Stage 1

Ground movements behind the retaining wall should be estimated as described in Section 2.5.2 assuming greenfield conditions, ie ignoring the presence of the building or utility and the ground above foundation level. Contours of ground surface movements should be drawn and a zone of influence established based on specified settlement and distortion criteria. All structures and utilities within the zone of influence should be identified.

Stage 2

A condition survey should be carried out on all structures and utilities within the zone of influence before starting work on site. The structure or utility should be assumed to follow the ground (ie it has negligible stiffness), so the distortions and consequently the strains in the structure or utility can be calculated. The method of damage assessment should adopt the limiting tensile strain approach as described by Burland *et al* (1977), Boscardin and Cording (1989) and Burland (2001); see Table 2.5 and Figure 2.18.

Table 2.5 *Classification of visible damage to walls (after Burland* et al, *1977, Boscardin and Cording, 1989; and Burland, 2001)*

Category of damage	Description of typical damage (ease of repair is underlined)	Approximate crack width (mm)	Limiting tensile strain ε_{lim} (per cent)
0 Negligible	Hairline cracks of less than about 0.1 mm are classed as negligible.	< 0.1	0.0–0.05
1 Very slight	Fine cracks that can easily be treated during normal decoration. Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	< 1	0.05–0.075
2 Slight	Cracks easily filled. Redecoration probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weathertightness. Doors and windows may stick slightly.	< 5	0.075–0.15
3 Moderate	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. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired.	5–15 or a number of cracks > 3	0.15–0.3
4 Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15–25 but also depends on number of cracks	> 0.3
5 Very severe	This requires a major repair involving partial or complete rebuilding. Beams lose bearings, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	usually > 25 but depends on number of cracks.	

Notes

1. In assessing the degree of damage, account must be taken of its location in the building or structure.
2. Crack width is only one aspect of damage and should not be used on its own as a direct measure of it.

Date Nov 2015	Job no. 1706/02	Sheet 25
Engineer FGo		
Checked by		
Project ELLERDALE ROAD		

THE ABSOLUTE DEFLECTION IS NOT IMPORTANT.
THE DIFFERENTIAL SETTLEMENT IS, AS THIS IS WHAT CAUSES CRACKING

STRAINS CAN BE RELATED TO Δ: GREATEST DIFFERENTIAL SETTLEMENT

THE GRAPH OVERLEAF SHOWS THE TOTAL VERTICAL DEFLECTIONS CALCULATED
THE BUILDINGS HAVE BEEN DRAWN TO DETERMINE THEIR DIFFERENTIAL VERTICAL MOVEMENT. THEIR EXTENTS - CLOSEST AND FURTHEST POSITIONS FROM THE EXCAVATION HAVE BEEN DETERMINED FROM THE CONTOR MAP

RESULTS

STRUCTURE	CLOSEST CONTOR (m)	δh_1 (MM)	FURTHEST CONTOR (M)	δh_2 (MM)	$\delta h_1 - \delta h_2$ (MM)	L (M)	Δ (MM)	ε_h %	Δ / L %
a/	1	12.3	6	8.6	3.7	5	0.8	0.074	0.016
b/	0	16.8	4	13.7	3.1	4	0.4	0.078	0.01
c/	5	9.1	18	2.0	7.1	13	0.4	0.055	0.003
d/	15	7.0	30	0.8	6.2	15	1.1	0.041	0.007
e/	17	5.8	30	0.8	5.0	13	0.8	0.038	0.006
f/	20	4.5	35	0	4.5	15	0.9	0.030	0.006
g/	25	2.6	50	0	2.6	20	0.6	0.013	0.003

Date NOV 2015
Engineer FG
Checked by Ske
Job no. 1706/02
Sheet 27
Project ELLERDALE ROAD

DAMAGE ASSESSMENT

THE HORIZONTAL AND VERTICAL STRAIN WILL BE USED TO ASSESS THE BURLAND DAMAGE CATEGORY FOR EACH STRUCTURE IN ACCORDANCE WITH LIMITING STRAINS FOR EACH CATEGORY OUTLINED IN TABLE 2.5 (CIRIA C580) AND THE GRAPH IN FIGURE 2.18 (CIRIA C580)

LIMIT STRAIN TO 0.05% (BURLAND CAT 0 = NEGLIGIBLE)

