.3 and 0.2 for Mercia Mudstone grades IV, III and II respectively (Omer et al., 2003).

The base resistance Q_b is evaluated using undrained method explained by Fleming et al. (1992) and presented in Equation 5 (Braja, 2010):

$$Q_b = 9\, C_u\, A \quad (5)$$

where A = cross section area of the pile. The soil parameters used in the analysis are summarised in Table 1; in addition the pile was assumed of a plain bored type with circular cross section of a diameter of 0.43m (uniform along the pile length) and a total length of approximately 9m including 6.5m in Mercia Mudstone and the rest in the embankment fill (in accordance to as-built drawing).

According to the calculations the total ultimate capacity of a single pile (Q_u) was found to be approximately 990 kN. The working load, P_w, may be estimated from the ultimate pile capacities Q_s and Q_b divided by factor of safeties: F_s =1.5 and F_b= 3 as suggested in BS 8004 (1986) for bored cast in place piles:

$$P_w = \frac{Q_s}{F_s} + \frac{Q_b}{F_b} \quad (6)$$

Applying the factor of safeties explained above, the working load was found to be equal to 520 kN, which is about 11% smaller than the value reported in the as-built records (598 kN). This difference might be attributed to the method adopted for estimating the pile ultimate capacity (particularly shaft resistance Q_s). A recent study conducted on bored piles installed in Mercia Mudstone (Omer et al., 2003) has shown that the undrained method has underestimated the ultimate *shaft* resistance by as large as 50%. Nevertheless the bearing capacity identified from as-built, can be considered acceptable for the assessment.

5.2 *Negative skin friction*

Negative skin friction loads on pile foundations (also called downdrag force) is shear forces on pile due to downward soil movement relative to pile when the surrounding soil settles more than the pile. This force is developed when the pile is installed in hard stratum overlain by soft clay (Poulos and Davis, 1975). The soft soil may settle more than the pile, particularly if additional load is added on the surface e.g. another layer of embankment fill. The magnitude of the skin friction is proportional with thickness of the soft material and its rate tends to decrease with time as the soft clay consolidates (Poorooshasb et al., 1996).

Although the piles of Rashwood Bridge are constructed in hard ground materials, the risk of the development of negative skin friction along the softer material during the current project is trivial because of the following reasons:

- The as-built boreholes have shown that the soft clay, located on top of the harder stratum (Mercia Mudstone) is described as soft in limited places with a small thickness of less than 1.5m.
- Any possible negative skin friction which could have possibly developed after construction, should have finished by now, as soil tends to become stable with time after the completion of construction.
- The build-up of the downdrag would have been relieved by additional pile settlement arising from the increased loading on the piles.
- However skin friction may develop again if the effective stress condition is elevated e.g. by long term additional loads applied on the soil in the vicinity of pile foundation, which is unlikely to take place in the maintenance work considered in this study.

6 STRUCTURAL ANALYSIS OF PILE GROUP

6.1 *Details of the foundation system and loading conditions*

As-built construction drawings have shown that the south abutment bank-seat and wing walls are piled into Mercia Mudstone using a total of 24 bored reinforced concrete piles each having a circular cross section with 0.43m diameter. The piles are arranged in two groups, each group has a 1.2m thick pile cap. A plan view of the south abutment (only one pile cap is shown) with a cross-section of the bank-seat are shown in Figure 3 (see also Figure 2). The front row of the abutment piles are raked at 1 in 3 which is a procedure commonly used to resist lateral forces. The pier walls have spread footings and are founded in Mercia Mudstone.

To determine the loads transferred from the superstructure to the pile foundation system a structural analysis of the whole bridge (which is out of the scope of this paper) was conducted by the structural team considering three modelling situations: (i) before the maintenance work to assess load distribution associated with historical differential settlement, (ii) during the maintenance and bridge deck jacking to establish the required jacking loads to raise the bridge deck into a permanent position. The second model was also used to assess the capacity of the superstructure with the existing differential settlements combined with the proposed vertical jacking coexistent displacements. The deck end supports positions of the model was adjusted to reflect the temporary jacking scenario. (iii) The third model represents the bridge deck in its permanent position and was used to assess the superstructure capacity with the adjusted differential settlements after the bridge deck has been jacked. In this model the deck end support positions of the model were reverted to their existing locations to reflect the permanent position scenario.

Figure 4 summarises the loads (estimated from the structural analysis) transferred from the bridge superstructure to eleven (No.11) Bearings of the pile cap during jacking process and after completion (where bridge deck is at its permanent position). The structural analysis indicated (Figure 4) that the pile cap at Bearing 17 would experience the highest increase in loading during the jacking process, and therefore the piles located around this bearing are expected to experience the largest forces.

