The effects of tunnelling on piled structures on the CTRL

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ABSTRACT: The construction of the tunnels for the Channel Tunnel Rail Link (CTRL) project in London afforded the opportunity to obtain valuable case studies on the effects of tunnelling on piled foundations. The tunnel alignment was generally set to avoid interaction with overhead structure, but due to the depth of penetration of the piles supporting major bridges, tunnelling had to be carried out relatively close to the piled foundations of these bridges. This paper describes three case studies, briefly describing the assessments carried out prior to tunnelling under these bridges in London as well as selected monitoring results obtained as the tunnels advanced underneath the bridges.

1 INTRODUCTION

1.1 *Channel Tunnel Rail Link*

The tunnelling work on the Channel Tunnel Rail Link project in the greater London area was completed in March 2004. The work comprised the construction of approximately 35 km of 8.15 m external diameter bored tunnels under London. The tunnels were lined with a segmental precast fibre reinforced concrete lining, producing an internal tunnel diameter of 7.15 m. The tunnel alignment was set relatively deep below the surface under urban East London to minimise interaction with existing infrastructure. However, piled foundations for larger structures penetrate deep below the surface and tunnelling close to some of these was unavoidable. In particular, the works included tunnel excavation under a number of piled bridges.

Comprehensive assessments had to be carried out for all existing structures along the tunnel route to assess the effects that the construction of the large diameter tunnels might impose on these structures. The number of case studies in the literature reporting on the effects of tunnel construction on nearby piled structures are however limited, so that significant uncertainty existed regarding tunnel-pile interaction prior to the commencement of the project. Available literature is generally limited to the results of physical and numerical modelling of the problem (e.g. Jacobsz *et al.*, 2004 and Mroueh & Shahrour, 2002).

1.2 *Expected tunnelling-induced pile behaviour*

Based on centrifuge modelling of piles carrying significant base and shaft loads, Jacobsz *et al.* (2004) described a zone of influence around a tunnel where piles, with their bases located within this zone, could suffer large settlements depending on the volume loss incurred. Piles with their bases located within this zone suffered a reduction in the load mobilised on their bases with increasing volume loss due to stress relief caused by the tunnel, with load transfer occurring to the pile shafts to maintain equilibrium. During this stage piles underwent little settlement, settling by an amount approximately equal to the greenfield surface settlement at the location of the pile, both in sands (see Jacobsz *et al.*, 2004) and in clays (see Loganathan *et al.*, 2000). Once volume loss had increased sufficiently to reduce the base load so much that the maximum shaft capacity is mobilised, the piles underwent large and sudden settlements.

Pile shafts carrying significant friction loads exert significant stresses on the surrounding soils. Due to the stress-dependant stiffness of soil, the loads exerted by the piles result in an increase in soil stiffness, so that a subsurface settlement profile develops that differs from the greenfield situation near the piles. This subsurface settlement is more uniform with depth close to the piles, with settlement approximately equal to the greenfield surface settlement.

If a constant load is maintained at the cap of a pile located within the zone of influence of a tunnel, a reduction in the pile base load will result in the mobilisation of positive shaft friction. With positive friction acting on the pile shaft the soil in contact with the shaft will not settle more than the pile, as this would cause negative friction.

This paper describes the settlement predictions carried out for three bridges along the route of the CTRL tunnels in London. All three bridges are located along Contract 250 of the CTRL which extended from Dagenham in East London to Barrington Road Ventilation Shaft in Barking. The first bridge, Renwick Road Bridge, is supported on end-bearing piles, while the other two bridges, Ripple Road Flyover and the A406 Viaduct, are supported on friction piles. The construction of the CTRL tunnels underneath the foundation of these bridges afforded the opportunity for the predicted settlement to be compared to actual behaviour. The results are presented here.

2 CASE STUDIES

2.1 *Renwick road bridge (end-bearing piles)*

2.1.1 *Bridge description and underlying geology*

This road bridge spans the London-Tillbury-Southend railway line and is located in Dagenham, East London. A simplified cross-section along the bridge centreline is presented in Figure 1 showing the bridge deck, piers, foundations and the local stratigraphy.

