Damage assessment and monitoring for buildings on the Elizabeth line

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NOTATION

BRE	Building Research Establishment
SCL	sprayed concrete lining
TAM	*tube à manchette*
TBM	tunnel boring machine

Synopsis

The Elizabeth line, due to open in December 2018, crosses London from west to east. The Crossrail project to construct the Elizabeth line has seen 21km of twin-bored tunnels constructed under central London, with eight new stations built on this section.

The damage assessment and monitoring carried out comprised a significant element of work in terms of the resources involved, both human and financial. The background to this work was the experience from a number of tunnelling projects in London, probably most significantly that from the London Underground Jubilee line extension. While all assets along the alignment were subject to the same process, the impact of the works around the stations and shafts was calculated to be greater than along the bored tunnels, and the extent of instrumentation and monitoring was correspondingly higher. Both automated with instrumentation installed and readily visible on many buildings in these areas throughout the duration of the works.

This paper looks at the damage assessment and monitoring of buildings around the stations, focusing in particular on the new station at Tottenham Court Road. It also provides an overview of the two very different tunnel construction methods used on the project – the so-called tunnel boring machine (TBM) and sprayed concrete lining (SCL) methods – and describes how these lead to the ground movement that is the principal source of potential damage to the buildings.

Finally, the paper considers briefly some of the lessons learned and how these might be applied to future urban tunnelling projects.

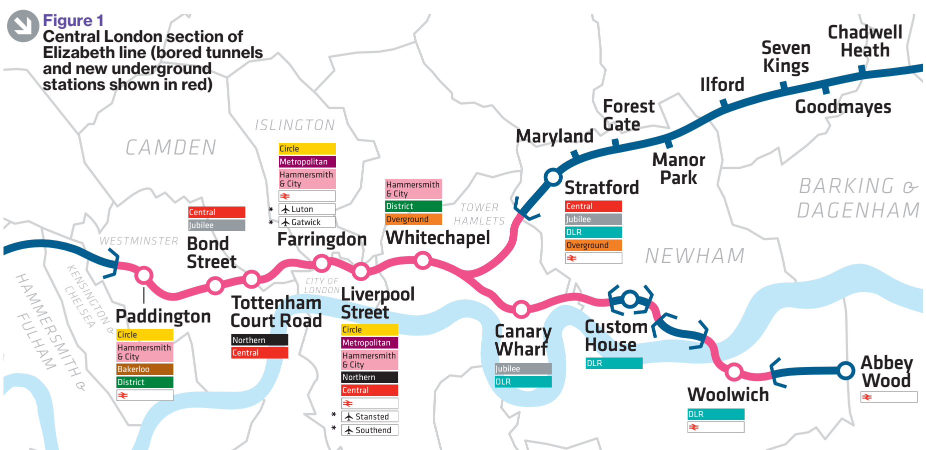

Figure 1 Central London section of Elizabeth line (bored tunnels and new underground stations shown in red)

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Introduction

The Elizabeth line, due to open in December 2018, crosses London from west to east. The project has seen 21km of twin-bored tunnels constructed under central London, with eight new stations built on this section of the line (Figure 1). It should be noted at the outset that this was a major achievement: tunnelling below crowded streets was completed with little indication above ground, other than instrumentation on buildings, of what was happening below. Some aspects of how this was achieved are described below.

The construction of tunnels, even using the most modern machinery and control methods, still results in some volume loss and corresponding ground movement. The impact of this movement on the assets, both above and below ground, can be assessed to varying levels of accuracy and the likely degree of damage predicted.

Damage assessment followed the process set out in Crossrail Information Paper D12[1] and relevant Crossrail Civil Engineering Design Standards, which in turn had been developed from earlier work on the London Underground Jubilee line and High Speed 1. This process covered assets including buildings (both low-rise masonry structures on shallow foundations and taller, framed structures on piled foundations), other structures and statutory services. Considering buildings alone, there were approx. 4000 buildings along the route, of which around 300 were listed.

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Figure 2
TBM cutter head, seen on two machines (named Ada and Phyllis) at Western Portal at start of tunnelling

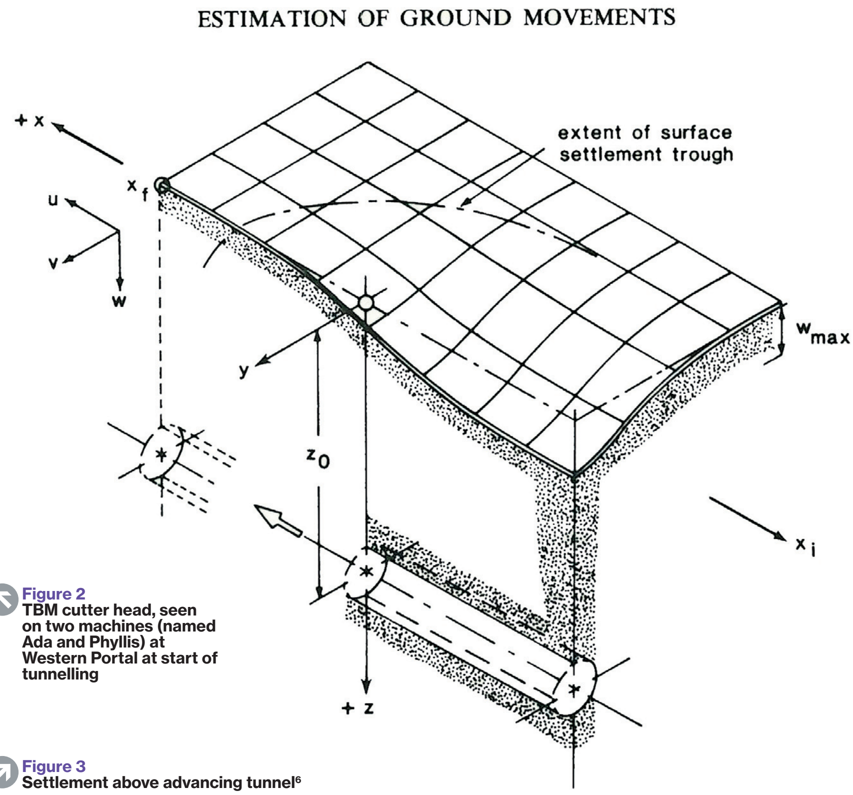

Figure 3
Settlement above advancing tunnel[6]

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This paper looks at the damage assessment and monitoring of buildings around the stations, focusing in particular on the new station at Tottenham Court Road; these included a number which were listed, some over 300 years old. Inevitably, for a project of this size and complexity, there are areas which have had to be omitted or covered only briefly. Further information may be found in other publications, notably *Crossrail Project: Infrastructure Design and Construction*[2–5] which comprises four volumes of papers.

The paper also provides an overview of the two very different tunnel construction methods used on the project and describes how these lead to the ground movement that is the principal source of potential damage to buildings.

