Assessment of Settlement due to EPBM tunnelling in London UK on CTRL Contract 240

Michael Francis, KBR, UK
Keith Jones, KBR, UK

ABSTRACT

Contract 240 of the Channel Tunnel Rail Link comprises construction of twin 4.7 km long running tunnels through a densely populated area of East London. The tunnels were constructed by EPBM in Sands and Sandy Clays, passing 4.4 m below the existing London Underground tunnels, and at depths of 20-30m below ground level beneath the Great Eastern railway lines into London Liverpool Street and a series of railway bridges and hundreds of buildings. This paper describes how the project team managed the challenges relating to the issue of settlement: mitigating settlement effects, prioritising the structures to be assessed, controlling resource expenditure by incentives and flexible response to urgent work relating to unforeseen events such as ground collapse and discovery of unusual structures.
1. THE CTRL PROJECT

The Channel Tunnel Rail Link (CTRL) Project is a 108 km long high speed railway which will connect the Channel Tunnel linking England and France to St Pancras Station in the centre of London. Section 1 of the line between Ebbsfleet on the outskirts of London and the Channel Tunnel is already completed. Section 2, of which Contract 240 forms a 4.7 km part, passes beneath a densely populated area of East London. CTRL Contract 240 comprises construction of twin 7.15 metre internal diameter tunnels driven from the location of the future Stratford Station eastward to a reception shaft at Barrington Road, with one intermediate deep shaft for ventilation of the finished structure, and 8 cross passages. The tunnels are separated by 15-20m in plan.

![Figure 1 - Location of CTRL 240, East London](image)

London & Continental Railways (LCR) holds the franchise to build and operate CTRL. Rail Link Engineering (RLE) is the Project Manager and Designer of the scheme. Union Railways North (URN) is the subsidiary of LCR responsible for building Section 2 and has appointed Costain Skanska Bachy JV (CSB) for the £120M Contract 240. KBR was selected as CSB’s designer responsible for assessment of tunnelling induced settlement, its impact on structures and design of measures to mitigate settlement effects and to support sensitive structures.
2. GEOLOGICAL AND ENVIRONMENTAL SETTING

2.1 Geology

The London Basin is a synclinal structure underlain by chalk aquifers, which outcrop in hills to the north and south. Above the chalk lies a sequence of sedimentary strata: Thanet Sands, overlain by the Lambeth group (formerly Woolwich and Reading Beds), then the London Clay and finally more recent Terrace Gravels.

The tunnels, at around 15 - 30 m depth below surface, are constructed mostly in the Thanet Sands which comprise a dense fine gained silty sand, and are usually water bearing. Extensive dewatering of the ground was carried out by means of deep wells into the chalk aquifer to permit construction of the ventilation shafts and to aid tunnelling. The reduction of water levels to tunnel crown or below throughout the route reduced the need to work in compressed air for maintenance interventions on the TBM head. It also enabled the open face excavation of the cross passages without techniques such as ground freezing. This has brought significant benefits for tunnelling such as tighter control of face pressure and reduced settlement.

2.2 Structures

The tunnels in Contract 240 pass beneath more than 800 structures, which include:

- London Underground (LUL) Central Line, among London’s oldest underground railway lines
- The Great Eastern railway line and associated railway over- and underbridges
- Numerous commercial, public and residential buildings

The buildings are of various forms ranging from modern steel and concrete buildings of up to 9 storeys on piled foundations, to poor quality Victorian era brick structures on shallow foundations. Land usage includes industrial, commercial to public and residential.
3. TUNNEL CONSTRUCTION

Excavation of the tunnels was carried out by two Earth Pressure Balance Machines (EPBM) supplied by Wirth to meet a rigorous specification laid down by the Client and enhanced by CSB. The machines were purpose-made to suit the project geology and requirements for minimising surface settlement in this built-up area. To maximise steerage flexibility and minimise settlement, the shield has active front articulation to allow small corrections to alignment as well as permitting the cutterhead to be retracted from the face, and rear passive articulation to allow the tailskin to follow the cut. The cutterhead is mounted 7.5mm above the axis of the shield to give minimal over-cut in the invert, and the three shield cans reduce slightly in diameter. To further reduce ground loss over the shield body, and thus surface settlement, injection ports were installed in the front shield and in particularly sensitive areas filler was pumped into the small void above the shield. An automatic tailskin grouting system monitors the pressure of the grout (a traditional sand, cement, PFA mix with retarder and plasticiser) in the annulus void and controls the pumps. Ten precast concrete segments comprise each ring which was built on advance of the EPBM.

