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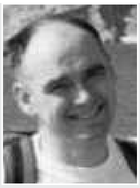

Charlotte A. Jones
Geotechnical Engineer, Buro Happold, Bath (formerly MSc student, University of Leeds, UK)

Douglas I. Stewart
Senior Lecturer, School of Civil Engineering, University of Leeds, UK

Christopher J. Danilewicz
Principal Geotechnical Engineer, Halcrow Group Limited, UK

Institution of Civil Engineers

Bridge distress caused by approach embankment settlement

C. A. Jones, MSci, MSc, FGS, D. I. Stewart, MPhil, PhD and C. J. Danilewicz, MA, MSc, CGeol, FGS

Surtees Bridge, which carries the A66(T) over the River Tees near Thornaby-on-Tees in the UK, has been showing signs of distress that predate its opening in 1981. Subsequent investigations have shown that the bridge distress is related to unexpectedly large settlement of the eastern approach embankment. Recent ground investigations prompted by a proposed widening of the river crossing have produced many new data on the alluvial deposits underlying the site, and explain why embankment settlement was so much larger than originally anticipated. Comparison of the geotechnical parameters obtained from the original and more recent ground investigations suggests that the original investigation significantly underestimated the thickness of an alluvial clay layer underlying the site, and that its coefficient of consolidation was overestimated. Settlement analyses using geotechnical data from the original ground investigations predict moderate embankment settlements occurring principally during construction. Settlement analyses based on all the available data predict far larger embankment settlements occurring over extended time periods. The latter analyses predict an embankment settlement similar to that observed and of sufficient magnitude to cause the observed lateral displacement of the bridge due to lateral loading of its piled foundation.

NOTATION

Δq	embankment load
C_c	compression index (slope of one-dimensional normal compression line on graph of e against $\log \sigma'_v$)
C_r	recompression index (slope of rebound line on graph of e against $\log \sigma'_v$)
c_u	undrained shear strength
C_v	coefficient of consolidation
d	pile diameter
e	void ratio
e_0	initial value of void ratio
E_p	Young's modulus of pile
E_s	representative stiffness of soft clay layer
h_s	thickness of soft clay layer
I_p	moment of inertia of a pile
K_R	relative soil–pile stiffness (defined by equation 2(a))
L_{eq}	equivalent length of pile between points of fixity
mOD	elevation measured in metres relative to Ordnance Datum
m_v	coefficient of volume compressibility
Δy	horizontal deflection of pile cap
Δy_q	non-dimensional pile cap deflection (defined by equation 2(b))
σ'_v	vertical effective stress

1. INTRODUCTION

The foundations for highway bridges must satisfy demanding movement criteria if a bridge is to perform satisfactorily over its full design life. Many highway bridge foundations, however, fail to meet these limits. A survey in the 1980s of around 300 bridges in the United States found that a third had undergone intolerable foundation movements.[1] Movement of bridge supports can affect all aspects of bridge performance, from visual appearance to vehicle ride quality, and in extreme cases can affect the structural integrity of the bridge.

Most common types of highway bridge can tolerate reasonable magnitudes of total and differential vertical settlement of their supports without serious distress. For example, a longitudinal angular distortion (differential settlement/span length) of 0·004 is likely to be tolerable for a continuous bridge. Horizontal movements, however, are much more damaging, and it is usually recommended that horizontal movements be limited to less than 38 mm.[1,2] Limiting the horizontal movement of bridge abutments founded on soft soil is a challenge to designers. The use of piled foundations is generally effective at limiting vertical movement, particularly when end-bearing onto a firm stratum or rock. Unfortunately, piles constructed through soft soil may be subject to lateral loads and movements as a result of time-dependent deformation of the soil underlying the approach embankments.[3,4]

Design guidance tends to focus on movements of the bridge foundations, with less attention being paid to foundation conditions beneath the approach embankments. Differential settlement between a bridge abutment and approach embankment can be damaging to the road pavement, although such damage is easier to remedy than damage to the bridge superstructure. A survey of several hundred highway bridges, carried out in Kentucky in 1968, found that about 80% had required some form of maintenance action to remedy faults caused by differential settlement.[5] Piling a bridge abutment to limit movement of the bridge superstructure would tend to accentuate this problem.

This paper reports on the geotechnical performance of a highway bridge that carries four lanes of traffic across the River Tees. Site investigations associated with plans to widen the river crossing to cope with increased traffic volumes have revealed that this 25-year-old bridge has suffered significant settlement-related distress. This distress is caused primarily by settlement of one approach embankment and the resulting movement of the associated bridge abutment. Based on limited construction information it is understood that the western approach embankment was built 18–24 months prior to bridge construction, and surcharged to increase the rate of settlement. It consequently shows no sign of recent movement. The eastern embankment, however, was built contemporaneously with the bridge construction, with no measures to increase the rate of settlement, and has subsequently undergone large settlements. This paper investigates the reasons why movements of the eastern bridge abutment are causing distress to the bridge, and why these movements were not anticipated during bridge design.

2. SITE DESCRIPTION

Surtees Bridge carries the A66(T) over the River Tees, approximately 1 km to the south-west of Thornaby railway station, Thornaby-on-Tees, Cleveland, at National Grid reference NZ 446 178. Surtees Bridge is located upstream and south of two railway bridges, one of which carries the Darlington to Saltburn railway line across the River Tees (Fig. 1). Historical Ordnance Survey maps show that Victorian railway sidings, related to iron works, covered much of the land now occupied by the eastern approach embankment of Surtees Bridge. These sidings were constructed on ground raised to the level of the railway, apart from one spur that ran down through the filled ground to a quay on the river (Fig. 2). Some land between the main railway line and the siding to the landing stage was used for allotments.