The paper then explains the three-phase damage assessment method used on the project, including the approach used for heritage assessment and protection. The approach to mitigation of the impacts of ground movement is described, including the extensive monitoring of both the ground and assets along the alignment. The process of compensation grouting, which was used widely as mitigation around the stations, is explained. Examples of buildings around the new Tottenham Court Road station are then used to describe in more detail some of the mitigation measures adopted during construction, the monitoring installation and monitoring results.

Finally, the paper considers briefly some of the lessons learned and how these might be applied to future urban tunnelling projects.

Tunnel construction methods

Two different construction methods were used for the construction of Elizabeth line tunnels: tunnel boring machine (TBM) and sprayed concrete lining (SCL).

There are two different types of TBM: the earth pressure-balanced TBM and slurry TBM. The selection of the appropriate TBM is dependent on the ground conditions. In the London Clay along most of the western part of the route, the earth pressure-balanced TBM was used; and for the tunnels driving through chalk, a slurry TBM was used. More detailed information can be found in specialist publications.

The TBM is equipped with a rotating cutter head (Figure 2) at the front of the machine's steel shield body. The machine is designed to apply face pressure to the excavated ground face so as to balance earth and groundwater pressure until the (permanent) tunnel lining is constructed. Precast concrete segment rings are assembled at the back of the TBM to support the ground, and the TBM pushes against the ready-built ring to move forward.

SCL excavation (ground mining) is completely different and is carried out with the use of excavators. The tunnel section is excavated for a short length (in London Clay, typically 1m), and shortly after the excavation, sprayed concrete is applied to the exposed ground to provide ground support. The process is then repeated, with successive excavation and sprayed concrete application cycles. When the tunnel section is large and full-face excavation is considered unstable, it can be excavated by dividing it into several smaller sections to limit the size of unsupported ground.

On the Crossrail project, TBMs were used for the construction of the running tunnels (internal diameter 6.2m) between the stations, and SCL was used for the construction of station tunnels such as platforms, concourses (internal diameter approx. 9m) and cross-passages (internal diameter approx. 6m). The running tunnels are broadly 20–35m below ground level between the stations. At some of the stations, due to the various constraints in the construction programme, the TBM drove through the platform tunnels before SCL excavation started. In these locations, the bored tunnels were then subsequently enlarged by SCL to the final profile.

When a tunnel is excavated, the ground loses force equilibrium around the tunnel and thus the ground deforms. The face pressure (in the case of TBM) and tunnel lining (segment lining or SCL) provide support to the ground, which can limit the ground movement, but in soft ground such as London Clay (as opposed to rock), it is not possible to construct tunnels with zero ground movement. This is due to the fact that deformation of the ground moves ahead of the excavation face (Figure 3)[6], and also that there is always a time gap between the excavation and the construction of the lining, resulting in further ground movement.

Global best practice widely accepts that tunnelling-induced ground movements in soft ground can be estimated by assuming the settlement trough fits the Gaussian probability curve (perpendicular to the tunnel drive) and the cumulative probability curve (parallel to the tunnel drive). The buildings are assumed to deform following the predicted ground settlement trough (known as the 'greenfield' settlement profile).

Movements along the tunnel alignment were generally predicted to be small, with correspondingly minor impacts on buildings. This correlated with the results recorded during the works; volume losses (Figure 4), particularly on the western drive (between Paddington and Farringdon) through London Clay, were generally lower (<0.5%) than the fairly conservative value of 1% assumed in the damage assessment calculations. Around the stations, the predicted values were higher; horizontal movements around the deep excavations were greater than those around the bored tunnels, and the larger platform tunnels and cross-passages were constructed using SCL techniques which also produce larger movements. Volume losses for SCL works were assumed to be 1.5% for the purposes of assessment. Further explanation of volume loss can be found in Burland[7].

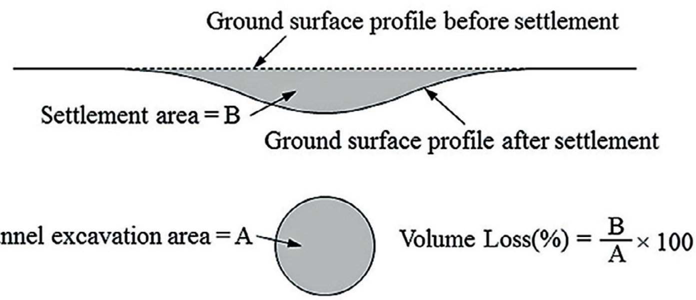

Figure 4 Definition of volume loss

Classification and assessment of building damage

Building damage classification

While the focus of this paper is 'damage', it is worth considering what is meant by this term. Damage is a highly subjective and often emotive subject; in relation to buildings, this is perhaps the case particularly where the perception is that the damage has been caused by the actions of others. It may relate to aesthetics, to function and serviceability – the serviceability limit state – or in more extreme cases of structural damage (with a possible risk of instability) – the ultimate limit state. Most buildings experience some degree of cracking at some stage, often in finishes, but might not be regarded as 'damaged'.

On the Crossrail project, it was recognised at the outset that the works would result in some degree of ground movement and that some damage was predictable – often seen as cracking, but with the potential for other consequences such as jamming of doors or windows. The classification of damage followed the procedure set out in Burland *et al.*[8] and Mair *et al.*[9]. Engineers will be familiar with the so-called 'Burland' classification described in BRE Digest 251[10]. For listed assets, an additional score was assigned to account for building sensitivity. Tables 1 and 2, reproduced from Crossrail Information Paper D12[1], show the values that were used to provide an overall risk level.

Damage assessment process

For the purposes of this paper, the process described is necessarily simplified to some degree, but it is intended that sufficient information is provided, together with appropriate references for further detail where required.

For all assets that were located within a zone such that they might be affected by the works, given in Crossrail Civil Engineering Design Standards Part 8[11] as those located within the 1mm settlement contour, a three-phase damage assessment process was set out as summarised below.

The standard methodology for building damage assessment adopted on the Crossrail project refers to a number of research papers and the methodology used on projects such as the Jubilee line extension, which assumed that buildings behaved as elastic beams and moved as per greenfield ground movements. Full references are included in Crossrail Information Paper D12[1]. The classification is considered to be conservative for many of the buildings, as it is based on case studies of loadbearing masonry buildings on shallow foundations. Framed buildings are considered to be more robust, but this is not quantified within the methodology. For buildings on piled foundations, an alternative methodology is adopted which considers settlements calculated at three different levels along the length of the piles.