The bridge is supported on driven piles, end-bearing in the Terrace Gravels. The dense Terrace Gravels are overlain by a 8 m to 12 m thick deposit of saturated very soft alluvial silts, clays and organic materials. Due to the consistency of the alluvium, the piles possess negligible shaft capacity. The Terrace Gravels are underlain by London Clay to considerable depth through which the CTRL tunnels were advanced.

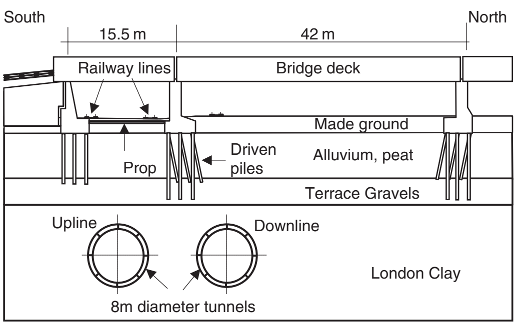

Figure 1. Renwick Road Bridge.

2.1.2 *Monitoring*

An extensive programme was carried out during the tunnelling phase of the project to monitor tunnelling-induced ground settlement. Surface settlement was monitored by means of precise level surveying at regular intervals in the vicinity of the tunnel boring machines (TBM), but at less frequent intervals further from the TBMs. A number of settlement monitoring arrays were located near the bridge.

In addition to surface settlement, the movement of the bridge structure was also monitored during the passage of the TBMs by means of an automatic total station which monitored a number of reflective targets mounted on the bridge.

2.1.3 *Prediction of tunnelling impact*

The piles supporting Renwick Road Bridge are end-bearing and due to the soft materials surrounding their shafts it was not possible to mobilise significant friction loads on the shafts during volume loss. For the purposes of assessing tunnelling impact it was therefore assumed that the movement of the piles would be equal to the settlement that would occur at the depth of the end-bearing pile bases. These movements were calculated for a range of volume loss values using the greenfield model proposed by New & Bowers (1994), providing similar predictions to the empirical model for clays by Mair *et al*. (1993).

Due to the piles being positioned at increasing distances from the tunnel centreline, significant differential settlement of the pier and abutment bases appeared likely. This would have resulted in a certain amount of pier rotation towards the tunnels, possibly resulting in some compression of the pre-stressed bridge deck which would have impacted detrimentally on the deck's structural integrity.

As-built drawings obtained for Renwick Road Bridge however showed that a number of concrete props were located immediately below the rail level between the foundations of the bridge pier and southern abutment (see Figure 1). An equilibrium calculation, treating the bridge pier and abutment as free bodies, disregarding loads from the ground, showed that rotation of the bridge piers could be prevented if the props were able to carry an axial compressive load equal to only a small fraction of their apparent capacities. An inspection of the props was carried out and they were judged to be of adequate strength to prevent rotation of the bridge piers, even should considerable loads be imposed by the soil.

The predicted effects of the construction of the CTRL upline tunnel on the bridge was therefore limited to settlement, the magnitude of which would have depended on the volume loss achieved under the bridge. Passage of the downline, which followed a number of months behind the upline, would have induced some settlement and rotation of the pier and relatively little impact on the southern bridge abutment. The settlement recorded during the passage of the TBMs are compared to the predictions below.

2.1.4 *TBM operation near bridge*

Care was taken whenever the advancing tunnel faces approached sensitive structures to ensure that ground movements were kept to a minimum. This was achieved by applying adequate pressures on the TBM face to support the overlying ground while tunnelling.

2.1.5 *Observations*

The surface settlement recorded after the passage of the upline TBM under the bridge at an offset of 2 m from the tunnel centreline was about 5.5 mm, giving a volume loss of 0.28% assuming a Gaussian settlement profile. The settlement calculated using the method by New & Bowers (1994) at the depth of the pile bases at the average offset of the southern abutment was 6.8 mm which corresponds very well with the 7 mm of settlement measured on the bridge abutment. This confirms that the settlement of the end-bearing piles were similar to the greenfield settlement calculated at the pile bases.