Construction of the bridge took place between 1980 and 1982, except for the western approach embankment, which was built under an advanced works contract in early 1978. This

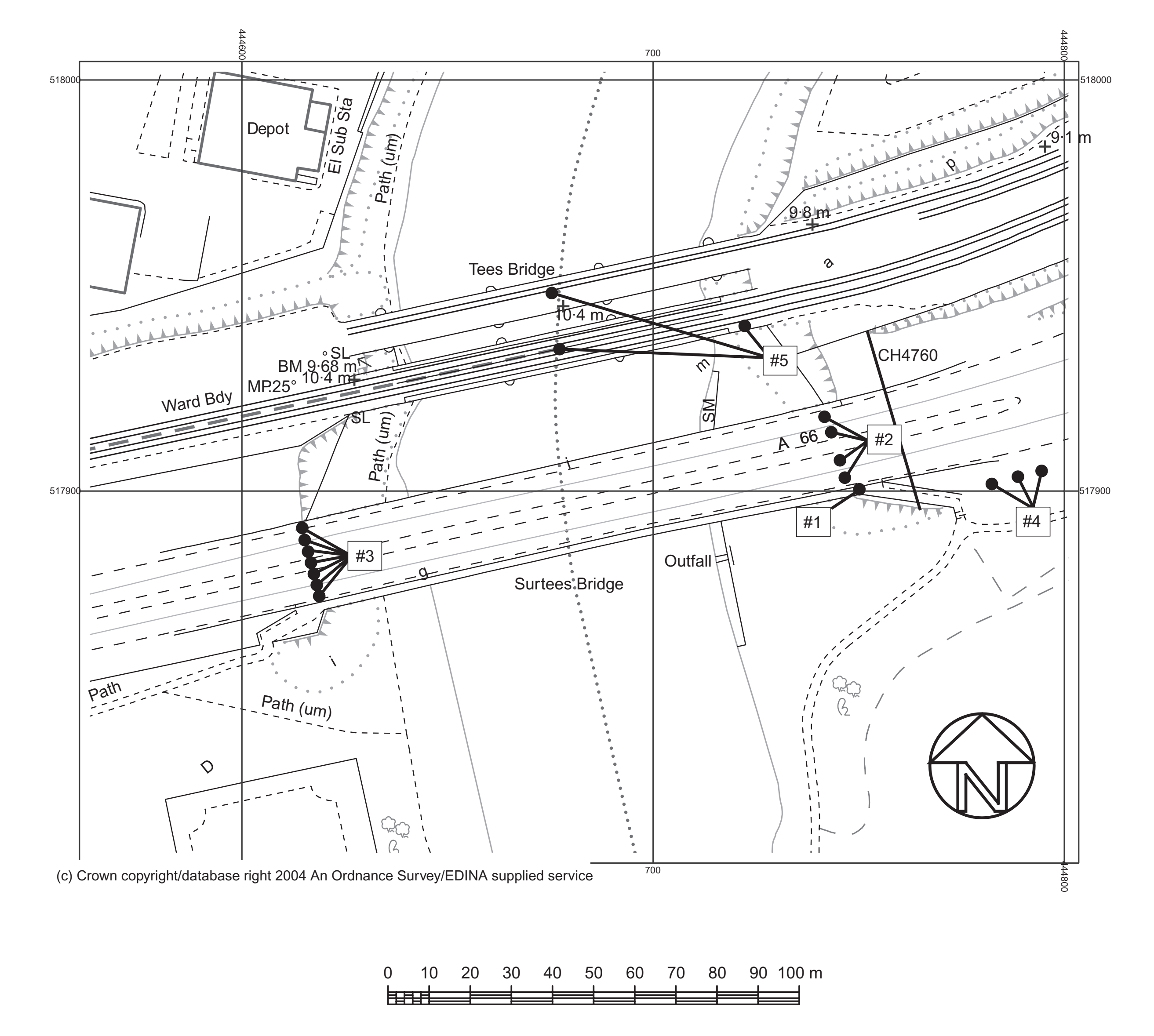

Fig. 1. Site plan (drawn using Edina Digimap Carto).

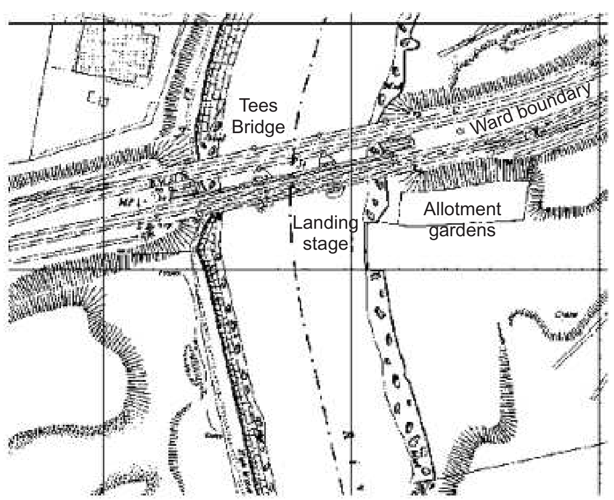

Fig. 2. Historical map showing the site in 1962 (excerpt from National Grid 1:1250).

embankment was raised in three lifts, surcharged, and allowed to settle for 18–24 months prior to implementation of the main construction contract. Information on the construction of the eastern approach embankment is more limited, but the absence of an advanced works contract for the eastern approach embankment indicates that it was built contemporaneously with the bridge abutments, whose construction started in 1980. The railway access to the riverside quay was infilled, but it is not known how much of the old siding fill was incorporated into the eastern approach embankment.

3. GEOLOGICAL SITE CHARACTERISATION

At least six separate ground investigations have been undertaken at the site of Surtees Bridge since 1973: two to facilitate bridge construction[6,7] and four more recently to assess the reasons for continued settlement of the approach embankment and their implications for bridge widening[8–11] (see Table 1 for a full chronology). During these investigations a total of 49 boreholes have been advanced in the vicinity of Surtees Bridge, together with non-sampling CPT investigative methods. Uncorrected SPT data acquired from borings immediately around the eastern abutment, presented in Fig. 3, are a guide to the ground conditions beneath this abutment. Based on all the ground investigation data, published geological maps[12] and geological studies of the region,[13] a ground model has been developed for the site (see Fig. 4).

The recent geology at the site comprises alluvial deposits (brown alluvium and grey alluvium[13]), which infill a valley that was cut through Devensian glacial deposits by post-glacial erosion. The more recent brown alluvium occurs principally within the present river channel, with grey alluvium underlying the brown alluvium and extending beneath the approach embankments. The buried early post-glacial topography affects the level of the top of the glacial deposits across the site. Beneath the bridge and much of the eastern approach embankment the glacial deposits are encountered at approximately −15 mOD, but they rise to near ground level to the east and west. The underlying Triassic Sherwood sandstone bedrock is generally encountered at a consistent level of −22 mOD across the site. Made ground, including former railway siding fill and blast furnace waste, is found beneath the bridge approach embankments. Engineering descriptions of the main soil horizons, together with their reduced levels at chainage 4760 (whose position is shown in Figs 1 and 4), are presented in Table 2.

