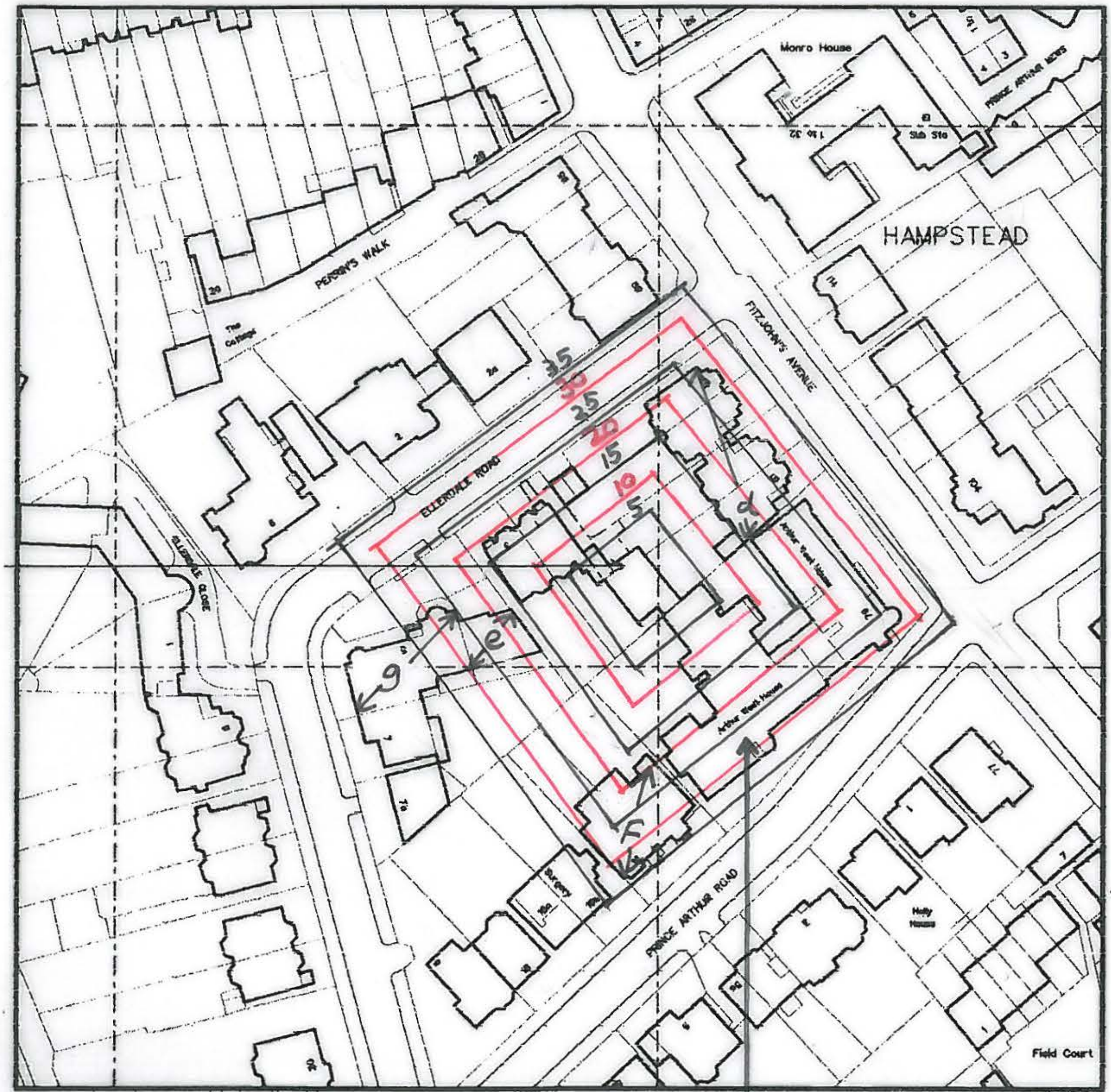