Passage of the downline TBM under the bridge was associated with larger settlement and fairly atypical ground movements, deviating from the Gaussian curve, were observed. The volume loss near the bridge was variable, but was generally just in excess of 1%. This was probably the result of non-uniform ground conditions, characterized by pockets of very soft consistency within the alluvium, which often caused problems in controlling surface movements in the area leading up to Renwick Road Bridge. Figure 2 presents the settlement records measured at four locations on the pier between the tunnels using a robotic total station. Also shown is the settlement record from a monitoring point on the surface adjacent to the bridge in line with the pier. The surface settlement immediately after passage of the TBM was about 18 mm, increasing to 20 mm as consolidation occurred, with the pier settling somewhat less. This corresponds to predicted ground movements that the settlement at depth is somewhat less than the surface settlement at an offset equal to that of the pier from the tunnel. Again it appears that the piles settled by the same amount as the soil at the depth of the pile bases.

2.2 *Ripple road flyover (friction piles)*

2.2.1 *Bridge description and underlying geology*

Ripple Road Flyover is located in Barking, east London and also spans the London-Tillbury-Southend railway line. A simplified cross-section of the bridge is presented in Figure 3. Each bridge pier is supported on a total of 31 piles of which 12 are vertical and 19 raking. The vertical piles were of driven-cast-in situ construction, while the raking piles were bored.

The piles extend to a depth of approximately 25 m below surface, possibly into the highly variable Harwich formation. The majority of the pile shafts are in the London Clay, which is overlain by approximately 4 m of Terrace Gravels. The surface is underlain by made ground varying in thickness between 1 m and 2 m. The CTRL tunnels are located in the sandy materials of the Lambeth Group.

Due to limited as-built records, the exact depth to which the piles extended was not known. A magnetometry survey, capable of registering the steel driving shoes on the driven piles, was undertaken to determine the depths of piles potentially interfering with the tunnel alignment. Based on the survey the pile base closest to the tunnel was estimated to be approximately only 1 m clear of the tunnel lining. The fact that the TBMs did not clash with any piles confirmed the success of the magnetometry survey.

2.2.2 *Monitoring*

Ground settlement was monitored using precise levelling and the bridge by means of a robotic total station reading reflective targets.

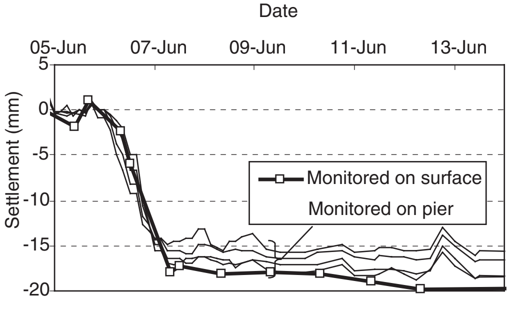

Figure 2. Settlement at Renwick Road Bridge during passage of downline TBM.

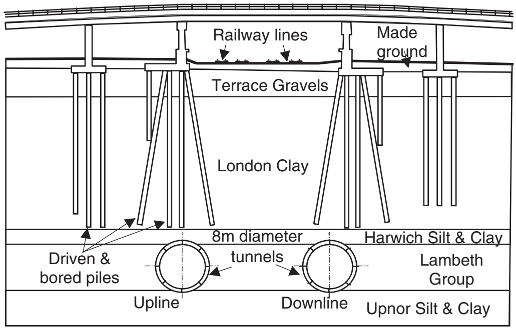

Figure 3. Ripple Road Flyover.

2.2.3 *Assessment of tunnelling impact*
The pile bases of Ripple Road Flyover all fall within a zone of influence with respect to the tunnels where a significant reduction in the mobilised base load is possible due to volume loss (Jacobsz *et al*., 2004). Due to the uncertainties regarding the effects on the bridge of tunnelling this close to the piles, a conservative approach was adopted in the design of mitigation works.

