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Jim Johnson
Ove Arup & Partners Ltd, Leeds, UK

David Gwede
Ove Arup & Partners Ltd, Leeds, UK

Andrew Smith
Webber Associates, Knaresborough, UK

Ian Webber
Webber Associates, Knaresborough, UK

Mike Murphy
Amec Capital, Projects, UK

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Jim Johnson
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Ian Webber
Webber Associates, Knaresborough, UK

Mike Murphy
Amec Capital Projects
UK

Abstract

The proposed A650 Bingley Relief Road in West Yorkshire, UK, required the construction of a new dual carriageway to relieve traffic congestion in the town of Bingley. Part of the scheme involved constructing the road adjacent to an existing tied sheet pile wall, the "Canal Tied Wall" that had a history of movement. In this area the road passes over a kettlehole with peat and soft silts between 10 m and 14 m deep. The Canal Tied Wall forms the boundary between the realigned canal and the proposed new trunk road. It consists of two sets of steel sheet piles approximately 3.5 m apart with stainless steel tie-bars and mass concrete shear walls connecting them at the top.

In May 1994 following the completion of the Canal Tied Wall construction and during excavation of material on the roadside of the wall, a 45 m length of wall moved horizontally in excess of 200 mm (into excavation) with associated maximum vertical movement at the top of the wall of 230 mm. The wall was monitored from 1994 until 2001 and subsequently through the construction of the new road.

The Highways Agency's requirement for the Canal Tied Wall was "to carry out stabilisation works .... together with any remedial works required to the structure to overcome current and future settlement problems". It was originally envisaged by the Highways Agency that a piled solution would be required.

The basis of the Tender design was to reduce the load on the wall and hence stabilise the rate of movement towards the canal. The proposed design consisted of excavating the existing fill (originally placed at the time of construction of the wall) to a level about 0.8 m above its base, and constructing a reinforced earth wall with a gap between it and the sheet pile wall.

The geotechnical solution that was eventually adopted was to reduce the load exerted on the wall by the ground behind it, and to surcharge the soft deposits to reduce long term settlements. The system adopted used a mass wall constructed of precast lightweight concrete blocks built behind the tied wall. This solution realised savings to the anticipated cost of the scheme whilst meeting the performance requirements of the specification.

Numerical analysis was used to assess the anticipated performance of the ground, the existing structure, and the behaviour of the adjacent railway line during the construction operation and into the future.

The lateral movements realised in practice were significantly smaller than those predicted using even relatively sophisticated modelling even though the modelling had been calibrated using data gathered during the advance works. This was as a result of changes to the construction sequence made on site. Whilst these were relatively minor the effect on the movements appears to have been significant.

Interpretation of consolidation tests proved to be difficult even with the benefit of quite extensive settlement monitoring during and after the advance works. Several possible combinations of parameters gave an equally good fit to the data. The Observational Method was therefore adopted to provide a framework for adjusting the design during construction, subject to the observed behaviour of the ground. The flexibility this provided enabled necessary changes to the surcharge design to be made during construction, while maintaining control over the stability of the wall.

Key Words
Sheet Piles, Peat, Surcharge, Finite Element Analysis, Observational Method, Instrumentation.

INTRODUCTION

For much of its length, the new A650 Bingley Relief Road runs between the Airedale Railway Line from Leeds to Skipton to the south and the Leeds and Liverpool Canal to the north. Over a length of about 200 m (Fig 1) there was insufficient room for the new road between the canal and railway. In 1994 advance works were carried out to divert the canal to the north. A steel sheet pile retaining wall (the "Canal Tied Wall") was installed to form the boundary between the realigned canal and the new road, and to support the towpath of the canal.

After construction of the wall, soft material was excavated from the bed of the former canal and replaced with rockfill to support the new road. During excavation, significant lateral movements of Canal Tied Wall were observed over a length of about 60 m. A maximum movement of 230 mm towards the excavation was measured. The excavation was backfilled, and the movements reversed direction. By 1999, a net movement of about 35 to 40 mm towards the new canal had been measured, with a continuing rate of movement of the order of 10 mm per year. In the same period, a maximum settlement of 138 mm was measured on the wall and a maximum settlement of 1.0 m measured on the fill behind it. Boreholes put down after movement of the wall had occurred indicated that soft deposits in the area were deeper than anticipated in the design of the advance works and that the sheet piles had not reached the underlying stronger materials.

The successful Tender for the new road was submitted by Amec Capital Projects Ltd., with Ove Arup & Partners as their Designers and Webber Associates as Geotechnical Sub-Consultants for Canal Tied Wall.