Phase 1

Simple criteria (predicted settlement from bored tunnels or from the excavations less than 10mm and predicted ground slope less than 1/500) were used to eliminate buildings subjected to minimal effects. This set the limiting criterion as Damage Category 1

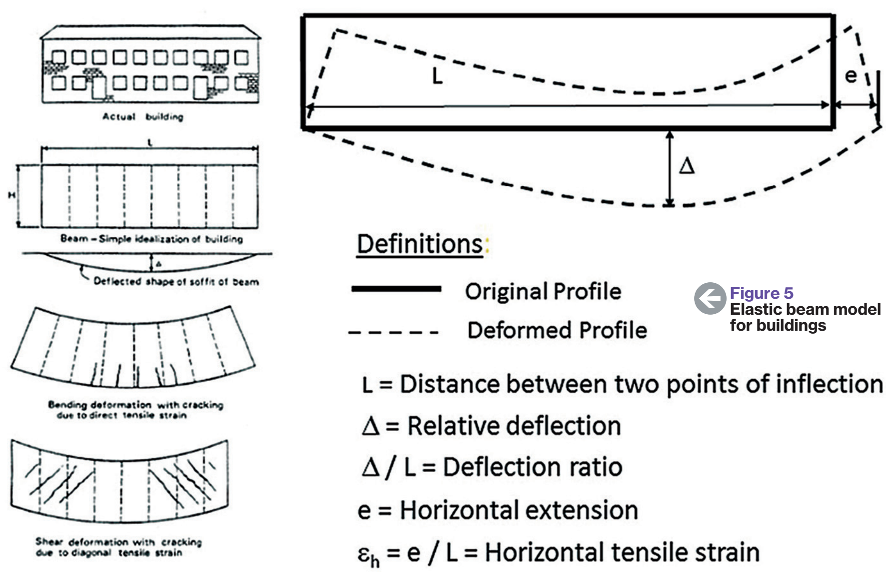

Figure 5 Elastic beam model for buildings

TABLE 1: BUILDING DAMAGE CLASSIFICATION*[1]

Risk category	Max tensile strain (%)	Description of degree of damage	Description of typical damage and likely form of repair for typical masonry buildings	Approx. crack width† (mm)
0	0.05 or less	Negligible	Hairline cracks	
1	>0.05 and ≤0.075	Very slight	Fine cracks easily treated during normal redecorations. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection	0.1 to 1
2	>0.075 and ≤0.15	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible; some repointing may be required for weathertightness. Doors and windows may stick slightly	1 to 5
3	>0.15 and ≤0.3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Repointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weathertightness often impaired	5 to 15 or a number of cracks greater than 3
4	>0.3	Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility services disrupted	15 to 25 but also depends on number of cracks
5		Very severe	Major repair required involving partial or complete reconstruction. Beams lose bearing, walls lean badly and require shoring. Windows broken by distortion. Danger of instability	Usually greater than 25 but depends on number of cracks

* Based on work of Burland *et al.* (1977)[8] and includes typical maximum tensile strains for the various damage categories (column 2) used in Phase 2 settlement analysis
† Crack width is only one aspect of damage and should not be used on its own as a direct measure of damage

('very slight') as defined by Rankin[12], and these buildings were not subject to further assessment. This phase comprised an initial screening using upper bound parameters and assumed greenfield conditions.

Phase 2

In the next phase, a generic assessment was undertaken for buildings within the 10mm settlement contour. The greenfield settlement is imposed on buildings, i.e. it is still, conservatively, assumed that the settlement behaviour is not modified by the stiffness of the building, which is taken to be completely flexible. In addition, the deformation due to horizontal ground movement is considered.

Figure 5 shows the simplified elastic beam model for the simple case where a building (represented as a two-dimensional (2D) element) is located transverse to a tunnel below and entirely within the sagging zone of the settlement trough. In practice, of course, there was great variation in the orientation of buildings in relation to the tunnel alignment and, in some cases, the eastbound and westbound running tunnels were sufficiently close so that the resulting settlement troughs had multiple sagging and hogging profiles due to the interference of the two troughs. A building's response to the settlement is also influenced by the relative location of the building in relation to the sagging or hogging profile of the settlement trough.

TABLE 2: SCORING FOR SENSITIVITY ASSESSMENT OF LISTED BUILDINGS[1]

	Criteria	
Score	**Sensitivity of structure to ground movements and interaction with adjacent buildings**	**Sensitivity to movement of particular features within building**
0	Masonry building with lime mortar not surrounded by other buildings. Uniform facades with no particular large openings	No particular sensitive features
1	Buildings of delicate structural form or buildings sandwiched between modern framed buildings which are much stiffer, perhaps with one or more significant openings	Brittle finishes, e.g. faience or tight-jointed stonework, which are susceptible to small movements and difficult to repair
2	Buildings which, by their structural form, will tend to concentrate all their movements in one location	Finishes which, if damaged, will have significant effect on heritage of building, e.g. cracks through frescos

Using the procedure described by Burland[6] and Mair *et al.*[9], the risk category for each building was assessed as defined in Table 1. For those where the category was assessed as *less* than 3, i.e. 'negligible', 'slight' or 'very slight', the assessment process was taken no further other than for:

- buildings with a foundation level >4m, or >20% of the depth to the tunnel axis for those affected by the bored tunnels
- buildings on shallow foundations and within a distance from an excavation equal to the greater of the excavated depth of superficial deposits or 50% of the total excavation depth
- listed buildings
- buildings where it was considered that further assessment was needed to determine whether protective works were required and/or what these should be.

Phase 3

In the next phase, buildings were considered individually rather than as part of an area analysed generically.

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Figure 6
Compensation grouting shaft with TAM array installed[14]

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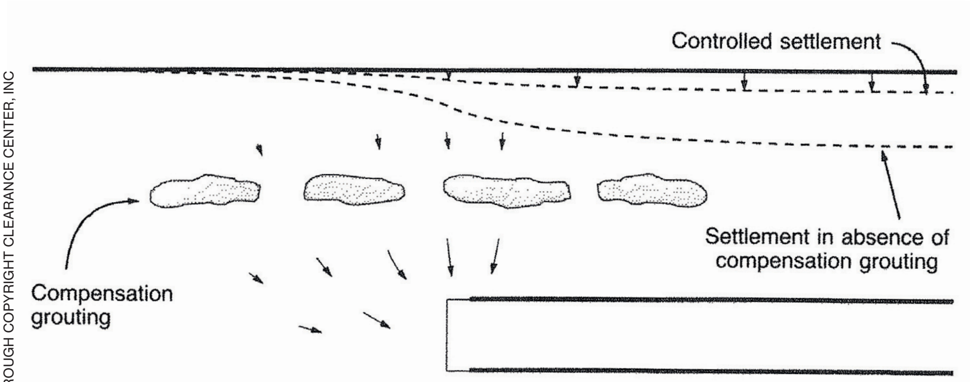


Figure 7
Principles of compensation grouting[15]