STRUCTURE

d/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.007}{0.05} = 0.14$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.041}{0.05} = 0.82$$

$L/H = 1.3$ FROM FIGURE 2.18 → OKAY

e/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.006}{0.05} = 0.12$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.038}{0.05} = 0.76$$

$L/H = 1.3$ FROM FIGURE 2.18 → OKAY

f/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.006}{0.05} = 0.12$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.03}{0.05} = 0.6$$

$L/H = 1.3$ FROM FIGURE 2.18 → OKAY

g/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.003}{0.05} = 0.06$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.013}{0.05} = 0.26$$

$L/H = 1.25$ FROM FIGURE 2.18 → OKAY

LIMIT STRAIN TO 0.075% (BURLAND CAT 1 = VERY SLIGHT)

STRUCTURE

c/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.003}{0.075} = 0.04$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.055}{0.075} = 0.73$$

$L/H = 1.1$ FROM FIGURE 2.18 → OKAY

LIMIT STRAIN TO 0.15% (BURLAND CAT 2 = SLIGHT)

STRUCTURE

a/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.016}{0.15} = 0.11$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.074}{0.15} = 0.49$$

$L/H = 1.7$ FROM FIGURE 2.18 → OKAY

b/

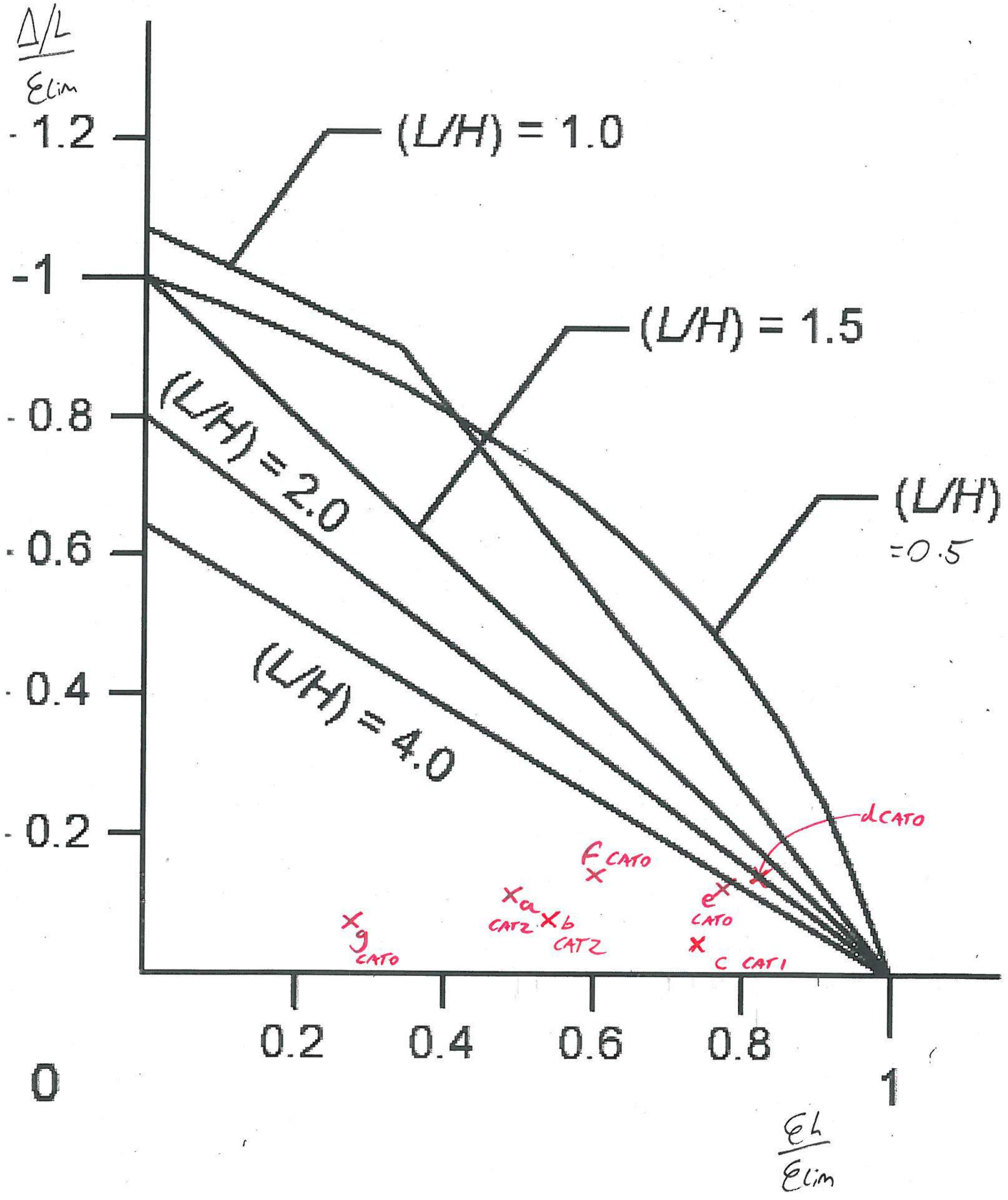
$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.01}{0.15} = 0.07$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.078}{0.15} = 0.52$$