The solution proposed in the successful tender comprised the replacement of some of the rockfill behind the wall by a reinforced earth wall, with its face a short distance behind the wall. The purpose of this was to reduce the force on the wall and to lower its resultant point of action, and hence stabilise the wall. A surcharge was to be applied to reduce the long-term settlements of the road to acceptable values.

During detailed design the solution was refined, with the reinforced earth wall being replaced by lightweight concrete blocks. This further reduced the load on the wall, and also reduced the required magnitude of surcharge.

The aim of the design was that movements of the wall during and after construction of the road should not be greater than during the original construction of the wall. Predictions of the stability and movement of the wall and of settlements behind it were carried out using both simple stability and deformation analyses and two dimensional finite element analyses.

During construction of the road, monitoring was carried out on the wall, on the ground behind it and on the adjacent railway. The measurements were compared with limits derived from the analyses and hence used to control the construction process.

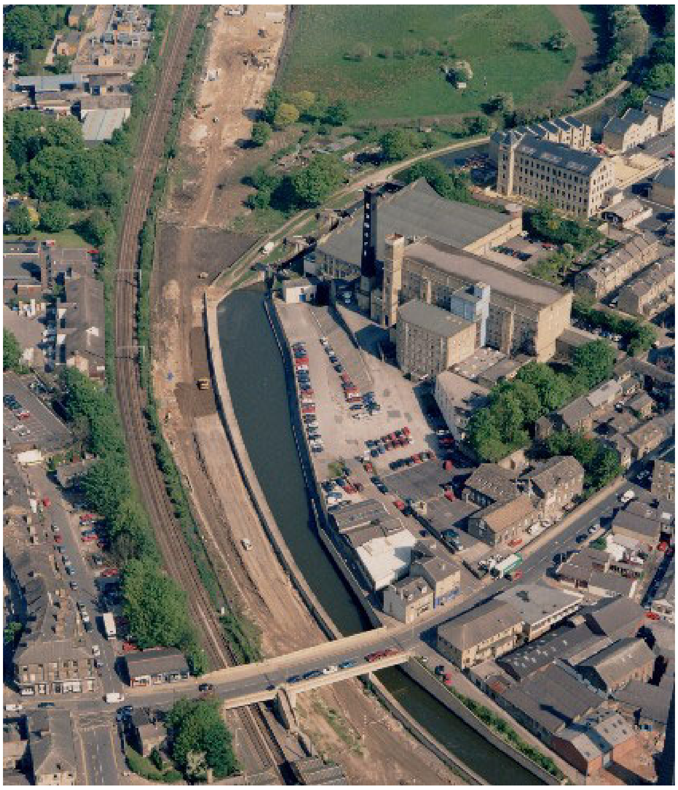

Fig. 1. *An aerial view showing the canal, 'wall', proposed road earthworks and railway line.*

GEOLOGY AND SOIL CONDITIONS

Soil conditions at Canal Tied Wall were assessed from investigations carried out both before and after construction of the wall. The various investigations carried out are summarised in Table 1, and key exploratory hole locations shown on Fig. 2. A cross section is shown on Fig. 3.

Table 1 Summary of Site Investigations at Canal Tied Wall

Date	**Investigation Carried Out**
1983	5 boreholes in general area
1994	5 boreholes in area of wall movement
2000	5 boreholes in area of wall movement
2001	2 static cone tests in area of wall movement
2001	4 rotary boreholes with piezometers in area of wall movement

The overall succession of strata in the area comprises made ground and recent deposits overlying glacial deposits and then Upper Carboniferous rocks of the Millstone Grit series.

Made ground, believed to be associated with the original construction of the canal and railway was found to be between 2.6 and 5.25 m thick. It consisted of variable proportions of clay, sand and gravel, sometimes with pockets of coal, brick, ash and wood.

Recent deposits consisted of peat overlying soft clay. The boreholes made in 1983 indicated the total thickness of soft material to be between 2.6 and 6.0 m, with the lowest level of the base being 68.7 mOD (ground level was approximately 78 mOD).