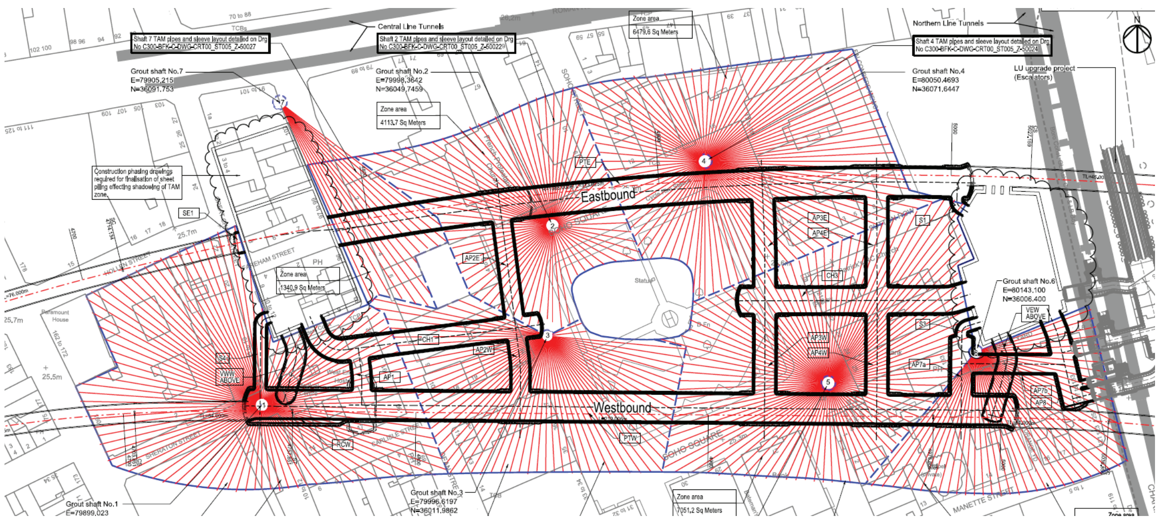


Figure 8
TAM array in Tottenham Court Road station; radially located red lines indicate TAMs from compensation grout shafts (indicated as o on plan)

The Phase 3 assessment was taken through several iterations as required, the intention being to understand whether increasing levels of accuracy would credibly reduce the risk of damage to an 'acceptable' level, with a risk category (or, for listed buildings, a total score) below 3. Refinements included numerical modelling of the soil–structure interaction in conjunction with the tunnel excavation, and also more detailed assessment of the actual structure. Visual inspections were undertaken by structural engineers, often in conjunction with built heritage specialists for the listed buildings, to determine the form of the building and its condition. In a few cases, generally associated with buildings with retained facades but also where the visual inspection identified other specific areas of concern, structural investigations were specified.

The findings from these surveys were included in the Damage Assessment Report produced for each individual building. Perhaps unsurprisingly, defects were identified in some buildings which were felt to need remedial works irrespective of the predicted impact of the Crossrail works; as a matter of professional practice, these were drawn to the attention of the building owner in a brief report, although in the majority of cases this prompted neither a response nor subsequent action by the building owner. Where it was felt that failure to carry out necessary repairs ahead of the works entailed some level of risk to the structure, these were undertaken by Crossrail.

These surveys were entirely separate from the *defect surveys* discussed below, which were undertaken on properties within the zone of influence of the works.

Listed buildings were assessed to determine their damage category in the same way as non-listed buildings. They were then, however, subject to a more detailed assessment process involving:

- agreement of methodology through consultation with English Heritage (now Historic England) and local authorities
- a desk-based study (of available information taken from archives, etc.)
- examination of damage assessment results for the listed buildings
- site visits (by structural engineers and/or heritage specialists to examine form, context (adjacent buildings), features, alterations where visible, repairs, condition)
- initial assessment (identification of sensitive features, fixtures, structure and their weaknesses)
- scoring of structural sensitivity to potential damage
- scoring of heritage sensitivity to potential damage
- identification of buildings where further assessment, mitigation or other measures might be required.

Approaches to mitigation

General approaches

Given the number of properties within the zone of influence of the works (within the 1mm settlement contour) and the range of construction types, age and use, it was reasonable to anticipate some degree of pre-existing deterioration in at least some of these. Defect surveys as described in Crossrail Information Paper D12[1] were carried out in advance of the works to provide a record of the pre-works condition as a reference for agreeing any changes which could be ascribed subsequently to the works. Around the stations and shafts, where demolition of adjacent buildings was required, the impact of these preliminary works was assessed and defect surveys were undertaken prior to the commencement.

The starting point for mitigation was the commitment by Crossrail to keep *predicted* damage levels below Category 3, consistent with 'moderate'. For listed buildings, the total score was the key parameter: this was the combination of the risk category and the sensitivity score, with this combined impact to be <3. A refinement, however, was introduced during the works which recognised that no mitigation would be required in exceptional cases where there was a very high sensitivity score but negligible damage predicted. Here, it was this combined impact which was to be <3.

The primary and (preferred) means of mitigation was to control movement at source by controls on tunnelling and excavation with contractual limits on volume loss. Monitoring, both automated and manual, of asset and ground movements was widely used, with a range of instrumentation installed route-wide.

Ground treatment was also implemented where appropriate, although this in itself also has some impact, as described below.

In a limited number of cases, repairs and/or protection were indicated prior to the works, generally due to the pre-existing condition or the presence of sensitive elements such as stone 'cantilever' stairs, which in some instances were a cause for concern.

In many instances, the preferred solution, including for listed buildings, was to allow cracking to occur and to allow repair with appropriate materials and methods once ground movements had ceased, as indicated by ongoing monitoring (see below). In the majority of cases, pre-emptive interventions were thought likely to be more intrusive and lead to greater impact on historic fabric, an approach which was agreed with the heritage authorities.

Ground treatment: compensation grouting

Where physical mitigation was indicated, the ambition was to make this non-intrusive wherever possible. Around stations and shafts, *compensation grouting* was adopted as the principal means of mitigation. While this did not control settlements to an absolute (target) value, it reduced the unmitigated movement and, more importantly, was used to limit the deflection ratio (Fig. 5) to a value consistent with damage category 1 (very slight) or less, and a maximum ground slope of 1:1000.

Compensation grouting is described in more detail in other papers (e.g. Bezuijen, 2010)[13], but a brief outline of the technique is included here. A grout shaft is installed at a specified location to enable an array of *tubes à manchette* (TAMs) to be drilled out horizontally to lengths of up to 80m from the shaft. The tubes are installed radially at a number of depths within the shaft (Figure 6). Grout is injected through a selected TAM using two rubber packers which select the part of the tube where the grout will be injected. The grout is injected at high pressure so that the ground is fractured horizontally, then the grout penetrates through the fractured cracks and heaves the ground to compensate for settlements (Figure 7).