There is no record of erosion, other than reworking by the river, at the site after the alluvium was deposited. General filling to form the level of the railway sidings would, over time, have increased the effective stresses in the underlying deposits. This fill is very extensive, and any removal during construction of the bridge and approach embankments is expected to have been relatively minor. Groundwater level at the site is dominated by the river, which was tidal until completion of the Tees Barrage in 1995. Tidal variations in the water table under the embankment are likely to have been relatively small, and thus it is reasonable to assume that the alluvial deposits at the site of Surtees Bridge are lightly overconsolidated.

The original ground investigation for Surtees Bridge comprised 10 boreholes and one Delft sample hole (three boreholes near the western abutment, five in the river channel, and two boreholes and the Delft sample hole near the eastern abutment). The 1976 Soil Mechanics report[7] presents a longitudinal geological section along the line of the road based on these data. The relevant portion is reproduced in Fig. 5. Prior to construction it was thought that a layer of alluvial clays approximately 7.5 m thick was present under the eastern abutment. The alluvial clay was thought to be underlain by 9 m of alluvial silty sand resting on glacial deposits (such a ground model appears to be supported by the limited pre-construction SPT data shown in Fig. 3(a), but not by the more extensive post-construction SPT data shown in Fig. 3(b)). The pre-construction ground model of the western approach embankment indicated that it was underlain by up to 14 m of alluvial clays. This significant difference in the thickness of the alluvial clay layer in the east and west bank models may explain why a decision was made to construct the western approach embankment as advanced works prior to bridge construction.

4. DETAILS OF SURTEES BRIDGE

Surtees Bridge comprises a continuous deck supported by four intermediate piers (see Fig. 4). The distance between abutments is approximately 125 m. Road level is approximately 11 mOD across both approach embankments and the bridge deck. Paved earth slopes have been constructed in front of the abutments, and these slope down to the river banks. There is a wide river bank under the eastern end of the bridge at approximately 5·5 mOD, and a narrower bank under the western end of the bridge at approximately 3 mOD. River level is approximately 2·7 mOD and is maintained at that level by the Tees Barrage downstream of the site.

The four intermediate bridge piers (labelled B to E in Fig. 4) are each supported by four bored, cast in situ, 1·35 m diameter piles founded within the underlying bedrock. The western abutment (labelled A in Fig. 4) is a bank seat set upon the western approach embankment. Formation level of

Pre-construction		
Preliminary ground investigation[6]	1973	One Delft continuous sampler exploratory hole in the area now occupied by the eastern abutment.
Main pre-construction ground investigation[7]	1975–1976	Four boreholes (two in the river) and one field vane test near to the eastern abutment. Three other boreholes in the river. Three boreholes and one field vane near the western abutment.
During construction		
Western embankment constructed under an advanced works contract	Feb–May 1978	Embankment raised in three lifts to 4·3, 6·5 and 8·0 m above ground level.
Monitoring of western embankment	Jan 1979	Extensometers indicate up to 400 mm of consolidation settlement. Inclinometers show 95 mm and 140 mm of lateral movement extending to 4 m below original ground level.
Start of main construction contract	1980	
Observation during construction[18]	Nov 1980	Between 70 mm and 115 mm of ground settlement in the vicinity of pier E. Forward displacement of the pier is recorded (40 mm of forward movement after piling was completed).
Bridge open to traffic	1981	
Post-construction		
Principal inspection report	1982	Highlighted settlement behind the east abutment and recommended that this should be monitored and made up as necessary. Noted that bearing deflector plates on pier E were deformed.
Principal inspection report	1986	Cracking noted in the crossheads of piers B, C and D (B and C having been repaired). Bearings at abutment A show cracking of supporting plinths and deformation of deflector plates. Bearings on piers B, C and D show cracking of supporting plinths but no deformation of the deflector plates. Bearings on pier E are as per abutment A with some rusting on the soffits of the sliding surfaces.
Aerial photograph	May 1991	Differences in pavement colour indicate that the road surface had been renewed on both bridge approaches.*
Ground investigation[8]	1992	One objective was to assess the reasons for continuing settlement of the embankments leading up to the bridge and the implications for the proposed widening.
Surtees Bridge Category III check assessment of existing structure	1993	Minutes from a 1992 meeting state that settlement of the eastern embankment may have exceeded 490 mm. Deflection of pier C and D crossheads inferred from positional survey.
Aerial photograph	July 1995	Extensive resurfacing of western approach embankment and over 100 m of eastern approach embankment.*
Walkover survey	2000	Pier E was reported to be visibly 'out of plumb'.
Ground investigation[9]	2000	One objective was to monitor for ground movements in the east and west abutments.
Monitoring of inclinometers installed in 2000	Aug 2001	Deflections compatible with settlement of alluvial deposits were recorded.
Ground investigation[10]	2002	Aims included investigating excessive and continuing settlements of the eastern abutment.
Bridge expansion joint repaired	2003	Repair instructed following inspection. Cause and nature of damage not known to authors.
Ground investigation[11]	2004	Introduction states that monitoring indicates that ground movement was still occurring.
Bridge expansion joint repaired	2005	Additional repairs required due to inadequate work during 2003 repair.

* The road has been resurfaced periodically as part of routine maintenance, when minor differences in road level will have been corrected. However, there have been several changes in the authority responsible for maintenance, and the records have been lost.

Table 1. Chronology of Surtees Bridge

the western abutment is approximately 8 mOD. The abutment is supported on 29 precast, driven concrete Herkules type 800 piles, 0·3 m in section, installed in two rows. The row nearest the river contains nine vertical piles installed alternately between ten piles raking forward at 1H:5V. The heel of the abutment is supported on ten piles that are raked backwards at 1H:5V. The eastern abutment (labelled F in Fig. 4) has a formation level of approximately 4·7 mOD. It is founded on 51 Herkules type 800 piles installed in three rows. Of the 51 piles, 13 form a single row of vertical piles beneath the heel of the abutment, with the remaining 38 piles forming two rows of raking piles that are pitched towards the river with a rake of 1H:3V. The driven lengths of all the abutment piles are believed to be between 30 and 33 m, with end levels corresponding to bedrock.