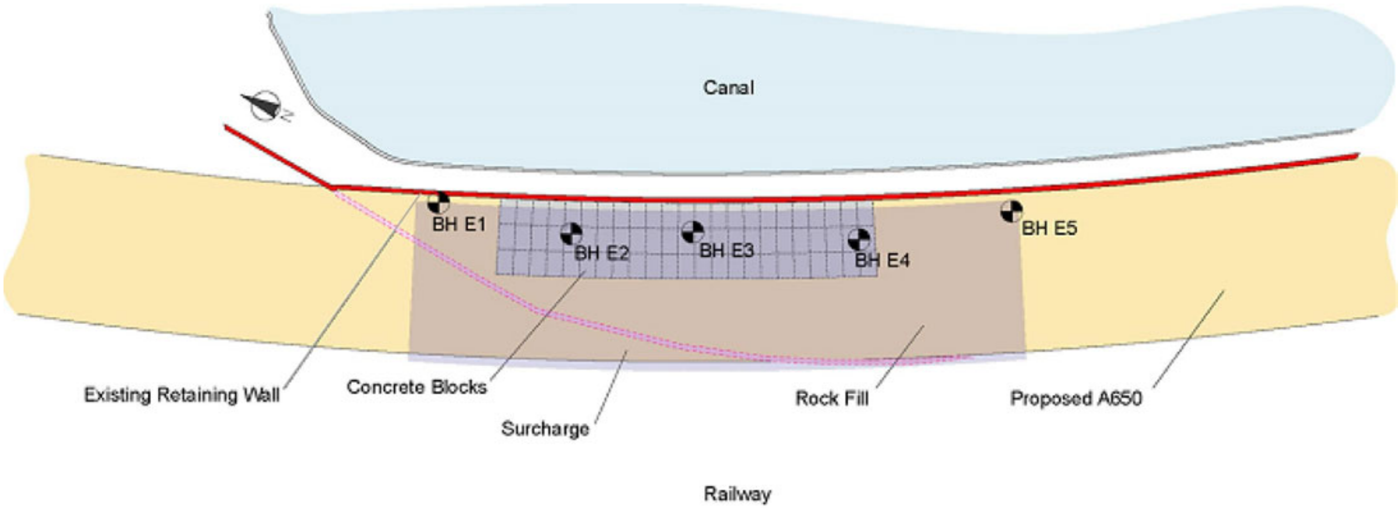

Fig. 2 *Schematic plan of special earthworks, borehole location, canal wall and canal*

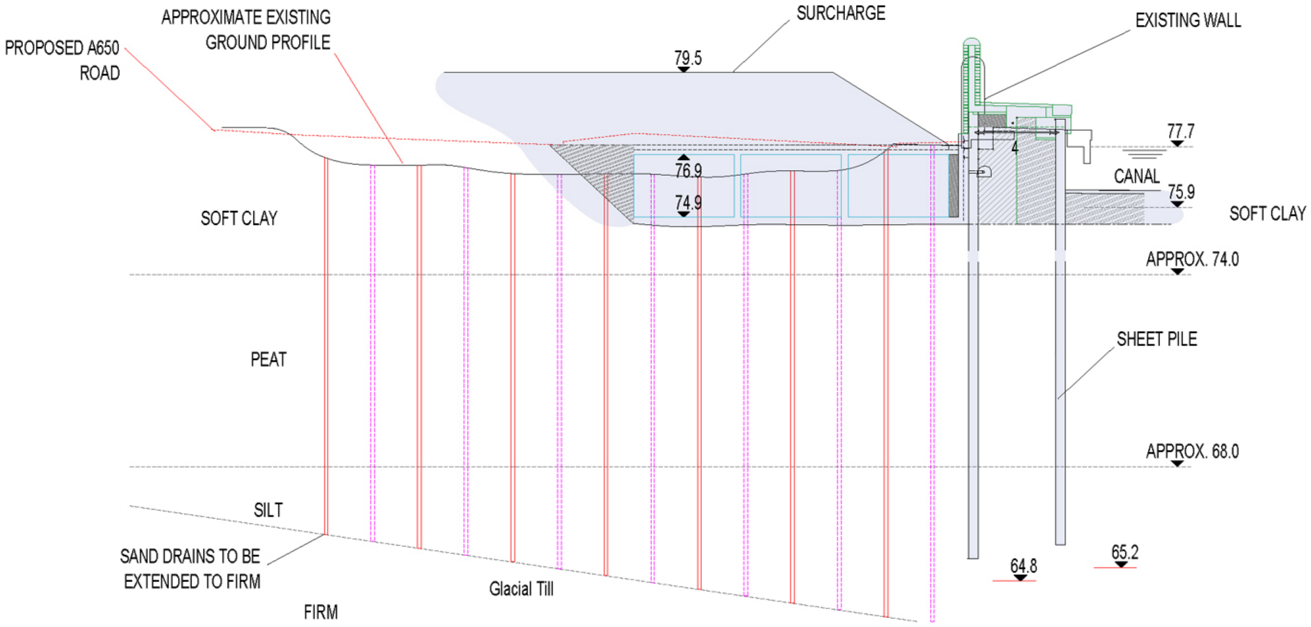

Fig. 3 *Idealised section through special earthworks and canal wall.*

Boreholes put down after movement of the wall occurred identified an area, coinciding closely with the length of movement of the wall, where the soft deposits were locally thicker than previously found. The lowest level found for the base of the soft deposits was 64.0 mOD.