In contrast to the raking bored piles which were assumed to be carrying small base loads, the vertical driven-cast-in situ piles were expected to be carrying potentially significant base loads. For a driven pile the maximum base load that could be "locked in" by negative shaft friction after construction would be equal to the undrained base capacity. Loading of the pier during construction and during the bridge's operational life would only have resulted in small settlements, so that it was assumed that additional load would not have been mobilised on the pile bases. Mitigation works were therefore devised which entailed grouting around the piles within the Terrace Gravel so that additional shaft capacity could be mobilised there to compensate for the loss of a mobilised base load equal to the combined undrained base capacity of the 12 driven piles.

In addition to enhanced shaft capacity within the Terrace Gravels, the grouting would have formed a "near-solid raft" underneath the pile cap, enabling the foundation to bear directly onto the London Clay. Given this solution the grouted piled bridge piers were expected to settle by the same amount as the ground surface.

2.2.4 *TBM operation near bridge*
The upline TBM passed under the bridge between 11 and 17 September 2003. As the TBM approached the bridge, a relatively high face pressure was maintained, resulting in about 2 mm of heave ahead of the face (see Figure 4). The TBM was stopped under the bridge for nearly two days by which time approximately 4 mm of settlement had occurred at the surface. As the TBM moved on some more settlement occurred, resulting in a total settlement of between 8 mm and 10 mm, i.e. a volume loss of about 0.6%.

The downline TBM passed under the bridge between 6 and 10 October 2003 causing approximately 11 mm of settlement just before the bridge (see Figure 5), i.e. a volume loss of 0.8%.

2.2.5 *Observations*
The surface settlements observed near the bridge are presented above.

The settlement of the bridge pier above the upline tunnel was about 8 mm and that of the pier above the downline tunnel 10 mm. These settlement are similar to the surface settlement, as would be expected given the mitigation works carried out.

2.3 *A406 viaduct (friction piles)*

2.3.1 *Bridge description and underlying geology*
The A406 viaduct is a large bridge located in Barking, East London, spanning across the Gospel Oak to Barking and the District Line railway lines and other infrastructure. The bridge and the underlying geology are illustrated in Figure 6. Also shown in Figure 6 are

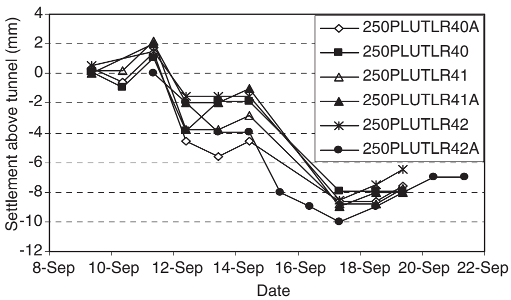

Figure 4. Settlement above upline TBM at Ripple Road.

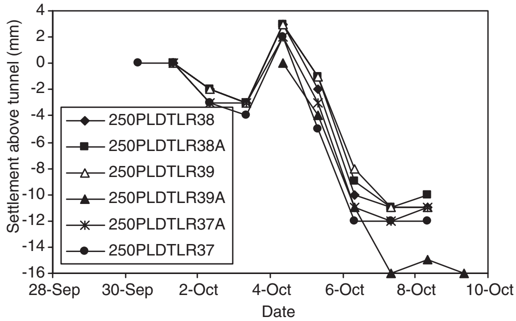

Figure 5. Settlement above downline at Ripple Road.

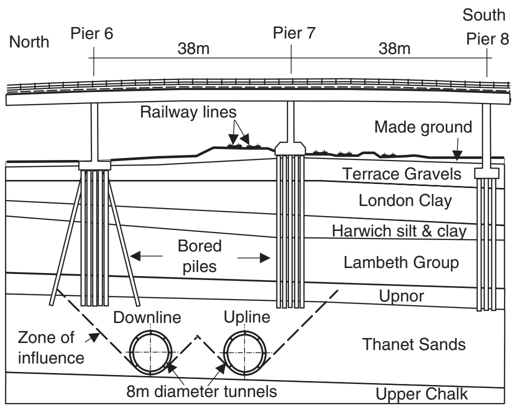

Figure 6. The A406 viaduct.

zones of influence around the tunnels in which a potential for tunnelling-induced pile base load reduction exists (Jacobsz *et al.*, 2004).