The bridge deck is horizontally restrained at abutment F and partially restrained at piers C and D (labelled R in Fig. 4). There are sliding bearings (labelled S in Fig. 4) at abutment A and piers B and E. Abutment F provides horizontal restraint of the bridge deck via the pile arrangement. Piers C and D are restrained to reduce their effective length in relation to buckling. Piers B and E are half the length of piers C and D and do not need to be restrained by the deck. There is an expansion joint between the deck and abutment A that is filled with a flexible plastic inlay. This allows for thermal expansion of the

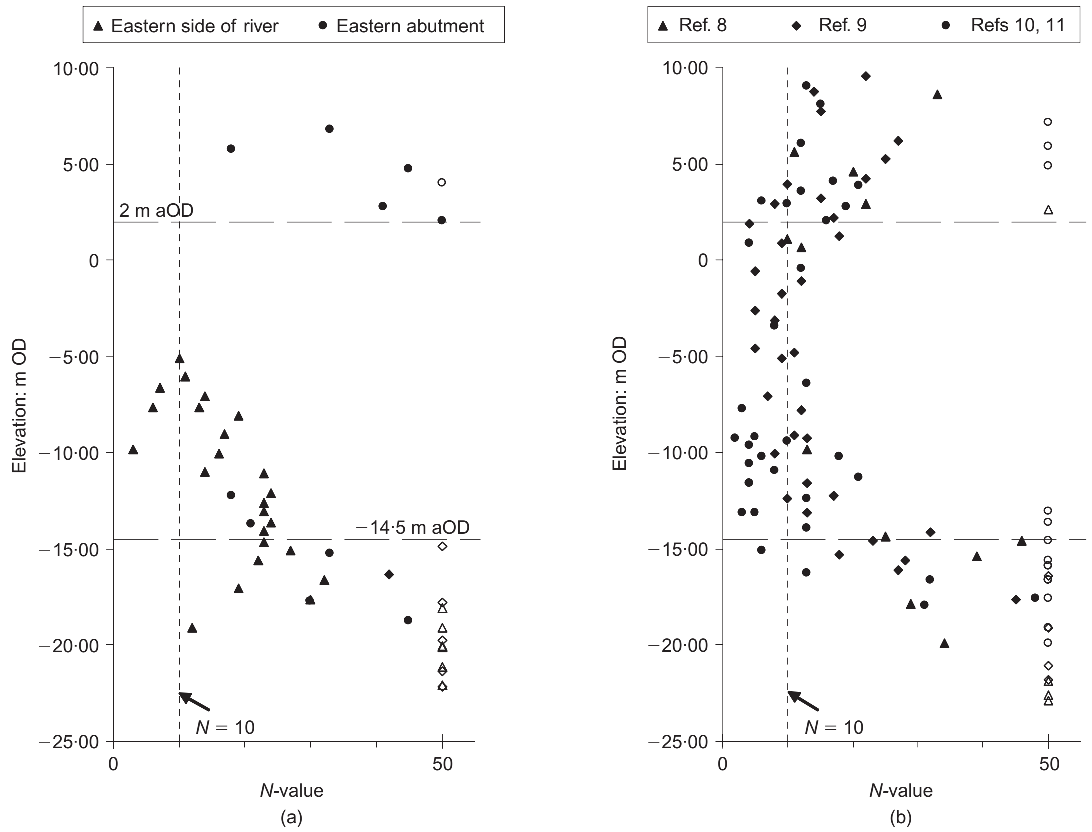

Fig. 3. SPT *N*-values measured in borings from within about 30 m of centreline of eastern bridge abutment: (a) pre-construction data (data from nearby borings in the river channel have been included to provide sufficient data for comparison); (b) post-construction data. *N*-values are uncorrected for depth, tests using a standard sampler and cone are not separately identified, and open symbols are used to show tests driven to refusal

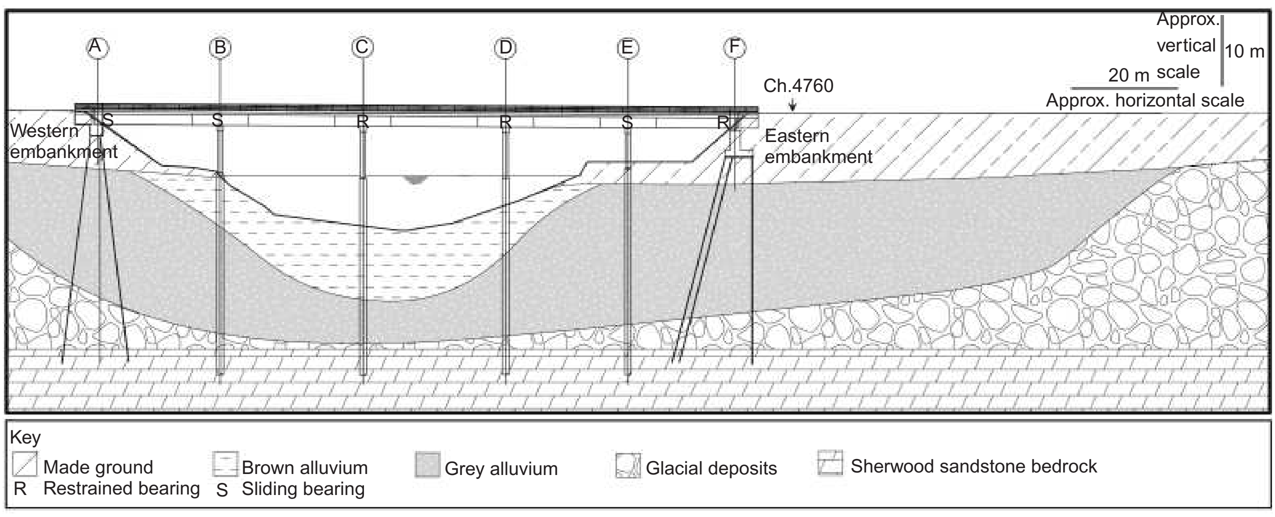

Fig. 4. Simplified longitudinal geological section, showing details of Surtees Bridge

bridge, while the plastic inlay prevents runoff and debris from the road entering the expansion gap.