It is believed that the depression in the surface of the glacial deposits may result from a kettlehole. This phenomenon occurs when a lens of ice is trapped within glacial deposits. It subsequently melts, leaving a depression in the surface, which then is filled with soft deposits.

The glacial deposits consisted of stiff sandy clay with gravel or dense clayey gravel. They were not fully penetrated by any of the boreholes in the area, and the underlying Carboniferous strata were not reached.

SOIL PROPERTIES

Properties of the soft deposits are summarised in Table 2. It can be seen that the moisture contents of the peat in particular were extremely high. The results also indicated that the soft deposits within the area of the kettlehole were not only thicker than elsewhere, but also wetter and softer.

Table 2: Summary of Average Soil Properties

Property	**Within Kettlehole Area**		**Outside Kettlehole Area**	
	Clay	**Peat**	**Clay**	**Peat**
Moisture Content (%)	132	580	52	286
Plastic Limit (%)	72	281	46	242
Liquid Limit (%)	151	537	90	393
Plasticity Index (%)	79	256	44	151
Undrained Shear Strength (kPa)	9	14	15	35

Values of the coefficient of volume compressibility, m_v, are presented on Fig. 4. The results indicated that the peat was only slightly more compressible than the clay, but that both materials were more compressible within the area of the kettlehole than outside it.

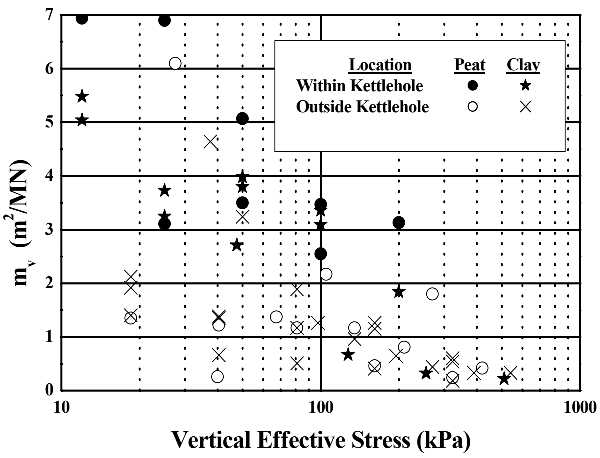

Fig.4 *Values of Coefficient of Volume Compressibility*

DESCRIPTION OF CANAL TIED WALL

The wall consists of two rows of Frodingham No. 4 steel sheet piles approximately 3.5 m apart, with 40 mm diameter steel tie bars and mass shear concrete walls connecting them at the top. For overall stability the two rows of piles were considered to act in unison connected by the shear walls and ties. The idealised structure is shown in Fig. 3.

A design level of 68.6 mOD was adopted for the top of the glacial material, and the piles, 13.4 m long, were to be embedded 3.7 m into it. In fact, because the top of the glacial deposits was lower than indicated by the initial boreholes, the sheet piles were founded entirely in the soft clay and peat over a wall length of about 30 m.

DESIGN OF STABILISATION WORKS

The Highways Agency's requirement for the Canal Tied Wall was "to carry out stabilisation works .... together with any remedial works required to the structure to overcome current and future settlement problems". As described above, the situation at the time of the Tender was that the wall had moved a net distance of 35 to 40 mm towards the realigned canal, and was continuing to move at about 10 mm per year. It had also settled 138 mm, with the latest readings indicating it to have stabilised at that value for about a year. The fill behind the wall had settled 1.0 m, with the latest readings indicating a continuing rate of about 70 mm per year. As a consequence of this settlement, the ground level was up to 1 m below the finished road level.

The basis of the Tender design was to reduce the load on the wall and hence stabilise the rate of movement towards the canal. It was also necessary to reduce the post-construction settlements of the road to acceptable values. The proposed design consisted of excavating the existing fill (originally placed at the time of construction of the wall) to a level about 0.8 m above its base, and constructing a reinforced earth wall with a gap between it and the sheet pile wall. A surcharge would then be placed to accelerate settlement during the construction period. Removal of the surcharge would thus reduce the settlements occurring after completion of the carriageway.

The aim of the design was that movements of the wall during construction of the road should not be greater than during the original construction of the wall. The excavation in the fill was therefore to be shallower than the excavation that was made immediately after construction of the wall. It was accepted that the surcharge would apply a greater load towards the canal than had been applied in the past. However, analyses were carried out to check that the magnitudes of movements and bending moments would be smaller than had occurred in the past, although they would be in the opposite direction.