$L/H = 2.7$ FROM FIGURE 2.18 → OKAY.

SUMMARY

STRUCTURE	BURLAND CAT
a/	2
b/	2
c/	1
d/	0
e/	0
f/	0
g/	0


Δ/L
εlim
1.2
1
0.8
0.6
0.4
0.2
0
0.2
0.4
0.6
0.8
1
εh
εlim
(L/H) = 1.0
(L/H) = 1.5
(L/H) = 2.0
(L/H) = 4.0
(L/H)
=0.5
d CAT0
f CAT0
a CAT2
b CAT2
e CAT0
c CAT1
g CAT0

Appendix M – Ground Engineering technical paper

TECHNICAL PAPER

Prediction of party wall movements using Ciria report C580

Richard Ball *and* **Nick Langdon**, *CGL Card Geotechnics, and* **Mark Creighton**, *Galliford Try Construction*

Summary

This paper presents a case study regarding the prediction of party wall movements using Ciria Report C580 – *Embedded retaining walls, guidance for economic design* for the cased small diameter continuous flight auger (CFA) piles that formed the basement box for a site in Pont Street, London.

Installation movements were determined based on Ciria C580 case study data for pile lengths which varied around the perimeter of the basement from 10m to 22m. Extensive monitoring was installed and monitored continuously during construction through 2011 and into 2012. The results are compared to original predictions and it is concluded that installation movement predictions from Ciria guidance can be significantly reduced for controlled contiguous piled wall installations with consequent benefits to party wall negotiations.

Introduction

Many of the basements currently being constructed in London are directly adjacent to neighbouring properties with basement walls generally formed by piling or by underpinning party wall structures. Consequently they are subject to stringent party wall negotiations, and the calculation of ground movements and consequent prediction of building damage is critical.

This paper describes the construction of a piled two-storey basement in Pont Street, London, and provides a rationale for reducing predicted piled wall installation movements at the analysis stage. Monitoring data is provided demonstrating the veracity of this approach, and back analysis has been undertaken to better understand the relationship between predicted and actual ground movements.

Where basements are constructed using piled perimeter walls, a proportion of ground movement will occur due to the installation of the piles, through ground loss and elastic closure of the pile bore. Current best practice is based on Ciria report C580, which presents a historical data set for vertical and horizontal ground movements caused by piled wall installation, predominantly obtained from the London area.

Ciria C580 recommends an upper bound line for predicting ground movements of 0.04% of the wall depth. In the case of the Pont Street basement, this value gave rise to a prediction of unacceptable ground movements and building damage and this prompted a review.

This case study shows a value of 0.02% to be appropriate for a well-constructed piled wall in typical London ground conditions and presents monitoring data obtained throughout