The concrete bridge piers support a composite deck comprising continuous longitudinal and transverse steel plate girders supporting a reinforced concrete deck slab (Zanardo *et al.*, 2004). Two bridge piers, i.e. Piers 6 and 7, are affected by the CTRL tunnels. The piers are supported on 23.5 m long bored piles, extending through various geological strata as illustrated. Pier 6 is supported by two pile groups of 17 × 750 mm diameter piles each and Pier 7 by one group of 50 × 650 mm diameter piles.

The CTRL tunnels were constructed in the Thanet Sands with the tunnel crowns approximately 4 m below the level of the bored pile bases.

2.3.2 *Monitoring*

Ground settlement was monitored using precise levelling and the bridge by means of a robotic total station reading reflective targets.

2.3.3 *Assessment of tunnelling impact*

The A406 viaduct is supported by friction piles. As the bored piled foundations supporting the bridge would not have undergone large settlement since construction, the piles were assumed to be friction bearing, carrying small base loads. The transfer of load from the pile base to the shaft was therefore not considered to be the primary mechanism that would control behaviour as in the case of Ripple Road Flyover without mitigation. Instead it was assumed that the piles would act as slender elastic members, deforming with the surrounding ground. The bridge owner expressed concern about the possibility that tunnelling-induced ground movements might result in overstressing of the piles. The effects that these movements would have had on the structural integrity of the piles were investigated.

The distribution of vertical movement along the pile shafts of the pile groups was calculated using the method by New and Bowers. Assuming no slip between the piles and the surrounding ground, vertical displacements were converted to vertical strains. Horizontal ground movements were calculated and converted to bending strains acting on the piles. The strains resulting from vertical and horizontal ground movements were added and evaluated. Figure 7 presents the strain distribution thus obtained for the piles group supporting Pier 7. The evaluation was based on 1% volume loss, the contractual requirement expected from the CTRL tunnel contractors.

The piles were predicted to be generally in compression due to potential downdrag from the tunnelling-induced ground movements, with bending responsible for the variation in strain as represented by the double lines for each pile in Figure 7. Bending-induced strains were small compared to the effects from vertical movements.

The magnitude of the maximum induced strains, although significant, was substantially below the ultimate strain for concrete (i.e. typically 2000 με to 3500 με). The strains were based on the assumption that the pile group would be forced to move with the greenfield ground movements. The likelihood of this occurring was further investigated as described below.

From the vertical strain distribution in the piles, vertical stress distributions were calculated from which the distribution of shear stresses on the pile shaft could be determined. Assuming that shear stresses on the pile shaft were related to normal stresses via a friction coefficient, very large normal stresses were calculated (in places in excess of 1 MPa), especially near the pile bases. Due to the stress-dependent stiffness of soils, such stresses would result in considerable stiffening of the ground so that a greenfield distribution would not occur near the piles. Actual movement should be less than greenfield movements, i.e. the assumption of greenfield condition is conservative. The fact that greenfield movement at 1% volume loss would not have overstressed the pile shafts showed that tunnelling would not have resulted in overstressing of the pile shafts. In addition, the magnitude of settlement that the bridge would have undergone at 1% volume loss had to be assessed, as these movements were required to assess the effects of tunnelling on the bridge structure (see Zanardo *et al.*, 2004).

Centrifuge tests showed that at volume losses typically encountered in practice (up to 1.5%) piles with reserve shaft capacity settle by an amount similar to the ground surface. In a study using full-scale piles above the CTRL tunnels in East London it was observed that tunnelling-induced pile settlement was similar to the surface settlement at low volume losses (Selemetas *et al.*, 2005).

The foundation of Pier 7 was therefore expected to settle with the ground surrounding the piles by an amount similar to the greenfield surface settlement.

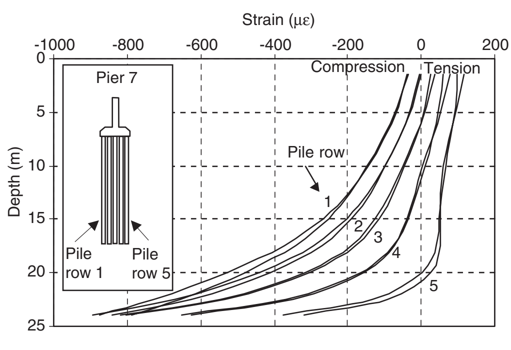

Figure 7. Strain distributions in pile group at Pier 7.