5. GEOTECHNICAL PARAMETERS

Geotechnical parameters for each lithology described in Table 2 are summarised in Tables 3 and 4. Table 3 presents median values of relevant geotechnical parameters based on data from all six ground investigations conducted at the site. The settlement parameters for the glacial deposits should be treated with caution, as the borehole logs indicate that the two samples tested were recovered from locally finer soil; however, owing to the depth of the glacial deposits, the

Soil horizon	Description	Top of formation at Ch. 4760: mOD
Made ground (Victorian fill)	Medium dense/firm grey brown, sandy gravelly clay with partings of silt. Gravels of sandstone, limestone, brick, slag and clinker, with sand, occasionally of ash	3·5
Brown alluvium	Soft to firm, thinly laminated, brown sandy silty CLAY/sandy clayey SILT with a little gravel and occasional organic fragments	2·0
Grey alluvium	Soft to firm, thinly laminated, grey very silty CLAY/very clayey SILT with occasional gravels and parting of sand	1·5
Glacial deposits	Medium dense to very dense brown silty SAND and GRAVEL with some cobbles	−14·5
Sherwood sandstone	Very weak to moderately strong, thinly bedded, red brown, fine to medium grained SANDSTONE	−22·0

Table 2. Engineering descriptions of soil horizons beneath site of Surtees Bridge

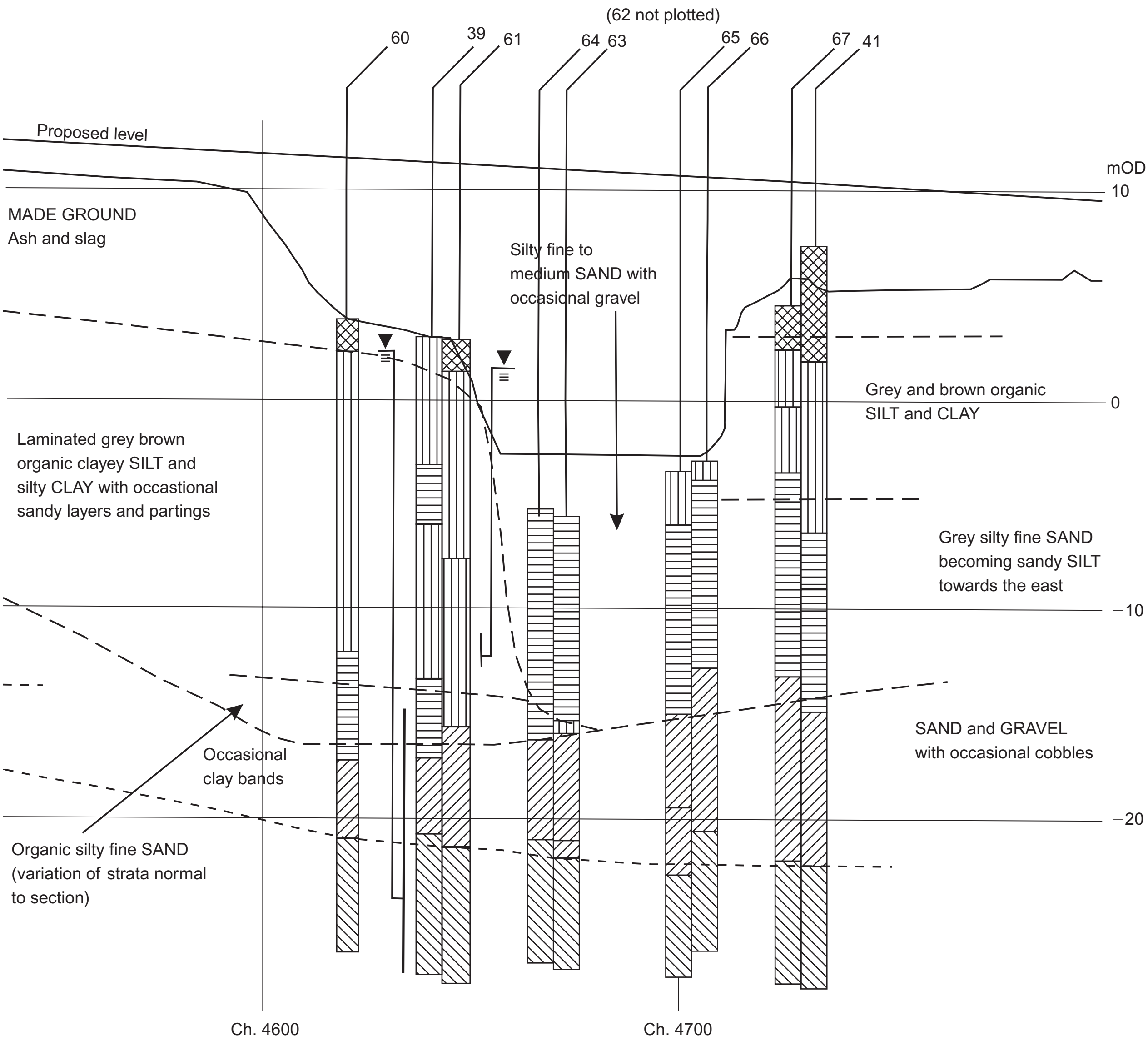

Fig. 5. Extract from the pre-construction longitudinal geological section

settlement of the eastern approach embankment is insensitive to these parameters. Compression of the grey alluvium will have dominated the settlement of the eastern approach embankment, and its parameters are therefore presented in Table 4 in more detail. Table 4 reports separately data from prior to construction (two consolidation tests) and the mean and median values for the full dataset (51 consolidation tests). Approximately equal values for mean and median indicate that the data distribution is close to symmetric, and either value can be used to characterise the dataset. A significant difference between the mean and median indicates a skewed dataset, and in such circumstances the median is

	Victorian fill	Brown alluvium	Grey alluvium	Glacial deposits
Soil properties				
γ_{bulk}: kN/m^3	18·15	17·85	17·95	21·09
γ_{sat}: kN/m^3	18·16	17·89	18·10	20·88
D_{10}: μm	20	1	2	230
e_0	0·81	0·99	0·93	0·48
c_u: kN/m^2	98	27	38	140
Settlement parameters				
m_v: m^2/MN	0·11	0·24	See Table 4	0·07
C_v: m^2/year	1·83	5·6	See Table 4	1·83
C_c	0·29	0·26	See Table 4	0·11
C_r	0·06	0·03	See Table 4	0·03

Table 3. Geotechnical parameters (median values)

	Pre-construction dataset	Complete dataset	
	Mean*	Mean	Median
m_v: m^2/MN	0·08	0·22	0·22
C_v: m^2/year	10·90	12·31	2·74
C_c	0·22	0·29	0·26
C_r	0·03	0·04	0·03

* There are only two pre-construction values: thus the mean is equal to the median.