FIGURE 1

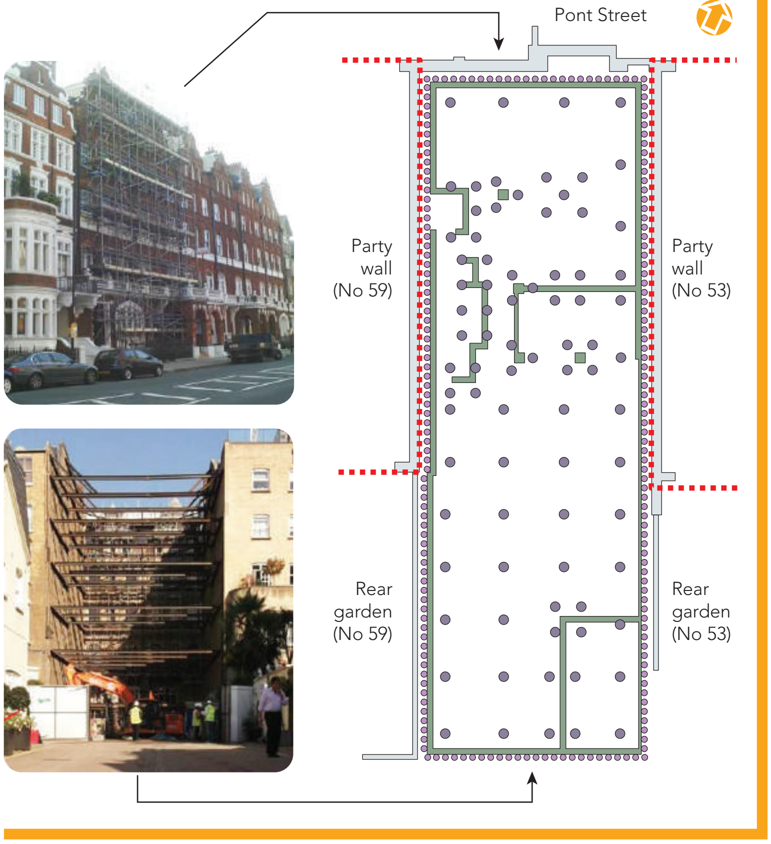


Figure 1 Site views and plan layout

FIGURE 2

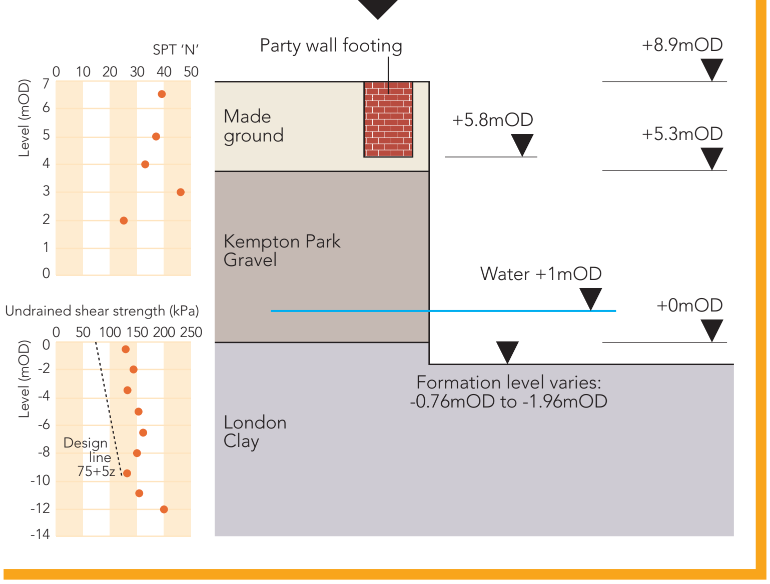


Figure 2 Conceptual site model

Figure 3 Propping scheme (1st level) looking towards Pont Street

the development to support this conclusion. It is recognised that good construction control was critical in realising this level of movement.

Site description

The site fronts onto Pont Street with Clabon Mews to the south and rear. The site was occupied by two mid-terrace Victorian properties five storeys high with a single basement level and mansard roof (see Figure 1). Party wall properties were of a similar style. The site is long and thin, extending some 40m back from its frontage on Pont Street, with an overall width of 12m.

Ground conditions

The ground conditions on site are summarised in Figure 2. They comprised made ground over dense River Terrace Gravels to a depth of 7.5m below the existing ground level, with the London Clay beneath this. Groundwater was recorded at the base of the River Terrace Gravels.