Although compensation grouting is intended to mitigate ground movement, the installation of the grout shaft and TAMs themselves results in some ground settlement. While vertical ground movements due to the installation of the shaft are insignificant, the TAM installation process can cause settlement, in some cases of a magnitude of 10–20mm, influenced by both ground conditions and also the installation method. Settlements may also be larger in the zone closer to the shaft where the density of TAMs is much higher (see the red radial lines near the compensation shafts in Figure 8). This initial settlement is later compensated for by injecting grout through the TAMs before the main tunnel construction work starts, although this process, known as 'priming', can itself result in ground heave in excess of that anticipated.

While the grout process might be thought of as a continuous reactive process, the reality is not a smooth re-levelling, but rather a series of small step changes. Where settlements are predicted, the ground may also be lifted *in advance* by a 'jacking' process. The movements can be tightly controlled using a series of hydraulic levelling cells (Figure 9) generally installed in the basement of a building – or more accurately a group of basements nearby.

Further information on implementation of the process is provided in the case studies below.

Mitigation in buildings

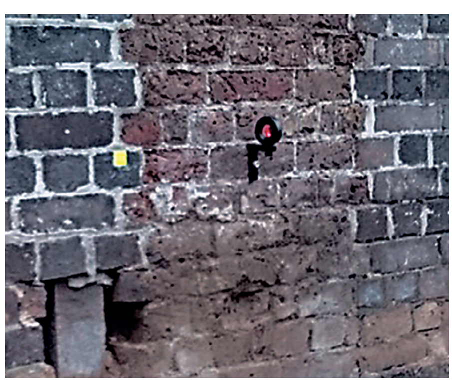
The general approach to mitigation has been described above, i.e. minimising physical intervention where possible, in conjunction with monitoring of buildings. This resulted in a relatively low level of building works, complemented by site visits when concerns were raised either by building owners/occupants or by unexpected trends in monitoring results.

Works included application of film to windows in a small number of buildings

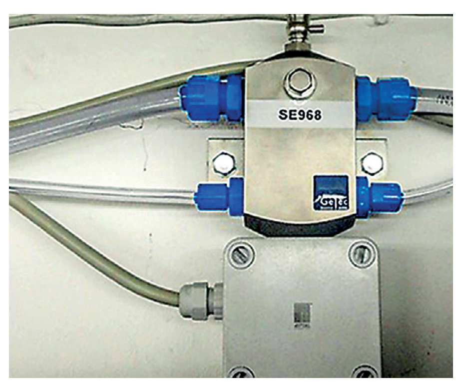
Figure 9
Monitoring techniques

a) Invar scale on building facade – used for precise manual levelling

b) Prism on building facade – part of automated monitoring installation using Automated Total Stations



c) Hydraulic levelling cell – part of automated monitoring installation used to control building slopes



where there were concerns that movements might lead to breakages, and a number of ‘protection’ schemes for elements such as stone ‘cantilever’ stairs where it was considered from inspections that there was, albeit low, some risk of collapse. In these cases, a structure was designed to be in position in the event that there was a collapse, but it was installed initially without contact to avoid imposing stresses into the element.

Repairs were carried out prior to the works in some buildings where defects were identified which required rectification more urgently. There were also repairs implemented during construction in some cases, even where it was not clear that the Crossrail works were the contributory factor: the overriding principle was to mitigate risk as far as reasonably possible.

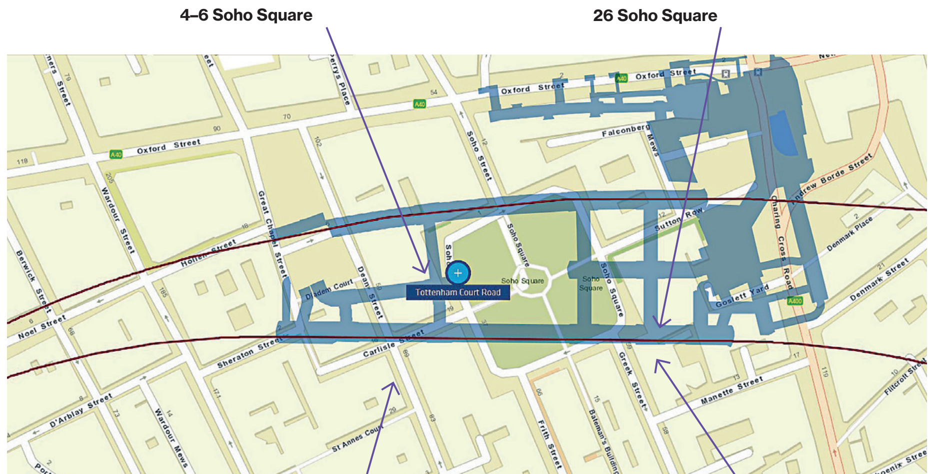


Figure 10
Locations of case study buildings[16]

Monitoring

Monitoring of both buildings and the ground was undertaken extensively across the project as part of the asset protection strategy. This provided information on when and how contingency measures should be adopted.

It confirmed that the ground and the assets were behaving as anticipated. It also both provided information for design verification and allowed construction control, providing confirmation that excavations were being implemented in a controlled manner.

Techniques used included manual monitoring using studs and Invar calibrated scales (Fig. 9a); automated monitoring of prisms (Fig. 9b); hydrostatic levelling cells (Fig. 9c) and tiltmeters; and, at a later stage, satellite technology to look at ground movements in specific areas. This is an area where there are continuing developments and some changes might be anticipated for subsequent projects.

As is common practice, a ‘traffic light’ system of trigger levels was adopted: **green** (proceed, no issues), **amber** (monitor more frequently, review calculations and start implementing contingency measures if trends continue) and **red** (a value not to be exceeded; in cases where this occurs, measures to be implemented to prevent further movements, with work suspended).

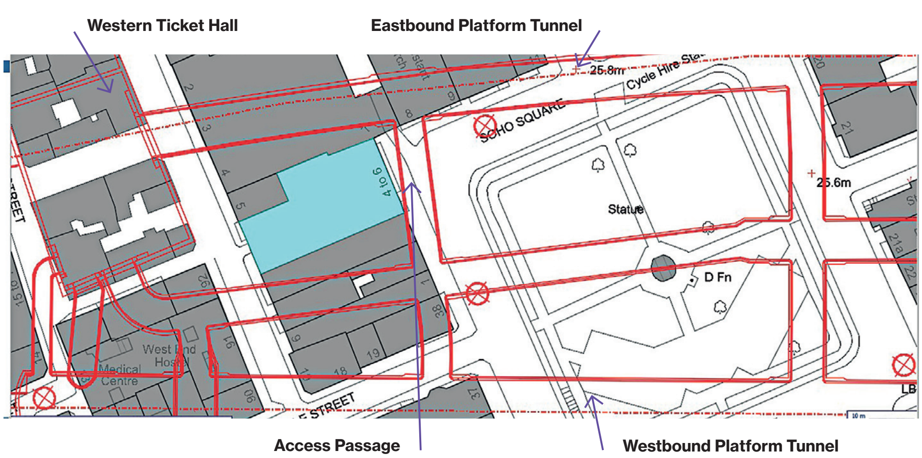
Amber values are close to those calculated from analysis, while **red** levels should be based upon an acceptable ‘damage’ criteria.