Table 4. Settlement parameters for the grey alluvium

considered a better indicator for characterising the dataset.[14,15]

The initial void ratio (e_0) and coefficient of volume compressibility (m_v) of the grey alluvium exhibit considerable scatter at shallow depths. This is probably because the measured values have been greatly influenced by stress increases generated by construction and filling across the site. Thus any apparent depth trends will not necessarily be a good guide to the situation prior to construction of embankments and are not considered further.

The mean pre-construction value of m_v for the grey alluvium is significantly lower than either median or mean for the whole dataset. This is probably an artefact of the stress increment used to evaluate m_v, which was 50 kPa in the pre-construction investigations but $\geqslant$100 kPa in the post-construction investigations. Re-evaluation of the pre-construction data using Casagrande's construction[16] indicates that the grey alluvium was lightly overconsolidated, with a pre-consolidation pressure about 30 kPa greater than the vertical effective stress prior to sampling. Thus a 50 kPa stress increment would have given a predominantly overconsolidated response. The higher stress increments of the recent testing would have been dominated by compression in the normally consolidated section of the consolidation curve.

The coefficient of volume compressibility (m_v) is defined as the slope of the one-dimensional consolidation response (on a graph of void ratio e against vertical effective stress σ'_v divided by the specific volume ($1 + e_0$). Thus the relationship between m_v and recompression index C_r (the slope of a graph of e against the logarithm of σ'_v) is defined by the equation[17]

$$m_v = \frac{C_r}{2{\cdot}3\sigma'_v(1 + e_0)} \quad (1)$$

If it is assumed that the grey alluvium is overconsolidated throughout the consolidation test stress increment, then equation (1) yields values of m_v very similar to those obtained from the pre-construction investigations. This observation tends to confirm that the m_v values measured in advance of construction reflected an overconsolidated soil response, and any calculation based on these values would underestimate the likely settlement induced by embankment construction.

The compression index (C_c) and recompression index (C_r) of the grey alluvium as calculated from pre-construction data are similar to the median values for the whole dataset. This consistency between the pre- and post-construction consolidation data permits C_c, C_r and an estimate of the pre-consolidation pressure to be used to back-calculate embankment settlement.

A further feature of the settlement parameters for the grey alluvium is a significant difference between the median and mean values of the coefficient of consolidation (C_v) for the whole dataset. This may be taken to indicate that the dataset is highly skewed, and that the mean value is strongly affected by extreme values (the mean is three times the upper quartile value of 3·8 m^2/year). The C_v values from the two pre-construction consolidation tests are similar to the mean C_v value for the whole dataset. This suggests that they were not representative of the grey alluvium.

6. EVIDENCE OF BRIDGE DISTRESS

There have been reports of excessive ground movements at Surtees Bridge since its first principal inspection (details are given in Table 1). By 1992 concern about these movements was sufficient to make assessing the reasons for continuing settlement of the eastern embankment a primary objective of the ground investigations performed for the proposed widening of the bridge.[8]

A bridge inspection undertaken in 2000 indicated that the bridge deck had moved westward relative to abutment A and pier B. The displacement of the bridge deck relative to pier E is easterly, however. The articulation of the bridge is such that only piers C and D and abutment F can transfer horizontal forces to the bridge deck. Piers C and D are unlikely to be the cause of the bridge deck movement, because their foundations are remote from any source of lateral load. It was therefore deduced that the bridge deck had moved westward due to forward movement of abutment F. The movement of the bridge deck relative to pier E therefore indicates that there has been a westward movement of the head of pier E greater than westward movement of the bridge deck. Westward displacement of pier E during construction of abutment F is noted in a report prepared by Bullen and Partners in 1993.[18] Movement of abutment A is not thought to be the primary cause of closure of the expansion joint above that abutment, because there are no visible signs of settlement of the western approach embankment, and it cannot account for the movement of the bridge deck relative to piers B and E.

During a walkover survey conducted in May 2005 a number of signs of distress were observed at locations indicated in Fig. 1. The most obvious sign of distress (#1 in Fig. 1) is a step in the public footpath adjacent to the westbound carriageway on the eastern approach embankment. Assuming that originally there was no step in footpath height between the bridge and the eastern embankment, and that the footpath has not been resurfaced since it was constructed, then there has been a vertical displacement of about 0·2 m at this point (Fig. 6). Also there has been an estimated 100–150 mm of horizontal movement over the sliding bearing on abutment A (#2 in Fig. 1), as shown in Fig. 7, where only a small gap now remains at the expansion joint. Other signs of distress include lateral tension cracks in the road surface (#3 in Fig. 1) adjacent to the joint between the bridge deck and eastern approach embankment (despite the road being resurfaced several times since the bridge opened) and lateral tension cracks located within the soil of the soft verge on the south side of the eastern approach embankment (#4 in Fig. 1).

Monitoring of inclinometers installed beneath the eastern approach embankment in 2000 and 2002 indicated horizontal displacement within the grey alluvium. The displacement pattern takes the form of a bulge, with no preferred direction to the movement. The pattern is interpreted as buckling of the inclinometer tubes within the soft alluvium as it is compressed by the overlying made ground. Deformation does not occur in the stiffer made ground above or sand and gravel deposits below. It is believed that the inclinometer tubes are 'gripped' by the made ground, and so cannot slide to relieve the axial stress that eventually leads to buckling. The inclinometer data indicate the eastern embankment is still settling.

Other structures in the vicinity of Surtees Bridge show signs of distress, suggesting that excessive settlement is an issue in this locality. For example, the nearby Tees Bridge (location #5 in Fig. 1) has a speed restriction imposed, and its most easterly pier has recently been strengthened.

7. SETTLEMENT OF THE EASTERN APPROACH EMBANKMENT

The amount and rate of primary consolidation settlement at chainage 4760 have been calculated using Terzaghi's one-dimensional method.[19] The initial stresses were calculated from the soil profile shown in Fig. 8, assuming that the Victorian embankments could be represented by equivalent trapezoidal pressure distributions acting at a ground level (3·5 mOD). Osterberg's method[20] was used to calculate the vertical stress increase, which idealises the foundation soil as a homogeneous, isotropic, elastic material. The error caused by assuming isotropic elasticity is typically about ±20% even when the soil is systematically non-homogeneous, anisotropic and non-linear.[21] Embankment construction was simulated by removing the pressure representing the Victorian fill and replacing it with a trapezoidal pressure distribution representing the new embankment. Removal of the Victorian railway embankments is assumed to occur simultaneously with placement of the new fill.