TABLE 1: GEOTECHNICAL DESIGN PARAMETERS

Stratum	Design Level (mbgl) [mOD]	gb (kN/m³)	Cu (kPa) [c']	f'	Eu (MPa) [E']
Made Ground	0 [7.5]	18	n/a	28	5 + 4z[a]
Kempton Park Gravel	1.7 [5.8]	19	n/a	37	40 + 5z[a]
London Clay	7.5 [0.0]	20	75 + 5z[a] [5]	24	50 + 5z[b] [40 + 4z][b]

a. z = depth below surface of stratum
b. Based on Burland J B, Standing J R, and Jardine F M (eds), Building response to tunnelling, csae studies from construction of the Jubilee Line Extension London, Ciria Special Publication 200

Access for the site investigation was severely restricted and intrusive works were limited to a single cable percussion borehole to 20m depth and a series of hand-excavated trial pits. The site investigation data was augmented by CGL Card Geotechnics with historical borehole records, both publicly available and from within CGL's private archive.

In addition, local case studies were consulted for guidance to establish the soil profile and requisite geotechnical design parameters.

Geotechnical design parameters are summarised in Table 1. Parameters for the made ground and gravels were derived from standard penetration testing (SPT) within the borehole, correlating uncorrected "N" values to friction angle based on the relation proposed by Peck et al (1967). The undrained shear strength of the London Clay was established by quick undrained triaxial testing on undisturbed U100 samples and a conservative design line was chosen to fit within the bounds of those published by Patel, 1992. Drained, effective stress parameters were selected with reference to (Burland et al, 2001). Plots of SPT N vs level and undrained shear strength vs level are presented in Figure 2.

The scheme

The scheme comprised the demolition of the existing properties while retaining the façade, with the construction of a single new building and double storey basement across the entire site footprint (Figure 3). The basement was excavated to a depth of between 9.7m to 10.9m below existing ground level (Figure 2) within a contiguous piled wall consisting of 300mm diameter

Figure 4 Piling rig at work within basement

bored piles. The wall was propped at capping beam level and at approximate mid-height during construction using hydraulic props pre-loaded to 200kN to 250kN into position to prevent "relaxation" on excavation.

Working room was severely restricted within the basement, (see Figure 4) and piling was undertaken with a 3.6t Klemm 701 rotary rig using a cased segmental flight auger (SFA) system. The SFA system allows restricted headroom working, with 1m long auger segments sequentially added to the auger string to achieve the required pile depth.

The piles were fully cased through the River Terrace Gravels into the top of the London Clay, limiting the potential for ground loss during boring and the casings were rotated into position to reduce ground vibrations. In addition, piles were installed on a "hit one miss three" basis, such that horizontal stress relief/ground movement never occurred concurrently on two adjacent piles.

Negotiations with engineers responsible for safeguarding the party wall (neighbouring) properties were complex given the value and proximity of the neighbouring properties and the number of party wall stakeholders. In total, 19 party wall awards were made and these required predicted building damage arising from ground movements to fall within "Building Damage Category 1" or "very slight" damage in accordance with the classification scheme proposed by Burland (1974), and later modified by Boscardin and Cording (1989). In addition, an extensive monitoring system was required to be included in the construction process with regular monitoring against agreed movement trigger limits.

Predicted ground movements

Ground movements due to installation were predicted based on the proposed pile lengths, which varied around the