Two changes were made to the design during the detailed design process. The first was to install vertical sand drains through the soft deposits behind the wall. This measure was adopted because of uncertainty as to whether full consolidation of the soft material would be achieved during the eight-month period available for application of the surcharge. Sand drains were installed rather than band drains, because of the need to predrill through the rockfill.

The second change was to replace the reinforced earth wall with lightweight concrete blocks. The effect of this change was to reduce further the load on the wall. It also reduced the anticipated magnitudes of settlement behind the wall, and hence the required magnitude of surcharge, which in turn reduced the maximum temporary load on the wall during construction. A final advantage was that the blocks could be precast and lifted into the excavation. This operation was both faster and safer than construction of the reinforced earth wall which would have had to have taken place within the excavation.

STABILITY AND DEFORMATION ANALYSIS OF WALL

Stability

A conventional stability analysis of the wall at different stages of construction was carried out using factored strengths.

The analysis of the excavation made immediately after construction indicated a required toe depth of about 16 m: i.e. about 3 m deeper than the actual depth. This finding is consistent with the fact that significant movements actually occurred with the toe at 13.0 to 13.7 m.

Analysis of the final design was carried out using the same factored strengths (i.e. neglecting any increase of strength that had resulted from the placement of fill in 1994). The most critical situation occurred immediately after the placement of surcharge. This situation gave a required toe depth of 13 m, indicating that the wall would just satisfy stability requirements.

Movements

An estimate of lateral movements was made using the retaining wall analysis program FREW by OASYS. This uses an elastic spring model for the soil. The values of elastic moduli for the soft clay and peat were based on their shear strengths. In addition, a check was made that reasonable agreement was obtained with movements measured during the original construction of the wall.

The movements calculated by FREW for each stage of construction are presented on Fig. 5.

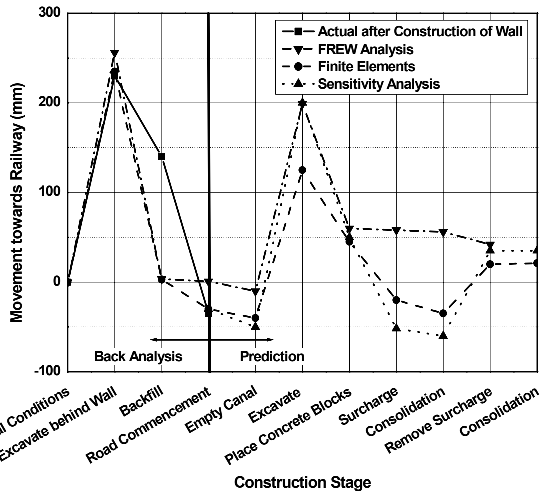

Fig. 5 *Horizontal Wall Movements Calculated using FREW & SAFE*

An estimate of settlements was made using the two-dimensional finite element program SAFE by OASYS. Again, an isotropic elastic model was used. However, the finite element formulation had some advantages over FREW, notably the ability to model separately the two sections of the canal tied wall and the connection between them, the incorporation of vertical as well as horizontal movements, and a more rigorous modelling of the consolidation process.

The movements calculated by SAFE for each stage of construction are also presented on Fig. 5. The program was calibrated using the movements measured during the initial construction of the wall.

In addition, a sensitivity analysis was carried out using reduced strengths and stiffnesses. This analysis was used to help assess criteria to be used in controlling the construction of the wall.

SETTLEMENTS OF FILL BEHIND WALL

As stated above, it was proposed to apply a surcharge in order to reduce long-term settlements of the road to acceptable values. The design of the surcharge was a critical part of the design of the stabilisation works as a whole, because of the load that the surcharge would apply to the sheet pile wall and to the adjacent railway.

In principle, the design of surcharge is simple. The settlement (usually only primary) occurring under the surcharge has to exceed the primary and secondary settlement expected to occur under the final loading.

The selection of appropriate design values for the soft deposits proved to be difficult. As can be seen from Fig. 4, the laboratory oedometer results indicated a considerable scatter of values of coefficient of volume compressibility, m_v. None of the tests had been extended to measure secondary compression, and the selection of appropriate values of the coefficient of consolidation from laboratory tests alone is notoriously difficult.

It was considered that back-analysis of the settlement records should aid the assessment of parameter values. However, it was found that very different interpretations could give equally good fits to the measured data. Figure 7 shows the results of two extreme interpretations, with the parameter values appropriate to each shown in Table 3. The first interpretation was that primary consolidation was complete in about 9 months, with the remaining settlement resulting from secondary compression. This was the interpretation used for the Tender Design. The second interpretation was that the entire settlement was represented by primary compression. As can be seen from Table 3, these two interpretations resulted in very different surcharge designs.