Fig. 6. (a) View westwards across Surtees Bridge showing a repair made to the footpath to compensate for settlement of the eastern embankment; (b) the change in height across the repair.

Fig. 7. A sliding bearing on abutment A

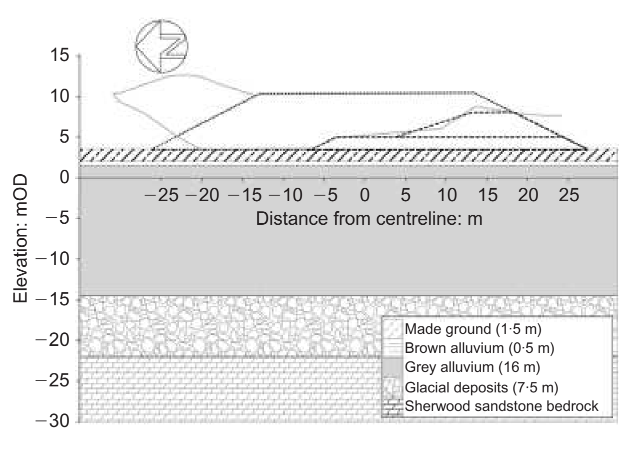

Fig. 8. Idealised cross-section at chainage 4760

For the purposes of the analysis it is assumed that the groundwater level at the time of embankment construction was about 1·5 m below ground level (2 mOD), which is slightly above mean river level before construction of the Tees Barrage. Median values of C_c and C_r were used in the analysis (Tables 3 and 4), and it is assumed that the maximum consolidation stress is about 30 kPa greater than the current vertical effective stress. The results of the settlement analysis for chainage 4760 are presented in Fig. 9 (the analysis ignores the settlement of the embankment fill). The maximum predicted primary consolidation settlement is 0·71 m, with the asymmetric distribution reflecting the smaller stress change under the site of the Victorian embankments.

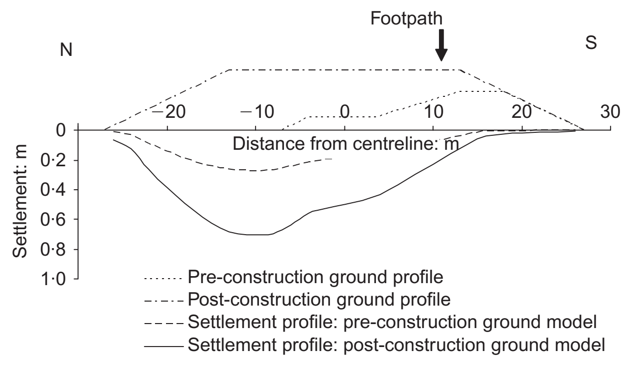

Fig. 9. Predicted primary consolidation settlement at chainage 4760

To apply Terzaghi's analysis of rate of settlement to the ground conditions at chainage 4760 it is necessary to further simplify the ground model by assuming that the 0·5 m thick layer of brown alluvium has the same consolidation properties as the 16 m thick layer of grey alluvium. This composite layer is assumed to be undergoing two-way vertical drainage into the Victorian fill and glacial deposits on the basis of particle size (Table 3 gives the median D_{10} of each soil horizon). The mean C_v value of the grey alluvium is unrepresentative, because the data distribution is highly skewed and strongly affected by outliers, so the median C_v value of 2·74 m^2/year is used in the rate analysis. The analysis suggests that the time for 95% primary consolidation is 28 years, and that about 93% of the consolidation settlement has occurred to date (Table 5).

For comparative purposes, the amount and rate of consolidation settlement at chainage 4760 have also been evaluated for the ground model available pre-construction. At that time it was believed that the layer of alluvial clays was only about 7·5 m thick under the eastern abutment, and underlain by silty sand. A total of six consolidation tests were conducted as part of the SI for Surtees Bridge: four on brown alluvium from under the western abutment, and two on grey alluvium from under the eastern abutment.[7] Mean C_c values were 0·33 and 0·22, and mean C_v values were 9·8 and 10·9 m^2/year for brown and grey alluvium respectively. The pre-construction ground model does not differentiate the brown and grey alluvium, but the borehole logs indicate that only grey alluvium was found under the eastern embankment. Consolidation properties were not derived for the Victorian fill, alluvial sands or glacial deposits during design of the bridge, probably because they appeared to be coarse and therefore were believed to be relatively incompressible (subsequent results have shown that this was a significant error for the fill).

The maximum predicted primary consolidation settlement calculated using the pre-construction ground model is only 0·28 m (see Fig. 9) and the time for 95% consolidation only 1·5 years (Table 5). Thus it seems that, pre-construction, only moderate embankment settlements would have been anticipated, and these were expected to occur principally during construction, when they were unlikely to cause bridge distress.

8. DISPLACEMENT OF PILES UNDER PIER E AND ABUTMENT F

It is believed that westward movements of pier E and abutment F were caused by deformation of the alluvial clays beneath the eastern approach embankment causing lateral loading of the piled abutment and pier foundations. To test this hypothesis, the movements of the pile caps at pier E and abutment F were evaluated using a very simple empirical method.[3] The method involves a design chart, developed from centrifuge model data and field observations, that relates the non-dimensional pile cap deflection Δy_q to the relative soil–pile stiffness K_R, where

	Based on pre-construction data	Based on all data
C_v: m^2/year	10·9	2·74
Drainage path: m	3·75	8·25
Time for 95% consolidation settlement: years	1·5	28·0
Current amount of consolidation for $t = 24$ years: %	>99·9	92·6

Table 5. Results of time–settlement rate analysis

$$\Delta y_q = \frac{\Delta y E_p I_p}{\Delta q d L_{eq}{}^4} \quad \text{(2a)}$$

$$K_R = \frac{E_p I_p}{E_s h_s^4} \quad \text{(2b)}$$

and Δy is the horizontal deflection of the pile cap, Δq is embankment load, d is pile diameter, L_{eq} is the equivalent length of the pile between points of fixity, E_p is the Young's modulus of the pile, I_p is the moment of inertia of the pile, E_s is the representative stiffness of the soft clay layer, and h_s is the thickness of the soft clay layer. The equivalent length, L_{eq}, is equal to either $1{\cdot}3L$ (where L is the pile length above the base of the soft layer) when horizontal movement is not restrained (pier E) or L when rotation is prevented by a rigid cap (abutment F).[3] The method takes no specific account of factors such as pile spacing, group size, group configuration or embankment shape, although a broad range of pile abutment configurations are represented in the dataset used to develop the design chart.