In many cases, as for other movement monitoring, it is trends that are important and any obvious cause which might underlie the increased movement should be investigated.

Case studies

Buildings around the new Tottenham Court Road station which went through the assessment process included some of the oldest on the alignment: around Soho Square and in Denmark Street, in particular, several were over 300 years old. There was also more modern construction, including Centre Point. A few examples of the older buildings are included here, with some further detail of the works carried out. Locations are shown in Figure 10.

Figure 11
Plan location of 4–6 Soho Square showing new station tunnels below



4–6 Soho Square

The building at 4–6 Soho Square is located at the northwest corner of the square (Figure 11). It is one of a number of buildings on ‘mixed foundations’, i.e. a combination of deep (piled) and shallow foundations. There is no prescribed methodology for buildings on ‘mixed’ foundations and the damage assessments were carried out on a building-specific basis. Due to its unusual form, the assessment and protection of 4–6 Soho Square are described in some detail.

The original building fronting Soho Square (Figure 12) is linked to 6 Dean Street on the west side. It was originally constructed as a warehouse *c.*1801–04 for John Trotter, ‘store-keeper general’ for army supplies during the Napoleonic wars. The warehouses were altered in 1816, when Trotter converted them into a ‘bazaar’. The shop front and ground-floor level were reconstructed *c.*1890. At that time, there was an open area between the rear of 4–6 Soho Square and the rear of 6 Dean Street. This phase of the building is of loadbearing brickwork with timber floors. It is four storeys at the front on Soho Square and three storeys at the rear on Dean Street; a vaulted brick basement with a concrete slab occupies the entire site.

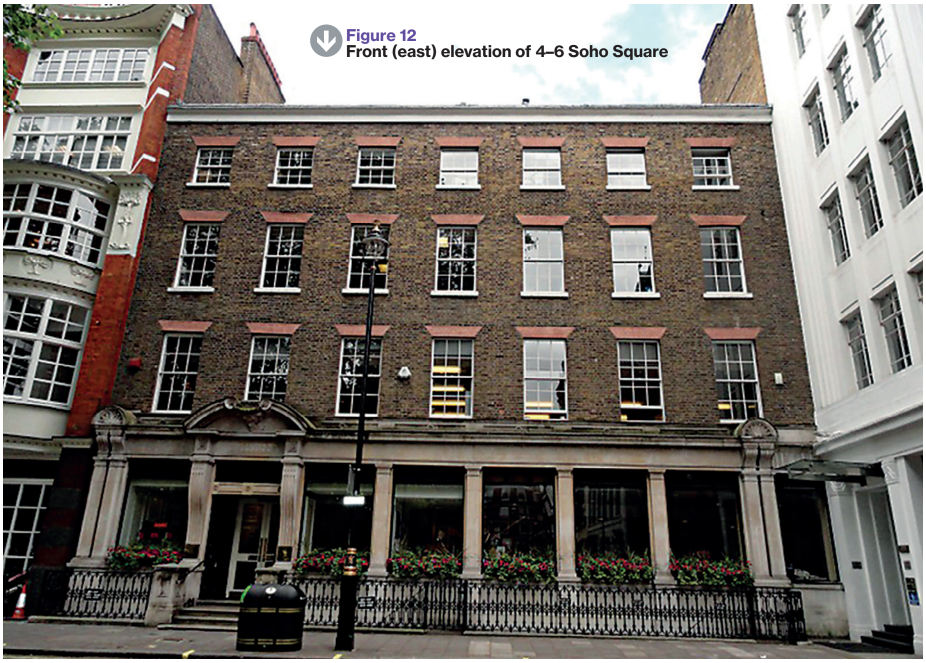


Figure 12
Front (east) elevation of 4–6 Soho Square

Alterations took place in the mid-1980s in both the basement and at the upper levels. The open area between the buildings fronting Soho Square and Dean Street was infilled to provide two wings of full-height accommodation to the north and south and a double-storey atrium in the centre. The two 'wings' are steel-framed with a mansard roof to the south wing and flat roof to the north wing. A number of archive drawings were obtained, although these did not show full construction details and attempts to locate any further information were not successful.

Figure 13 provides a schematic section through the building looking north, providing an overview of the different foundation systems.

It was recognised at an early stage of the detailed design that the specific arrangement of construction in this building was particularly complex in terms of its likely response to ground movements; a detailed assessment was therefore carried out.

The new floors comprise precast concrete units with an *in situ* topping. The units generally span across the width of the wings onto edge beams which, in turn, are supported on perimeter steel columns. The columns are tied together by transverse steel beams. The edge beams are connected to the party walls on the north and south sides. While the details on the archive drawings indicated that some provision for differential settlement between the buildings was intended, this would have been limited in magnitude and intrusive investigations showed that the tubes in which the threaded studs are located have been concreted up.

The implications of differential movement between 4–6 Soho Square and the adjacent buildings were therefore considered on the assumption that there was little, if any, provision to accommodate such movement.

The foundations of the original buildings are corbelled brick or stone strip and pad footings, although some walls were underpinned during construction of the 1980s link blocks. Foundations to the link blocks are piled, with the steel frame taken through the vaulted substructure. Trial pit information from the refurbishment shows strip footings under the original brick walls/vaults are typically founded at approx. 500mm below basement level, with various local deepenings at the original timber column positions to approx. 1500mm below basement level. The latter approximately matches the founding level of the pad

"THIS BUILDING WAS PARTICULARLY COMPLEX IN TERMS OF ITS LIKELY RESPONSE TO GROUND MOVEMENTS"