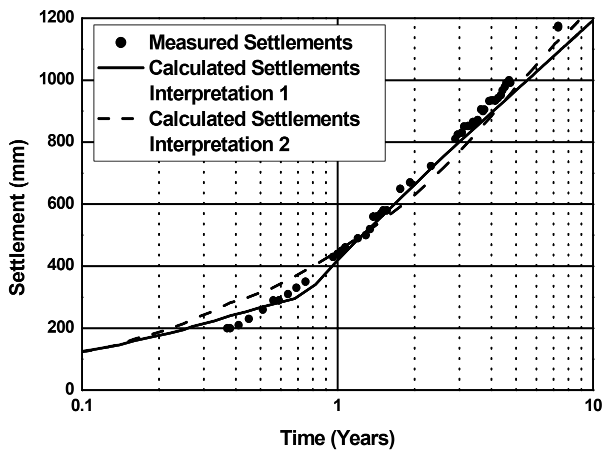

Fig. 7 *Measured and Calculated Ground Settlements*

The parameter values adopted for Detailed Design are also shown in Table 3. The adopted value of m_v (3 m^2/year) was selected on the basis of the oedometer results from the area of the kettlehole only (Fig. 2) at appropriate stress levels. The adopted value of the coefficient of secondary consolidation C_α (0.025) was based on correlations with moisture content published by Hobbs (1984).

The parameter values adopted for Detailed Design implied a requirement for vertical drains to be installed. The design was therefore revised to incorporate these.

Table 3 Consequences of Design Interpretations

	Interpretation 1*	Interpretation 2[§]	Design Values
Coefficient of Volume Compressibility, m_v (m^2/MN)	1	4	3
Coefficient of Consolidation, C_v, (m^2/yr)	25	2	3
Coefficient of Secondary Consolidation, C_α	0.075	-	0.025
Required Surcharge Height (m)	5	2	2
Vertical Drains Required?	No	Yes	Yes

*Interpretation 1 Primary Consolidation complete in 8 months
[§]Interpretation 2 Primary Consolidation not complete in 7 yrs

MONITORING AND CONSTRUCTION CONTROL

Development of Construction Control Strategy

In view of the sensitive nature of the wall, it was considered necessary to employ the observational method (Peck, 1969) and to monitor the wall during construction. It is a necessary feature of employing the observational method (e.g. Nicholson, 1994, Powderham, 1994) that it should be possible to set criteria for assessing the results of such monitoring and that there should be contingency plans ready in case these criteria should be breached.

The criteria set were based on the calculated values of expected movements. A system of "Amber" and "Red" limits was employed. The Amber Limits were based closely on the predicted behaviour of the wall, and served as warnings that the expected movement might be about to be exceeded. The Red Limits were based on the sensitivity analysis and were generally set about 50 % greater than the Amber Limits. They served as prompts that action should be taken to reverse the deteriorating trend. Such actions could comprise, for example, refilling the excavation or removing surcharge.

It was appreciated that pre-specified limits should not be applied blindly. The rates of movement and their rates of acceleration also need to be assessed, together with any variations from the construction sequence envisaged during the design process. This construction sequence was an integral part of the design, and was therefore specified at detailed design stage.

An additional complication was the fact that two reversals of movement were expected to occur during construction. The first would occur when the excavation was backfilled and the second when the surcharge was removed. It therefore followed that, if the movement during any stage of construction was greater or less than predicted, the limits on movements during subsequent stages would need to be reassessed.

Finally, the assessment of monitoring criteria was made more difficult by the fact that there were three separate but interrelated problems:

- lateral movements of the sheet pile wall
- settlement of the soft material behind the wall
- effects of construction on the adjacent railway

It was recognised that measures to improve one of these elements might have a detrimental effect on one or both of the others. For example, a deeper initial excavation behind the wall would have enabled more fill to be replaced by lightweight concrete, thus reducing long-term settlements behind the wall and the long-term load on the wall. However, this would have increased the potential short-term effects on both the wall and the railway.

Instrumentation

Instruments were installed to monitor the performance of the wall during construction.

The main method of monitoring the wall was by survey. Twenty-nine survey markers were fixed to a 70 m length of the wall. The advantages of survey markers are that they can be read and interpreted very rapidly, they give a direct measure of the movement of greatest interest, and they can easily be replaced if damaged (for example by vandalism).

Survey markers were also installed on overhead line gantries for the adjacent railway and in the ballast near the track.

Three inclinometers were installed adjacent to the wall. The purpose of these was to supplement the information from the survey markers, and in particular to confirm that movements at depth did not exceed those above ground and that no discontinuities of movement were occurring in the ground.