The eight arm cutterhead is driven hydraulically at 0-3 rpm and can develop a maximum torque of 15,525kN. The cutterhead is fitted with teeth, but can be fitted with discs (from the rear) if required, and monitoring devices on certain of the cutting teeth provide an early warning of wear. A foam generation plant produces even quality foam at variable flow rates, injecting into eight foam ports evenly distributed across a diameter and through static mixing bars in the cutting chamber. Polymer injection can take place into the foam or into separate water feeds directly into the cutterhead. Water from the site dewatering is used both for the spoil conditioning and the TBM cooling water supply.

4. METHODOLOGY FOR SETTLEMENT ASSESSMENTS
With more than 800 structures potentially affected it was important to ensure that all structures were assessed in an appropriate way so that damage to structures could be mitigated. Settlement assessment was carried out in two stages, considering at each stage the combined and separate effects of each tunnel drive.

All structures were subject to a first stage assessment carried out using green field settlement calculations, according to the criteria given by CIRIA\(^{(i)}\). This is a semi-empirical evaluation of the likelihood of excessive damage occurring to surface structures from tunnelling activities. The result of this assessment is a risk level for each point on the surface within the anticipated settlement trough, derived from predicted global vertical settlements and maximum slopes. The risk levels range from “negligible”, implying that not even superficial damage would be expected, to “high”, implying that structural damage to buildings and pipelines might be expected.

A second stage assessment was carried out on selected structures. The selection was based on the first stage assessment risk profile and other specific relevant factors, such as the nature, condition and function of each structure. The second stage assessment was carried out using the methods of Boscardin and Cording\(^{(ii)}\). Ground strains (horizontal strain and angular deformation) were evaluated at the location of the structure for each tunnel drive. Ground strains were evaluated in the green-field condition, conservatively assuming no contribution from the stiffness of the structure foundations or other adjacent features, and. Building materials are assumed in this method to be weak in tension and brittle, representing unreinforced masonry. The Contract required assessment of damage effects for a face loss value of 2%. In practice, typical values achieved were 1% or less, and with suitable operational controls on tunnelling could be maintained at below 0.5%.
The ground strain values for each individual tunnel drive and for the combined effect of the two drives were plotted on an interaction chart to give an indication of the level of damage which might reasonably be expected in a “conventional” masonry building at that location. Damage levels are categorised as one of five levels, based on I Struct E\(^{(iii)}\) and O'Rourke et al\(^{(iv)}\):

<table>
<thead>
<tr>
<th>Damage Category</th>
<th>Degree of damage</th>
<th>Description (ease of repair)</th>
<th>Approx. crack width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks of less than about 0.1mm width are classed as negligible</td>
<td>0.1</td>
</tr>
<tr>
<td>1</td>
<td>Slight</td>
<td>Perhaps isolated slight fracturing in building. Cracks in external brickwork visible on close inspection. <em>Fine cracks which can easily be treated during normal decoration.</em></td>
<td>&gt;1.0</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>Several slight fractures showing inside of building. Cracks are visible externally. Doors and windows may stick slightly. <em>Cracks easily filled, redecoration probably required. Some re-pointing may be required externally to ensure weather-tightness.</em></td>
<td>&lt;5.0</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. <em>The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.</em></td>
<td>5-15 or a number of cracks &gt;3.0</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. <em>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.</em></td>
<td>15 to 25 but also depends on number of cracks</td>
</tr>
<tr>
<td>5</td>
<td>Very Severe</td>
<td>Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. <em>This requires a major repair involving a partial or complete rebuilding.</em></td>
<td>Usually &gt;25.0, but dependant on number of cracks</td>
</tr>
</tbody>
</table>

In accordance with the Contract, mitigation was required where the anticipated damage level reached 2 (slight). Mitigation measures might include restriction of excavation face loss and other below-ground measures, shoring and other surface actions, or combinations of below-ground and surface measures. Throughout the tunnel drives, surface ground movements were monitored using a series of survey stations established across the settlement trough.
All relevant structures were inspected before and after tunnelling to identify pre-existing and tunnel-related damage. Topographic surveys quantified surface settlements due to the tunnel excavation, these were compared against the calculated values.

5. CASE STUDIES

This section presents some of the structures for which assessment was undertaken. Examples are taken from a variety of structural types to illustrate the range of structures encountered, also to demonstrate the way in which KBR and CSB were able to model the behaviour of real and occasionally unconventional structures.