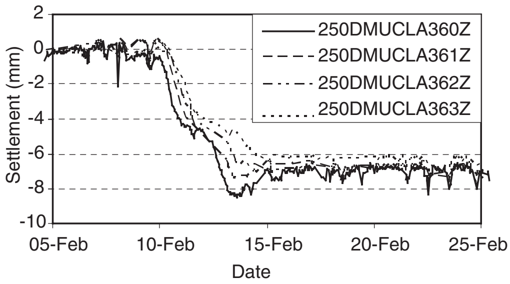

Figure 8. Surface settlements above the upline tunnel shoulder at the A406 viaduct.

2.3.4 *Observations*

Both TBMs passed in close succession under the bridge so that it is difficult to distinguish the effects imposed by the individual tunnels. The only surface settlement available were recorded above the upline tunnel's souther shoulder (i.e. 7 mm), immediately adjacent to Pier 7 (see Figure 8). The average settlement of Pier 7 was 6.5 mm, practically identical to that of the surface around it. Pier 6 settled by an average of 4.5 mm. The combined volume loss caused by the two tunnels was approximately 0.3%.

3 DISCUSSION

3.1 *Case studies*

In the case of Renwick Road Bridge supported on end-bearing piles, the settlement of the bridge could be predicted well by assuming pile settlement to be equal to the subsurface greenfield settlement at the depth of the pile bases.

Mitigation works at Ripple Road Flyover resulted in the bridge undergoing movements which were restricted to that of the surface. In hindsight, mitigation works prior to tunnelling under Ripple Road Flyover were not required. An analysis of the loads acting on the bridge and the capacities of the piles showed that the total factored static and live bridge loads could be supported by mobilising shaft friction of only 12.8 kPa. In order to accommodate the total base load that can be lost due to tunnelling an additional 4.5 kPa of shaft friction would be required, i.e. giving a total shaft load of 17.3 kPa. This is a low shaft friction value for London Clay, indicating that the piles possess a large reserve (i.e. unmobilised) shaft capacity. This has been confirmed by recalculation of the pile capacity. As the shaft capacity can comfortably accommodate the total load that can be lost at the base, large pile settlements would not occur, negating the need of mitigation works.

The foundations of the A406 bridge, where no mitigation works were carried out, moved by the same amount as the ground surface. Assuming the piles to be subjected to subsurface movements equal to the greenfield situation proved a conservative means for analysing the effects of tunnelling-induced ground movements on the structural integrity of the piles.

3.2 *Mechanism*

In addition to the mechanism controlling the behaviour of piles carrying significant case and shaft loads described in paragraph 1.2 (i.e. the transfer of load from the base to the shaft), it appears that the following applies to friction piles.

Friction piles carrying insignificant base loads are expected to move and deform with the surrounding ground, at least at the relatively small volume losses that typically occur in practice. However, the ground movements are different from the greenfield subsurface movements as explained in paragraph 1.2 and the discussion on the A406 bridge.

4 CONCLUSIONS

The following conclusions are presented:

At small volume losses end-bearing piles settle by an amount equal to the greenfield settlement at the pile base.

Friction piles change the greenfield subsurface settlement profile and settle by an amount similar to the greenfield surface settlement.

Care must be taken to ensure that volume losses are kept small in areas where load cannot be distributed from the pile base to the shaft, e.g. on piles in very soft clay end-bearing in sand.

Assuming friction piles to deform with the surrounding soil provides a conservative means for assessing the effects of tunnelling-induced ground movement on the structural integrity of the piles.

When assessing the effects of tunnelling on piled foundations a re-assessment of pile capacity should be carried out and compared to the maximum load that the foundation have to support. Often large factors of safety will be found and in many cases mitigation works will not be required because, provided that the piles shaft capacities are not exceeded, they will simply settle and flex with the ground surrounding them.

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