Survey markers were also installed on the ground surface behind the wall to measure settlements. These were protected by manhole rings, so that they could be extended upwards as the surcharge was placed.

Vibrating wire piezometers were installed in the soft deposits. Unfortunately, it was not practicable to install them underneath the area of the excavation and lightweight blocks, because of the risk of damage to the cables. They were therefore installed just outside this area, but still within the area of the surcharge and vertical drains.

CONSTRUCTION OF REMEDIAL WORKS

Construction of the remedial works began in late January 2002. A summary of the main construction events is presented in Table 4.

The first operation was the emptying of the canal. This caused movements of up to 33 mm (Fig 8), which were not considered to be of concern. However, shortly afterwards, a large piece of plant was stationed just behind the wall. This caused the maximum movement towards the canal to increase to 77 mm, with the maximum rate of movement reaching 23 mm/day. Movements slowed down significantly when the plant was removed, and reversed slightly when the canal was refilled. However, the episode did demonstrate the sensitivity of the wall to loading, and the need for vigilance during construction.

Table 4 Sequence of Main Construction Events

No*	Start	Finish	Event
1	28/01/02	8/02/02	Canal emptied and crane working behind wall
2	19/02/02	8/03/02	Excavation behind wall and lightweight blocks placed
3	8/05/02	25/06/02	Design surcharge completed to 79.5 mOD
4	1/08/02	26/08/02	Additional surcharge completed to 81.0 mOD
5	15/04/03	30/04/03	Surcharge removed

* Numbers correspond to stages indicated on Fig. 8

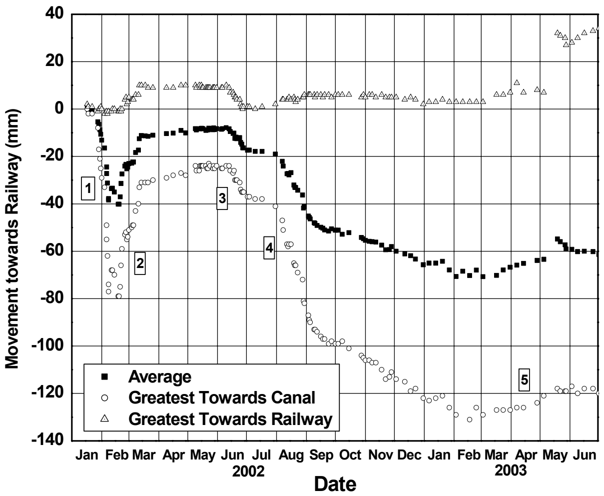

Fig. 8 *Lateral Movements of Wall During Construction* (Refer to Table 4 for Construction Events)

The excavation for placement of the lightweight concrete blocks was begun on 19th February. In order to minimise the effect on the wall, the excavation was carried out in stages. An initial excavation was made for the two rows of blocks furthest from the wall, with a berm being left in position adjacent to the wall until those blocks had been placed. Excavation adjacent to the wall was then carried out in three separate bays, with the blocks in each bay being placed before the next bay was excavated.

The effect of this strategy was that the movements of the wall were much smaller than the calculated values, which had been based on the assumption that the excavation would be a single operation, followed by backfilling. The maximum movement away from the canal was of the order of 50 mm, compared to the calculated values of about 100 mm from the finite elements and 200 mm from FREW.

At this time, however, movements of about 30 mm towards the excavation were observed on some of the nearby railway markers. These movements ceased when the excavation was backfilled.

Because the movements to this stage of construction had been different from those envisaged during design, it was necessary to reassess the Limits for the subsequent stage, that of placement of the surcharge. It was decided to redefine the Limits such that the permissible incremental movements during placement of surcharge remained the same. This implied a greater movement towards the canal than had been calculated. However, the permissible deflection from the immediate post-construction position would be less than had occurred when the original movements occurred, albeit in the opposite direction.

The surcharge was placed in half metre increments with an interval of at least a week between each increment. The first two increments cause virtually no movement of the wall. This was not surprising, as the unloading resulting from the replacement of fill by lightweight concrete was not reversed until about 0.7 m of surcharge had been placed.

The maximum lateral movement caused by the remaining three increments of surcharge was about 20 mm, which was smaller than had been calculated. At the same time, however, the monitoring of settlements behind the wall (Fig. 9) showed that these were also significantly smaller than had been calculated. A reassessment of the surcharge design was therefore carried out.