As the bridge beams are assumed to be simply supported at the south abutment the load transferred would be mainly vertical and therefore the moments about the reaction points are zero. However due to non-eccentric nature of this loading and interaction between soil-and vertical piles / raked piles, it would be expected to have some bending moments and lateral forces affecting the piles in addition to the axial loads (as discussed in the following section).

6.2 *Pile group analysis using Piglet*

The pile group was analysed using PIGLET (Randolph, 2006) which is widely used in construction industry to conduct structural analysis of bridge foundation (Hamza and Bellis, 2008). 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 dependent 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 long-term Young's modulus of the concrete of the piles was taken equal to 14.2 GPa in accordance with BD 44/95 (HA, 2013), the piles were assumed fixed to a rigid pile cap, and the loads were applied at the position of each Bearing or Jack. Based on the parameters recommended in Table 1, the lateral shear stiffness of the soil was considered equal to 0 at the base of pile cap level increasing linearly by approximately 1.67 MPa per metre depth.

Table 3 summarises the maximum and minimum axial load, lateral load, axial deflection, lateral deflection, and absolute values of moments predicted by PIGLET. The results indicated that all piles would experience compression stress and thus no pull-out force was identified. Less than 5mm of deflection with insignificant rotation of the pile cap were predicted by the software. However, the maximum axial load was noted to be relatively large on few piles.

Table 3. Maximum and minimum pile forces and deflections at cap level

	During jacking		After completion	
	Max	Min	Max	Min
Axial load (kN)	632	155	599	147
Lateral load (kN)	56	13	51	11
Absolute Moments (kN.m)	75	16	70	15
Axial deflection (mm)	4.8	3	4	2
Lateral deflection (mm)	4.5	3	4	2

The predicted maximum axial load on a single pile reached 632 kN, which exceeded the as-built allowable axial pile capacity P_w=599kN by 5.5%. However, given the short-term nature of the jacking process (7 days) and the factor of safety incorporated in the estimation of the working load (P_w) the pilling system was assumed to be capable to withsatnd the loading condition during and after jacking. These conclusions can be extrapolated to the assemssment of all other internal forces (lateral forces and bending moments) estimated within the piles as jacking would have smaller effect on these forces.

7 CONCLUSIONS AND RECOMMENDATIONS

A strategy used for geotechnical assessment of a bridge abutment supported by bored piles in Mercia Mudstone has been presented in this paper, discussing the stages implemented to ensure that safe working loads are applied during the maintenance work and no excessive deflection or rotation will result from the new temporary load distribution.

In addition to the geotechnical and structural analysis of the pile foundation, several challenges were discussed including the determination of reliable ground model, exploitation of the as-built data records and other existing data, assessment of the likely risk of future settlements at the bridge location associated with historical mining activities (brine pumping).

The elastic response of the pile group was analysed using PIGLET software, which has predicted an axial load exceeding the design load by up to 5.5% in few piles at the south abutment (the main location of the bridge maintenance work). However, given the short-term nature of the jacking process (1 week time) and the factor of safety incorporated in the estimation of the design load, the pilling system was assumed to be capable to withsatnd the loading condition during and after jacking. These conclusions can be extrapolated to the assemssment of all other internal forces (lateral forces and bending moments) estimated within the piles as jacking would have smaller effect on these forces.

The analysis of the pile group can be improved by using analytical tools that consider the non-linear mechanical behaviour of soil i.e. adopting strain and stress dependent stiffness. Nevertheless, the analysis