FIGURE 5

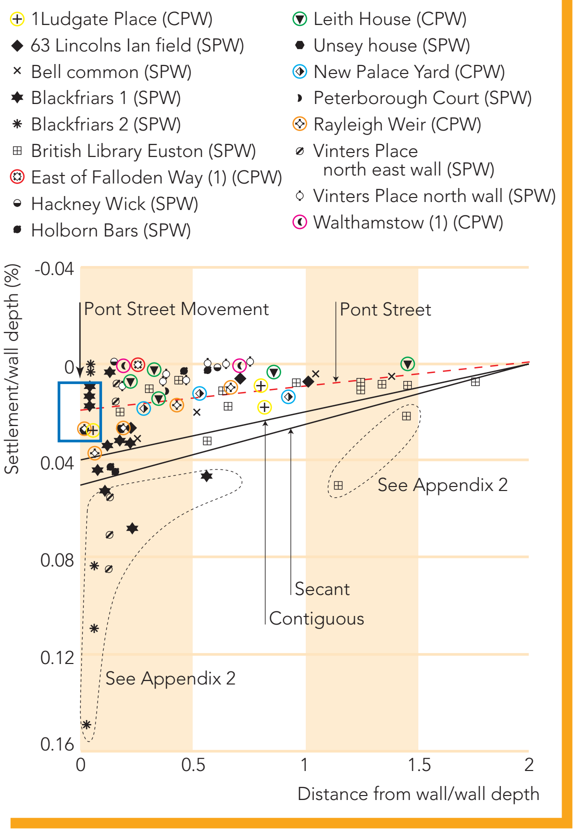
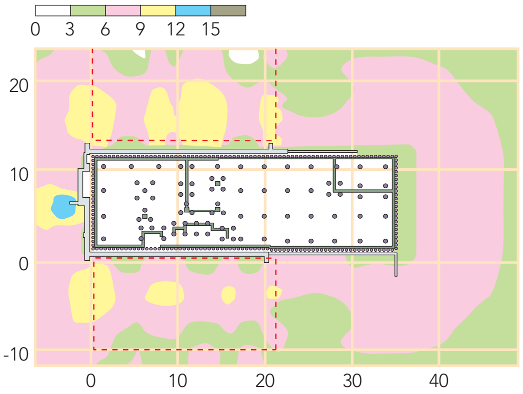


Figure 5 Case study data CIRIA C580 – with authors' rationalisation

FIGURE 6

Total predicted vertical movement



Total predicted lateral movement

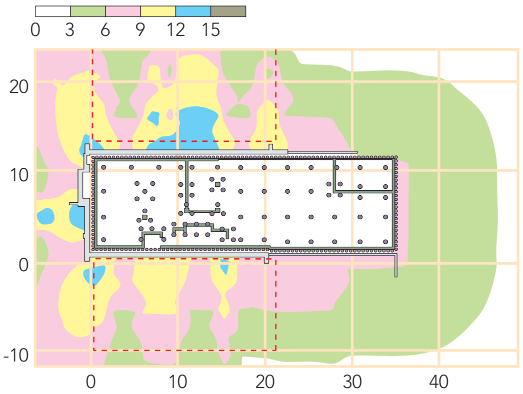
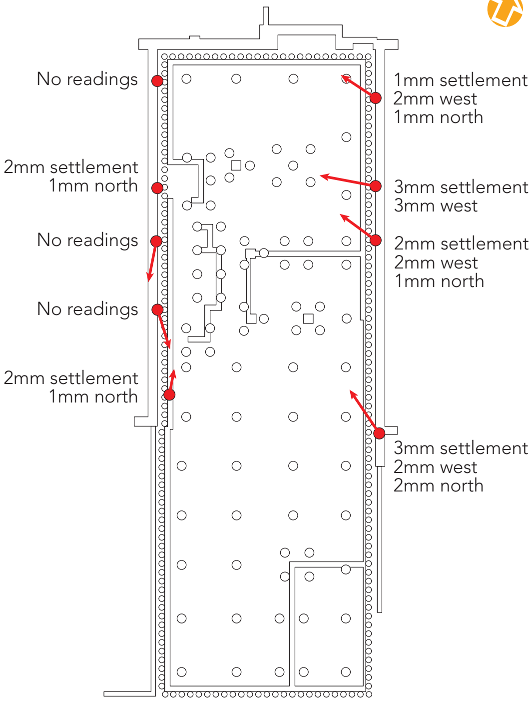


Figure 6 Predicted ground movements (total)

FIGURE 7



Displacements on party wall brickwork (ground floor level) as of 9.2.12. Pile installation completed

Predicted installation movement:
Lateral: 2mm to 4mm
Vertical: 2mm to 4mm

Figure 7 Monitored movements due to pile installation

perimeter of the basement from a general length of 10m, to up to a maximum of 22m where superstructure column loads were picked up on sections of basement wall. These pile lengths resulted in predicted ground movements (based on an installation movement of 0.04% pile length) of between 4mm and 9mm, giving rise to predicted Building Damage Categories in excess of the requisite "Category 1" criterion. This led to a requirement to rationalise and control movements derived from the installation of the piled wall.

Ground movement due to pile installation is thought to occur through ground loss during boring in granular deposits, with a potential additional component of movement derived from vibrational compaction. In clay soils, such as the London Clay, additional movement can potentially be developed by the bores caused by a relaxation of horizontal stresses (Bowles 1988). Given the depth of the London Clay on site, movements within the gravels were more critical and it was considered that these could be considerably reduced by casing through this stratum and by adopting the hit one miss three methodology described above.