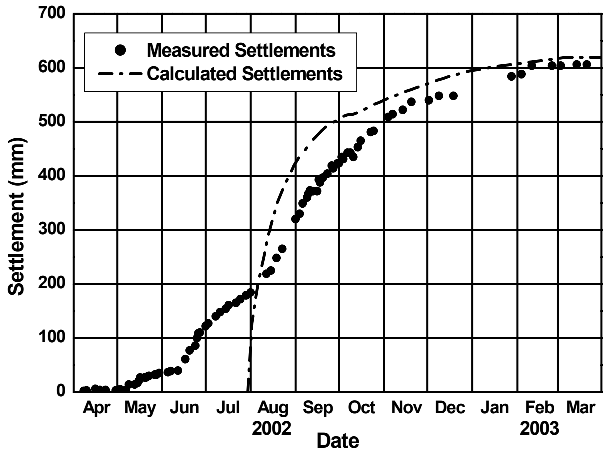

Fig. 9 *Settlements behind Wall during Surcharge Placement*

This reassessment benefited from the fact that pore pressure measurements were made during placement of the surcharge. This was in contrast to the initial assessment, described above, for which only settlement records and laboratory test results were available. This enabled a more confident distinction to be made between primary and secondary compression.

The piezometers indicated that 90 % pore pressure dissipation was occurring in about a week, implying a value of the coefficient of horizontal consolidation, C_h, of about 20 m^2/year. Bearing in mind that horizontal permeability is frequently greater than the vertical permeability, this value is considered to be reasonably consistent with that assessed for design (Table 3).

The measured rate of settlement after full dissipation of pore pressure, interpreted as secondary compression, was consistent with the design value of C_α of 0.025. However, the rate of settlement during pore pressure dissipation, interpreted as primary compression, implied a value of m_v of about 0.75 m^2/MN, which was significantly smaller than the value used for design. It is considered that there were two possible reasons for this change:

1. The soil had become less compressible as stress levels increased during consolidation;
2. The secondary compression that occurred between 1994 and 2002 had caused an apparent overconsolidation within the soil

The consequence of this reassessment was that estimated settlements after removal of the surcharge were increased. To achieve acceptable settlements during the 120 year design life, it was considered that it would be necessary to place an additional 1 to 1.5 m of surcharge. The fact that lateral movements of the wall had been much smaller than calculated gave confidence that this would be feasible. Again, the surcharge was placed in 0.5 m increments, so that movements of the wall could be assessed before the next increment was placed.

The additional increments of surcharge caused significant increases in both the lateral movement of the wall and the settlement behind it. The first increment caused a lateral movement of about 10 mm, which stabilised very rapidly, but the next two increments both caused movements of 15 to 20 mm, and movements continued after the third increment at an initial rate of about 1.5 mm per day, gradually slowing over the next month and then stabilising at about 0.3 mm per day. After two months, the Amber Limit of 105 mm was exceeded, but it was considered acceptable to allow movement to continue, as it was so slow and consistent. This was necessary in order to achieve sufficient settlement behind the wall to reduce the calculated long-term settlements to acceptable values. Full pore pressure dissipation took about two months, which was slower than previously measured, presumably because the permeability of the soils reduced as stress levels increased. The target settlement of about 550 mm was achieved by the end of 2002, although, for practical reasons, the surcharge was not fully removed until early April 2003.

During placement of the additional surcharge, further movements were observed on the markers for the railway. Surprisingly, these movements were towards the surcharge. It is considered that the original movements of the railway markers may have set up a preferential mechanism of movement. It was decided to remove some of the surcharge over the northbound carriageway (adjacent to the railway), and this appeared to be effective in slowing the rate of movement of the markers to acceptable values.

After the full surcharge was removed, the wall moved back towards the railway by about 12 mm over a period of about six weeks. Virtually no movement has occurred since late May 2003.

CONCLUSIONS

A successful solution to the problem of stabilising Canal Tied Wall was achieved by means of a "soft" solution that reduced the load on the wall. The use of this solution made significant costs savings to the scheme that had originally been postulated using a piled raft. Reduced usage of raw materials and use of surplus spoil as surcharge material contributed to the overall sustainability of this solution.

However the design and construction were made difficult by the interaction of three elements: the wall itself, the settlement of fill behind it and the constraints imposed by the adjacent railway.

The lateral movements realised in practice were significantly smaller than those predicted using even relatively sophisticated modelling even though the modelling had been calibrated using data gathered during the advance works. This was as a result of changes to the construction sequence made on site. Whilst these were relatively minor the effect on the movements appears to have been significant.

Interpretation of consolidation tests proved to be difficult even with the benefit of quite extensive settlement monitoring during and after the advance works. Several possible combinations of parameters gave an equally good fit to the data.

The flexibility provided by the use of the observational method enabled necessary changes to the surcharge design to be made during construction, while maintaining control over the stability of the wall.

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