5.1 LUL Central Line

The CTRL tunnels pass only 4.4 m beneath the 4m diameter LUL Central Line tunnels, which were built in the 1930s using tapered cast iron segments with machined joint faces and no packing in any joints. The tunnels were considered to be both brittle and rigid in both cross and long section. The effect of settlement due to tunnelling was therefore likely to be very serious once the very limited flexural capacity was exceeded. CSB predicted ground loss would be less than 0.5% based on experience taking into account the settlement minimisation measures designed into the machines. This was accepted by LUL and the designers.
Although both Contractor and LUL accepted that it should be possible to achieve face loss values of less than 0.5%, it was recognised that the crossing would be made after only 30m of the drive had been made. It was therefore prudent to assume a greater face loss percentage than 0.5%. Extensive and refined analysis showed that by slackening the circle bolts and thereby introducing longitudinal discontinuities at roughly 4m centres the tunnel would no longer be in danger of breaking its back over the new tunnel drives. Removing the 70 year old bolts was not always easy as some were encased in the concrete of the trackbed.

Movement of the Central Line tunnels was monitored using electrolevels, displacement transducers and DEMEC studs placed across the now released circle joints, and in cross section using electrolevels and convergence arrays. The crown and the rails were monitored using precise levelling techniques. A full emergency plan and cascade of reactions to monitoring was established and a hot line set up between the
monitoring review team and the line controller. The tunnels passed beneath the Central Line with an observed maximum settlement of approximately 10mm. This corresponded to a face loss value of approximately 0.44%.

5.2 High Street North Bridge
The bridge, built circa 1840, is a four span continuous structure, with spans of approximately 9 metres and a deck width of 16.5 metres. The structure carries High Street North over the railway and other local roads. The deck is of jack-arch construction, with 11 wrought iron longitudinal beams, and brick arches spanning between the bottom flanges of the beams. End-spans bear on steel packers and padstones on the massive abutments. Intermediate support is provided by steel trestles, with the girders bearing directly on onto the trestle structure. The bridge axis is approximately perpendicular to the tunnel axis.

The conventional damage categorisation method described above is derived from observation on brittle masonry buildings. This method was not considered appropriate for this structure due to the continuity and configuration of the structure. The bridge was therefore assessed by modelling imposed support deflections on the continuous structure to represent the transverse troughs according to the tunnel excavation sequence. Resulting stresses and movements were examined and confirmed to be within acceptable limits. The abutments were also modelled by imposing ground movements corresponding to the longitudinal settlement wave following the TBM. On-site level monitoring confirmed that the actual movements were within the bounds assumed in the analysis.

5.3 University of East London Arthur Edwards Building
This structure is a 6 storey block comprising a rectangular ground and first floor approximately 31m x 40m, with the upper four floors having reduced plan areas of about 16m x 40 m. The column grid is generally at 3.6m centres in one direction and up to 8.2m in the other. The structure comprises an in-situ
reinforced concrete frame and floor slabs, and an external cladding of pre-cast concrete panels and windows. The internal dividing walls are mainly of exposed brickwork at the ground floor, and of plastered blockwork on the other floors.

There was little specific detail of the foundations. However, based on the recommendations of a geotechnical report prepared prior to building construction and a consideration of building form it was assumed for the purposes of the assessment to be founded on piles which would be no longer than 13 m long, and which would most probably have been 500mm diameter, augered construction. The toe level of the piles would therefore have considerable clearance from the new tunnels to be constructed at 30 m depth.

Although the assessment method described above was derived from simple masonry structures, it was possible to adopt similar principles in assessing this modern, framed, reinforced concrete building. A series of analyses was carried out to establish the ground movements at the supposed foundation level of the building. A range of effective founding depths was used, from the level of the toe of the piles to surface level. Damage levels were computed using the Boscardin & Cording/O’Rourke et al methods, which were considered to be conservative due to the inherent ductility of a modern framed structure. Control of face loss and associated monitoring of surface settlement profiles on the approach to the building were adopted as mitigation measures. Specific monitoring was carried out at sensitive areas of the building, such as glazing and corners of partitions. No damage was recorded.

5.4 Coronet Public House, the Grove Post Office and Lavender Street
The Coronet was a Victorian, two-storey brick-built Public House, with an extensive cellar, large open-plan bar area, kitchens and other ancillary areas on the ground floor, and a series of individual lodgings in the upper storey. It is the corner property in a terrace of residential and commercial buildings. During the
The usage and configuration of the structure would change significantly, with an extra storey to be added, and extensive remodelling of the interior. Programme details for the redevelopment were not readily available, but by the time the tunnel construction commenced, development work was imminent. Adjacent to the Coronet is a terrace of three storey Victorian properties (The Grove), with another parallel terrace (Lavender Street) set approximately 40m further back.