The design chart recognises that there is a marked increase in pile cap deflection as the stress increase due to the embankment exceeds about three times the undrained strength of the soft clay layer, owing to the onset of significant plastic deformation in the soft stratum. The stress increase due to construction of the eastern approach embankment was typically just under three times the undrained shear strength of the alluvium deposits. The limit was, however, exceeded under a relatively narrow section of the full-height embankment in an area not previously loaded by the Victorian railway sidings embankment. The pile cap deflections were therefore estimated using the upper limit for loading less than $3c_u$.

The parameters used to analyse Surtees Bridge are presented in Table 6. The Young's modulus of the grey alluvium has been estimated from the volume compressibility m_v by assuming a drained Poisson's ratio of 0·2.[22] The design chart assumes that the piles are vertical, but the authors recommend that a reduction of 25% be made in the predicted pile cap deflection if some of the piles are raking. This correction has been applied to the predicted displacements of abutment F.

The lateral pile cap deflections at pier E and abutment F are estimated to be about 0·34 m and 0·30 m respectively (Table 6). This prediction, which assumes that pile installation occurred before embankment construction was complete, is about twice the observed movements (abutment F has moved forward 0·10–0·15 m, and the westward movement of the top of pier E exceeds this amount). Given the very approximate nature of the analysis, this prediction is surprisingly good. (There is some uncertainty about the base level of the pile cap under abutment F, and the prediction is extremely sensitive to effective length; also, no allowance is made for pile restraint within the embankment fill.) The prediction for pier E must be treated with particular caution because two further, possibly opposing, effects have been ignored. First, the distance of pier E from the front slope of the approach embankment is similar to the thickness of the soft alluvial layer, so the foundations pier E would not feel the full effect of the approach embankment. Second, the prediction is for pile cap displacement, whereas it is not clear whether the displacement has been observed at the pile cap or cross-head of pier E. If the displacement of pier E was primarily rotational, then the cross-head displacement would be about 20% greater than the horizontal displacement of the pile cap.

Despite the shortcomings of the method, it has correctly predicted the order of magnitude of the deflections of pier E and abutment F. It is therefore concluded that lateral movement of the piled eastern abutment as a result of settlement of the approach embankment can explain the observed bridge deck displacement.

9. DISCUSSION

Surtees Bridge is showing signs of distress due to excessive settlement of the eastern approach embankment, and associated movement of the eastern bridge abutment. As a result there have been extensive post-construction ground investigations in the vicinity of that abutment—a process that has been given extra impetus by plans to widen the river crossing. This gave the authors the opportunity to reanalyse the performance of this structure using data not available to the original bridge designers, and speculate as to the cause of the ground movements.

A new ground model has been developed for the site of the eastern approach embankment, based on all the available data. This identifies the importance of a layer of soft grey alluvial clay, 16 m thick, only 2 m below the ground surface. A conventional analysis using this ground model predicts that the maximum consolidation settlement of the embankment will be 0·71 m. Under the footpath on the south side of the road the predicted settlement is about 0·19 m, which is comparable with the observed settlement at this point of around 0·2 m. The analysis also identifies that time for 95% consolidation of the grey alluvial clay is only just being approached. This agrees with the inclinometer data, which show that small

Parameter	Pier E	Abutment F
Pile diameter, d: m	1·35	0·305
Moment of inertia of pile, I_p: m^4	0·163	$0{\cdot}425 \times 10^{-3}$
Young's modulus of pile, E_p:* MN/m^2	26 000	34 500
Young's modulus of alluvium, E_s: MN/m^2	5·1	5·1
Thickness of soft clay layer, h_s: m	18·0	16·5
Δq: MN/m^2	0·126	0·126
L: m	21·5	20·5
L_{eq}: m	28·0	20·5
K_R	$7{\cdot}9 \times 10^{-3}$	39×10^{-6}
Δy_q	0·014	$0{\cdot}87 \times 10^{-3\dagger}$
Δy: m	0·34	0·30

* Determined from an assumed characteristic strength for in situ cast concrete of 30 MN/m^2 and the reported characteristic strength for Herkules pile concrete of 53 MN/m^2 using the relationship reported by Mindess *et al.*[23]
† Estimated by extrapolation of the trend line.

Table 6. Summary of pile head displacement calculations

deformations are still going on in the alluvial clay layers beneath the eastern embankment. A simple empirical analysis of the horizontal displacement of the piled bridge abutment has been used to show that the observed bridge deck displacements can be explained by this mechanism.

This raises the question of why large settlements of the eastern embankment were not anticipated from the original design. The answer seems to lie in the pre-construction ground model, which underestimated the thickness of the compressible alluvial clay layer under the eastern approach embankment by a factor of two. The data available before construction suggest that the alluvial layer changes from silty clay to sandy silt with depth on the eastern side of the River Tees. It also appears that the values of C_v measured for the grey alluvium before construction were about four times larger than the median value of the larger dataset now available. Thus it seems that relatively modest embankment settlement was anticipated and, owing to the higher C_v value and shorter drainage path, the majority of that settlement was expected to occur during construction.

Interestingly, the pre-construction ground model identifies that the alluvial clay layer under the western abutment was 12 m thick. Thus a larger amount of settlement occurring over longer time periods would have been anticipated prior to construction. Presumably this is why the western embankment was built in advance of bridge construction—to allow time for the embankment to settle before construction of the western bridge abutment.

10. CONCLUSIONS

Excessive movement of Surtees Bridge has been a concern since shortly after construction was complete. To date, the eastern abutment has moved forward by 100–150 mm, and the eastern approach embankment is estimated to have settled by about 0·7 m. These movements are the result of compression of an approximately 16 m thick layer of alluvial clay beneath the site, and have resulted in lateral loading of the piled foundations of the bridge abutment. Distortion of the bridge structure as a result of these movements has been a significant factor in the decision to replace the bridge when the crossing is widened in the near future.

The movements of the eastern bridge abutment and approach embankment were not anticipated prior to construction, because the original ground model for the site developed from data available before construction underestimated the thickness of a soft alluvial layer, and overestimated its coefficient of consolidation. This highlights the difficulties in characterising alluvial soils from the limited ground investigations conducted for many construction projects.

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