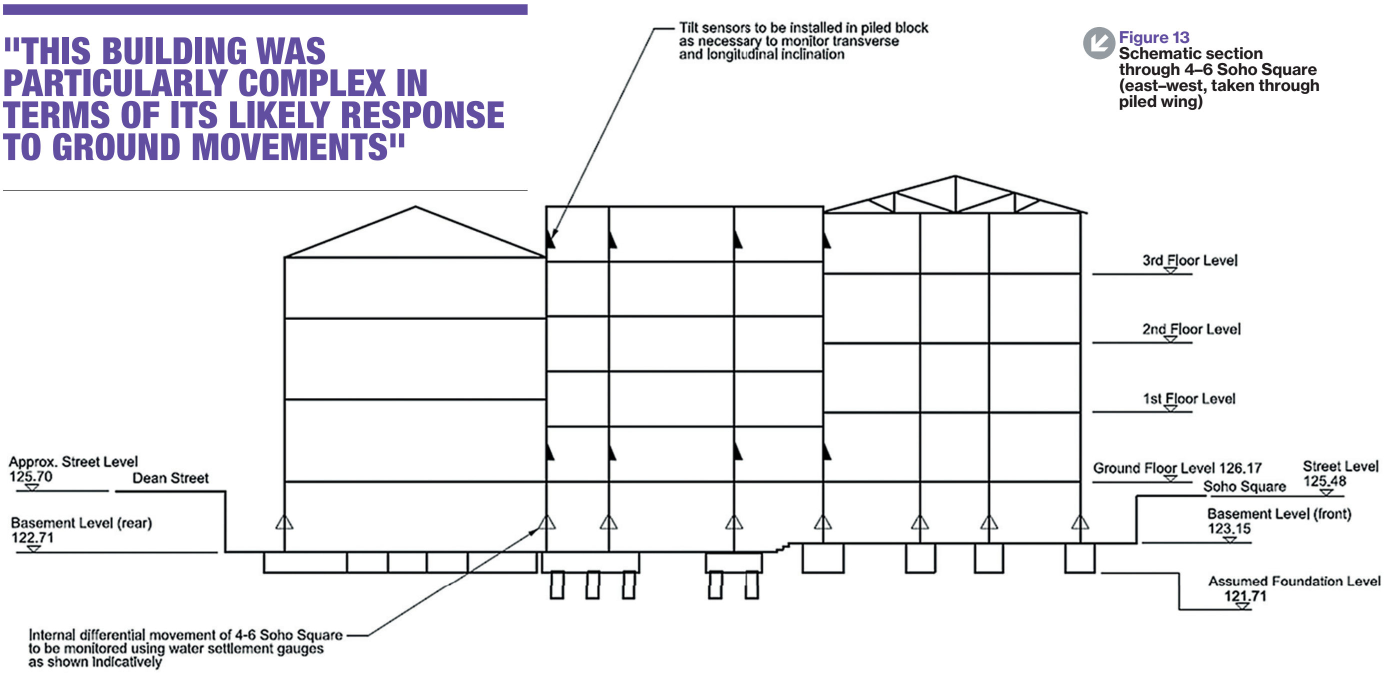


Figure 13
Schematic section through 4–6 Soho Square (east–west, taken through piled wing)

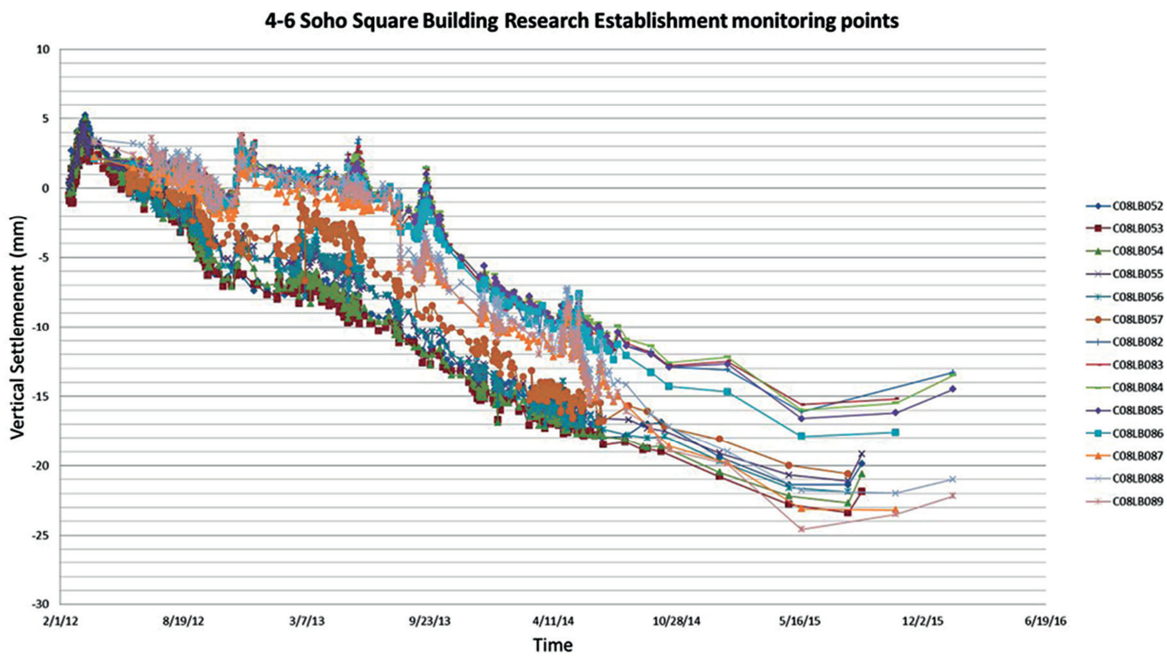


Figure 14 Time–settlement plot for 4–6 Soho Square

Figure 15 Heritage features: plasterwork in House of St Barnabas

footings under internal timber columns. Pile caps and ground beams for the new foundations are set just below the basement slab. Record drawings indicate that the walls adjacent to the new foundations were underpinned to a level approx. 1500mm to 2000mm below basement level, to enable the construction of the pile caps and ground beams. Crossrail archive information shows piles under the link blocks extend 22m below ground-floor level, although there is no record of pile diameters.

While it was accepted that the risk of some minor cracking to 4–6 Soho Square could not be eliminated, neither significant cracking of the walls nor of floor slabs and finishes were acceptable impacts in terms of damage to the structure and to its heritage value. Hence, it was concluded that mitigation measures were required, with any minor defects being repaired once movements had ceased.

The Phase 3 assessment of 4–6 Soho Square assigned a total score of 3 to the building, looking at the most onerous construction stages for critical sections through the building. This resulted in the adoption of compensation grouting as a protective measure to reduce the magnitude of ground movements, generally allowing a reduced damage category of 1 to be assigned to the building. This is consistent with possible crack widths of up to 1mm. The grouting was needed to address both differential movements between the piled and non-piled elements within the building, and also those between the piled elements and the party walls, recognising that the latter are also part of the adjoining buildings and thus subject to separate control. Grout shafts around Soho Square are indicated in Fig. 8, together with the TAM arrays from each. As part of the mitigation, compensation grouting could be used to control the movements of those parts of the structure on ground-bearing foundations, as was adopted commonly for buildings around the stations.

Although grouting was not used to control pile movements, in order to maximise coverage it was proposed, unusually, that the grout TAMs would be 'threaded' through the piled areas, requiring more accurate TAM positioning but offering greater control of ground movements. This also allowed grouting below the piled areas should this be necessary; again, while this was not a measure commonly utilised, it was decided after careful review that this would be instigated if differential settlements between these and the ground-bearing areas exceeded the 'trigger' level of 5mm specified in the Specification for Control of Ground Movements[17]. The same trigger level was specified for differential movements between the piled areas and the adjoining buildings, namely 3 and 7 Soho Square and 5 Dean Street. In the event, these triggers were not reached and no compensation grouting was required in the piled areas.

Settlement of the foundations to the party walls and the areas of slab adjacent was controlled by the grouting arrays below the neighbouring buildings. It was therefore essential to control the grouting process for all these buildings as a single unit and instrumentation was arranged accordingly. It was recognised there was still some risk that, in the basement areas below the piled wings, it might not be possible to mitigate the movements fully by compensation grouting; it was therefore accepted that some cracking could occur.

Compensation grouting was also used to control the differential settlements along the party wall lines to avoid damage to the connections between the party walls and the 1980s framed structures. In the absence of effective allowance in the construction details for differential movement, it was necessary to ensure that there was very specific control

Figure 16 Heritage features: stone staircase in 26 Soho Square (protection structure seen below)

"A PROTECTION STRUCTURE WAS INSTALLED BELOW THE STAIRS"

of the compensation grouting in order to minimise the impact of the predicted ground movements on the various elements of the building and its neighbours.

The following additional measures were undertaken to minimise ground movements at source as far as practicable, using controls on construction:

- A volume-loss control zone, where a lower volume-loss target is set, was introduced for the eastbound running tunnel. During construction, the TBM was driven with tighter face-pressure control when driving through the volume loss control zone.
- Volume loss was minimised during construction of the three adjacent station tunnels that affect settlement of the buildings. This was achieved by sequential excavation of the tunnel sections so limiting the size and the length of unsupported ground at the excavation face.
- Ground movements resulting from construction of the Western Ticket Hall were carefully controlled by minimising the deflection of the embedded wall.

Specific instrumentation was installed on the outside and inside of 4–6 Soho Square and the adjoining buildings to allow monitoring to be undertaken as part of the construction contract. This was for control of the compensation grouting process and movements of the building. This instrumentation comprised hydraulic levelling cells in the basement with prisms, Building Research Establishment (BRE) studs and Invar scales on the external facades. Tiltmeters were also installed on internal columns. Figure 14 shows the time–settlement plot for the BRE studs at ground-floor level. This provides some indication of the monitoring data obtained during the project; such plots enabled the impact of particular construction activities to be assessed.

During the course of the works, no more than hairline cracking (consistent with damage category <1) was identified. Given the works (structural and non-structural) being undertaken in the building concurrently with the Crossrail works, it was not possible to be definitive as to causation. It may be concluded that the mitigation measures adopted, namely compensation grouting and specific construction controls, prevented unacceptable levels of cracking being experienced during the course of the works. Monitoring also suggested that that there was no noticeable difference in response to ground movements between the piled and non-piled areas of the building.

While this was an unusually complex structure, the process described provides an overview of the extent of assessment and monitoring required to comply with Crossrail Ltd's obligations to safeguard the assets along the alignment.

The House of St Barnabas (1 Greek Street)

The House of St Barnabas is a Grade 1 listed building. Internally it has some very fine plasterwork (Figure 15) and it is acknowledged as a fine example of a Georgian interior; Pevsner[18] describes it as 'one of the best and best-preserved mid-C18 houses in London'. Located at the east end of Tottenham Court Road station, it was afforded special protection, with the appointment of separate heritage specialists, extensive pre-works condition surveys and monitoring installations both internally and externally. Pre-works mitigation included some repairs to brickwork and plaster and the installation of a protection frame below the fine open-well cantilever stair staircase.

26 Soho Square

Immediately to the north of the House of St Barnabas, the building at 26 Soho Square is Grade 2* listed. It also contains decorative plasterwork and another fine staircase (Figure 16). A protection structure was installed below the stairs until such time as ground movements were shown from the monitoring to have diminished to the specified level (<2mm per year).

Both here and in the House of St Barnabas, there was clearance between the protection structure and the staircase throughout, and no further works were required.

88 Dean Street

The building at 88 Dean Street, also Grade 2* listed, showed signs of past movement both externally and internally. Of most concern was the pronounced outward lean on the front elevation, visible from the street and confirmed by accurate survey. While investigations confirmed that the facade was restrained by the internal floors, the cause of the movement remained unconfirmed and there was some concern as to potential stability even with the very small ground movements predicted. Accordingly, it was decided that a scaffold structure would be erected externally, separated from the facade but designed to hold it in the event of major movements.

The facade was monitored and periodic inspections were carried out. In addition, some repairs were carried out while the scaffold was in place to improve the integrity of the facade.

Conclusions and lessons learned

The damage assessment process carried out followed that set out in Crossrail Information Paper D12[1], which was established at the start of the works, and was itself derived from other major tunnelling contracts undertaken in the late 20th century. The results may be deduced from the outcome, namely that there were no incidents which required urgent intervention during the course of the works. Post-works repairs were always anticipated, but it was possible to carry these out in a planned manner.

A number of useful lessons have been learned from what was done and it is important that these are considered for future works.

While overall extensive monitoring was carried out along the alignment, for any given building this was, in most cases, not sufficient to understand its behaviour and response to ground movement in detail. In the majority of cases, this is not an issue, but for a specific building where there are particularly delicate and/or valuable finishes (e.g. The House of St Barnabas), or where the overall condition pre-works is a concern, a more tailored approach is likely to be needed.

The damage category does not identify specific crack locations, but rather the likelihood of cracks up to a given width occurring. Likely locations could be predicted, but other cracks were found to occur, particularly where there were previously unidentified defects/weaknesses. Overall, damage levels were low.

There is some potential ambiguity as to whether the predicted crack will be in the finishes and/or in the structure. Originally, the methodology was for masonry buildings on shallow foundations: further review is needed for modern framed buildings on piles. Additionally, assessment of the behaviour of buildings where a facade on shallow foundations has been retained in a new development comprising a framed structure on deep piled foundations should be undertaken.

Further research work is being undertaken to look at the effects of building geometry and the extent of facade openings. The results of academic research should be evaluated for further projects requiring widescale asset assessment. Neither the visual inspections nor the condition surveys provided a detailed record of the building condition. In order to do this, the initial risk assessment has to identify where this should be prioritised. In addition, there are more sophisticated methods of recording condition using photography for subsequent comparison; while this would not be required in all cases, the initial screening should be used to determine where this is best used.

Development of more sophisticated monitoring is continuing. As for assessment methodology, alternative strategies may be used in the future.

The three-phase assessment system provided a good basis for asset protection. With any large infrastructure project, the number of assets implicated will be very significant and improvements in the methodology which may be more effective without increasing the risk should be kept under review.

The extensive use of compensation grouting around the new stations has effectively mitigated the impacts of ground movements. The effects of TAM installation may not always be negligible and selection of the appropriate method needs to be kept under review.

While the assessment process has proved effective over a wide range of structures, with different ages, in varying states of repair and with varied finishes, it does not remove the overriding requirement to use engineering judgement and proceed accordingly at all times.

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