Furthermore, the Ciria case study data was reviewed more critically: Ciria C580 compiles case study data from the installation of contiguous and secant piled walls. A total of 26 case studies are reported, and the ground conditions in each comprise a thickness of superficial deposits (alluvium, Terrace Gravels, made ground, glacial till) overlying the London Clay. The data is not sensitised to the proportion of gravels over the London Clay, and are based rather on "typical" ground conditions. It was considered the Pont Street site was not untypical with regard to the data.

The authors reviewed the case study data published in Ciria C580 and noted that for the installation of contiguous piled walls in ground conditions similar to those at Pont Street, a reasonable argument could be made for halving these movements to 0.02% of the pile length.

The 0.04% design value shown in Ciria C580 was clearly an "upper bound" value for all pile types (see Figure 5). Further to this, the new piles were to be cased through the near surface gravels, which was not a common process during the assembly of the Ciria data from the 1980s and early 1990s.

This argument was adopted for the ground movement analysis, reducing predicted installation movements to between 2mm and 4.5mm, reducing the predicted Building Damage Category to Category 1 and thereby having the potential to satisfy the requirements for party wall agreement.

A stringent monitoring scheme was specified, implemented alongside the controlled piling methodology previously described.

Geotechnical analysis

Predicted installation ground movements were combined with a retaining wall analysis and heave analysis to determine overall predicted ground movements at the location of the party wall foundations. The results are summarised in Figure 6 and were used to determine trigger limits against which monitoring was undertaken. This data is provided for completeness; however, this paper is concerned primarily with ground movements caused by the installation of the piled walls.

Monitoring

The site was monitored comprehensively to an accuracy of +/-2mm with a Leica TS30 Motorised Total Station and Leica DNA03 precision digital level. Eighty retro targets were fixed to the party wall structures and the basement capping beam, monitored weekly by an independent surveying company.

Recorded ground movements

Monitoring results are summarised in Figure 7, from targets installed on the party wall brickwork at approximately ground-floor level. The vectors show movements at the end of basement wall installation and prior to any excavation taking place. Settlement is recorded as being generally between 1mm to 3mm, with lateral movements of a similar order. These values agree well with the predicted installation movements of between 2mm and 4.5mm.

Monitored movements have been normalised by pile length and are summarised in Table 3. Table 3 includes normalised movements accounting for the worst error combination in readings (provided in square brackets), giving the maximum range of normalised movements.

It can be seen from the data presented that normalised installation movements for the site are on average of 0.01% of pile length. Allowing for the worst combination of errors (for example, a systematic error in the monitoring equipment) gives an average of 0.023% pile length. As this is very unlikely, it is considered that the results are generally consistent with the rationalised approach set out by the authors based on 0.02% of wall depth.

Conclusions

This paper provides a case study demonstrating that with good construction control, piled wall installation movements can be restricted to 0.02% of wall length, providing a significant reduction in predicted ground movements over the commonly adopted upper bound limit of 0.04% as published in Ciria C580.

It is noted in Ciria C580 that "the magnitude of ground movements depends upon the quality of workmanship. Large local ground movements can be expected where construction problems are encountered".

In this context the authors would suggest that 0.02% wall length for contiguous wall installation is a reasonable design value where construction controls – such as cased CFA piling and hit and miss construction – are put in place from an early stage with rigorous monitoring methodologies set against rationally derived trigger limits. All parties must be made aware of the potential risks to party wall properties, and all construction activities considered within the background of potential ground movements.

TABLE 2: RECORDED PILE INSTALLATION MOVEMENTS NORMALISED AGAINST PILE LENGTH

	Horiz/pile length (%)	Vert/pile length (%)
Location	[measurement error +/- 2mm]	
A2	0.006 - [0.023]	0.013 - [0.025]
A4	0.006 - [0.021]	0.012 - [0.023]
E1	0.011 - [0.025]	0.005 - [0.015]
E2	0.014 - [0.025]	0.014 - [0.023]
E2.5	0.012 - [0.027]	0.011 - [0.022]
E4	0.012 - [0.018]	0.013 - [0.022]

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