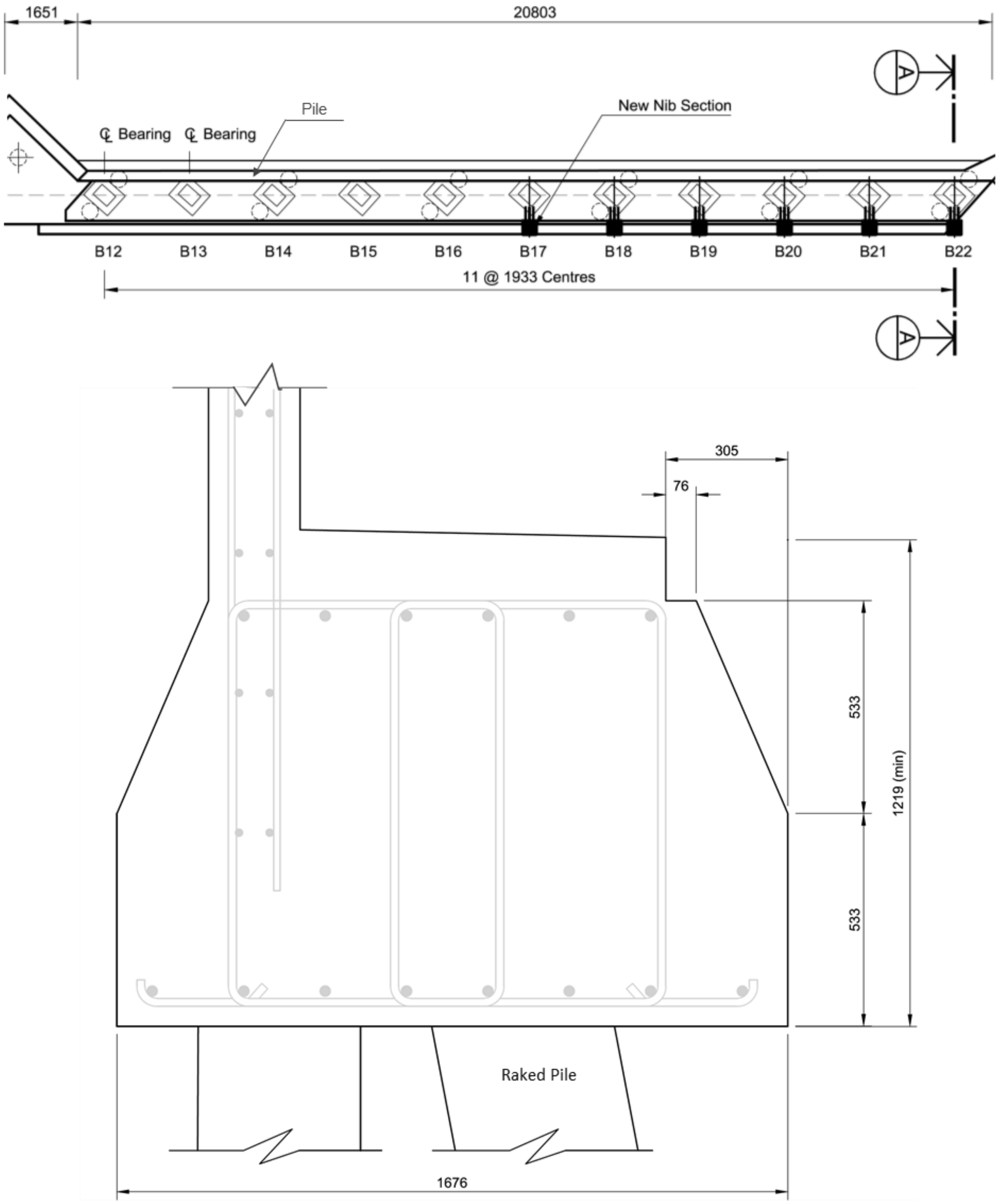

Figure 3. South abutment details: plan and section A-A (all dimensions are in mm).

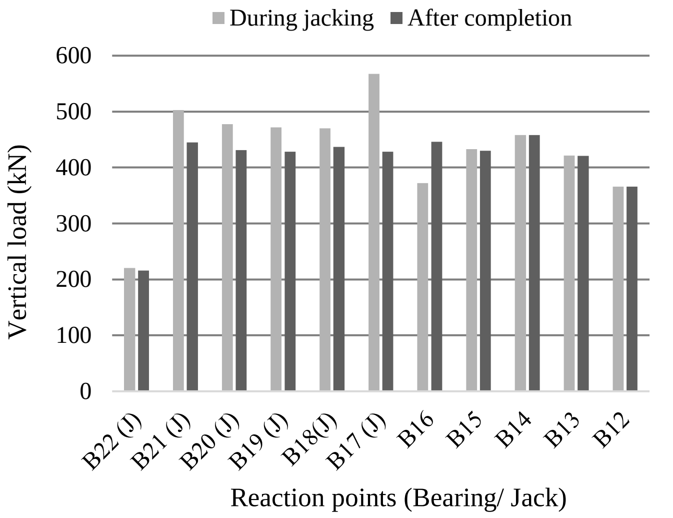

Figure 4. Estimated load on reaction points (Bearings and Jacks). The letter "J" denotes that jacking is taking place at the Bearing position. Figure 3 shows the locations of Bearings.

carried out was sufficient to satisfy the designers that the proposed method would not cause an unexpected behaviour to the existing bridge subs or superstructures.

8 AKNOWLEDGEMENT

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