Figure 3 - Coronet PH, The Grove and Lavender Street location plan

Stage 2 assessment of the Coronet, carried out in advance of the tunnelling work, indicated that some light internal reinforcement of weak corners and support of a bulging section of wall was required, although it was recommended that the structure be monitored for movement and growth of pre-existing cracks. However, as the first tunnel drive approached the building, the development contractor began work in earnest. Without access to the inside of the boarded-up structure, CSB and KBR stepped up monitoring of the structure from the outside. Access was finally made available, and the extent of internal demolition uncovered, when the development contractor claimed that a window frame had fallen in due to tunnelling activity. The building was urgently shored up by CSB to permit the passage of the TBM, while the situation was assessed. The developers had removed entire floor plates and internal shear
walls, weakening the building. This had been done concurrent with the tunnelling work, progress of which was very public knowledge in the locality. The CSB emergency shoring was to remain in place until the developer had reinstated the floor plates and provided adequate shear walls to ensure the final stability of the building. KBR’s photographic records also showed that the window frame in question had been detached from the façade for some months. Inspection of other windows showed that, by removing the plaster around this frame, the developer had removed all restraint, and that window had thus probably simply blown in during strong winds.

Assessment of the adjacent structures had confirmed that no specific mitigation against ground movement was required. However, once the first TBM drive had passed beneath The Grove and reached Lavender Street, a surface void began to open up between the two terraces. This rapidly expanded until a hole estimated as up to 8m maximum depth and 10m diameter had formed. The adjoining properties were evacuated, and the void stabilised by pouring approximately 300m$^3$ of concrete into it. Subsequently, further stabilisation was achieved by a campaign of grout injection. As part of a pre-planned response,

Figure 4 - The Coronet prior to and during redevelopment (shoring still in place, all but façade removed)
KBR assisted the construction team with emergency inspection and monitoring of properties prior to allowing residents to return, as well as with longer-term monitoring of one particular property, a Post Office adjacent to the Coronet. This building had clearly suffered long-term settlement in the past, evidenced by the repairs which were clearly visible in the structure and finishes, although this movement now appeared to be relatively slow. In the months following the ground collapse, pre-existing and new cracks opened up significantly, and were monitored to demonstrate that the rate of movement stabilised.

The cause of this collapse is conjectured to be the historic failure of one or more deep well shafts, which had been effectively bridged by a clay layer near the surface, forming a buried conical void. Ground movements due to tunnelling are thought to have broken the clay bridge, causing the overlying surface layers to collapse into the pre-existing void. The evidence of historic movement in adjacent buildings, which was reawakened by the sudden surface failure supports this hypothesis, although extensive reviews of historic records carried out prior to construction, and redoubled after the failure, did not find any record of the existence of such shafts. Following the collapse, an intensive campaign of ground investigation using techniques such as Ground-Penetrating Radar was undertaken, this identified a further, small abandoned shaft in a garden close by.

6. CONTRACTUAL RELATIONSHIPS

The project was executed under the New Engineering Contract (NEC). CSB entered into a partnering agreement with URN and other contractors in Section 2 of the CTRL project. This was designed to share risk and ensure that no section of the project would delay overall completion.

KBR was also engaged under the NEC form of contract Professional Services Contract (PSC). It was recognised before starting work that the scope of KBR’s work was difficult to predict and as a result a target cost incentive was established. A target price was agreed based on an estimate of the man hours at
approved rates. A fixed element of payment was made, spread over the first year, with further monthly payments based on hours worked and rates to make up the remainder of the target value. This ensured that everyone involved was motivated to manage the scope of work to a reasonable level consistent with meeting the requirements of the contract and lead to an excellent working atmosphere. KBR’s activities for CSB came within budget, and thus offered savings for both parties.

7. SUMMARY AND CONCLUSIONS

The construction of 4.7 km of twin bore railway tunnels through east London potentially affecting approximately 800 structures has presented significant challenges to the contractor and his engineer responsible for assessing settlement impacts.

In order to address the large number of structures the project team has:

- Adopted simple and rapid methods to determine where there may be significant impacts
- Adapted the simple methods and worked in more detail on selected structures.
- Collaborated with site team to establish actual construction operations and achievable parameters, and to prioritise work accordingly.
- Included site visits and detailed inspections, making some surprise discoveries in the process.
- Responded quickly to changes imposed by uncontrollable third party activities
- Acted on public concerns, ranging from actual or potential damage to properties, to complaints about noise from passenger trains in the new tunnel even before the tunnels were excavated.

The team has learnt that even exhaustive desk study and site-walking will not identify all hidden features which may affect progress and impact on the public, and to expect the unexpected. The challenges have been met through collaborative working relationships between project manager, contractor and those making the structural assessments. In particular, CSB personnel working on the project have, through
their collaborative approach, enabled KBR to effectively assess impact of tunnelling induced settlement and to design appropriate mitigation in a timely manner. The contractual framework and clear working relationships were a firm basis for this collaboration and the exceptional achievements of the project.

REFERENCES

(i) CIRIA, (1996) Project Report 30: Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas