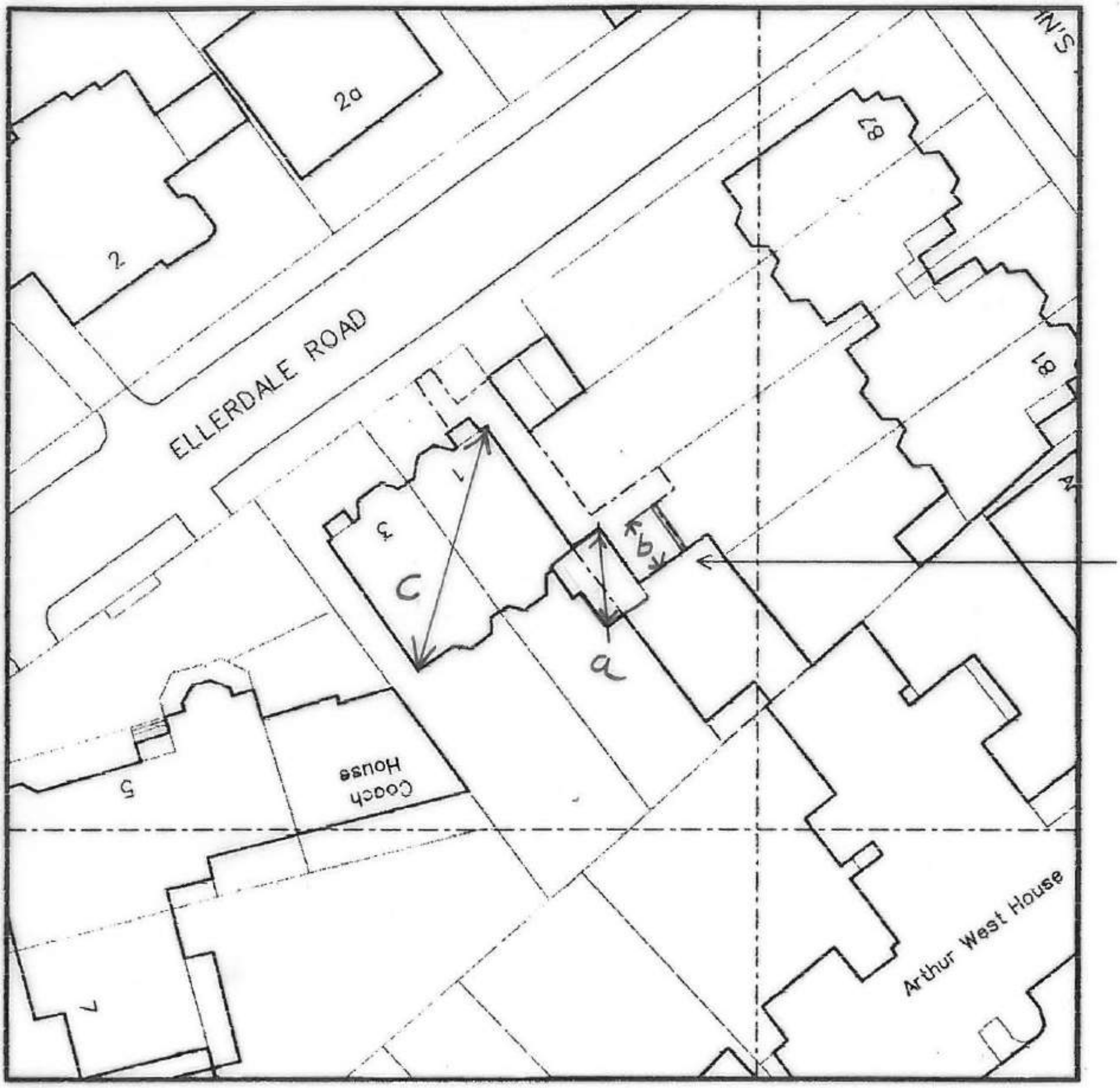


Site Location

SCALE 1:1250

ARTHUR WEST HOUSE
CURRENTLY UNDER
DEMOLITION



Block Plan

SCALE 1:500

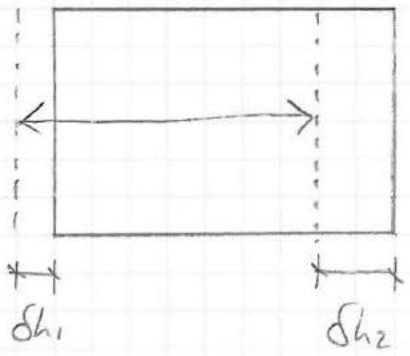
Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date OCT '2015	Job no. 1706/03	Sheet 5
Engineer FLo		
Checked by HB/SBe		
Project ELLERDALE ROAD		

CALCULATIONS OF HORIZONTAL AND VERTICAL STRAINS IN THE BUILDINGS DUE TO THE DEFLECTIONS

HORIZONTAL STRAINS

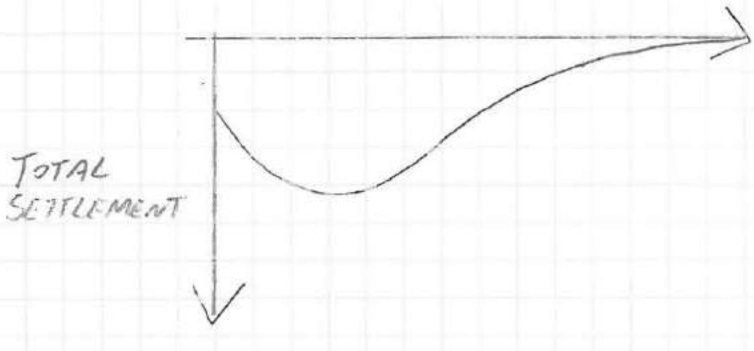


HORIZONTAL STRAIN

$$\varepsilon_h = \frac{\delta h_2 - \delta h_1}{L}$$

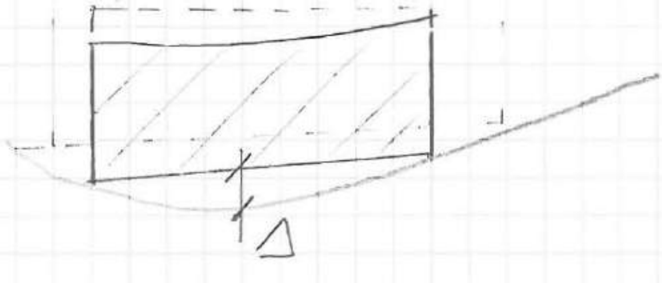
VERTICAL STRAINS

THE TOTAL VERTICAL DEFLECTION FOLLOWS THE FOLLOWING RELATIONSHIP:



VERTICAL STRAINS ARE MORE COMPLICATED TO CALCULATE THAN HORIZONTAL STRAINS.

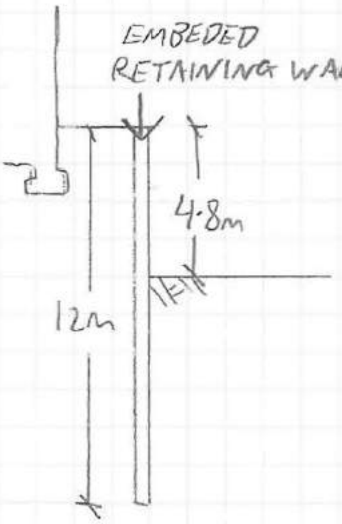
THE BUILDING WILL DEFORM THUS:



Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

CALCULATE PREDICTED GROUND MOVEMENT TO REAR EXTENSION OF 1 ELLERDALE ROAD (CONTOR INTO 6m)

INITIAL DESIGN



PILE DEPTH : 12m (CONSERVATIVE)
PILE TYPE : CONTIGUOUS

EXCAVATION DEPTH : 4.8m
ASSUME LOW SUPPORT STIFFNESS

1/ MOVEMENT DUE TO PILE INSTALLATION

BASED ON FIGURE 2.8 - C580.

DISTANCE FROM WALL FACE (m)	MOVEMENT DUE TO PILE INSTALLATION	
	HORIZONTAL (mm)	VERTICAL (mm)
1	4.4	4.6
6	2.4	3.6

2/ MOVEMENT DUE TO EXCAVATION - ASSUME LOW SUPPORT STIFFNESS

[FIG 2.11] - C580

DISTANCE FROM WALL FACE (m)	MOVEMENT DUE TO EXCAVATION	
	HORIZONTAL (mm)	VERTICAL (mm)
1	18.2	14.4
6	13.2	8.2

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date Nov 2015	Job no. 1706/02	Sheet 7
Engineer FGo		
Checked by SB		
Project ELLERDALE ROAD		

TOTAL MOVEMENT

DISTANCE FROM WALL FACE (m)	TOTAL MOVEMENT HORIZONTAL (MM)	VERTICAL (MM)
1	22.6	19.0
6	15.6	11.8

DAMAGE ASSESSMENT FOR EXTENSION TO 1 ELLERDALE ROAD

THE HORIZONTAL AND VERTICAL STRAIN WILL BE USED TO ASSESS THE BURLAND CATEGORY, IN ACCORDANCE WITH LIMITING STRAINS FOR EACH CATEGORY OUT LINED IN TABLE 2.5 C580 AND FIGURE 2.18

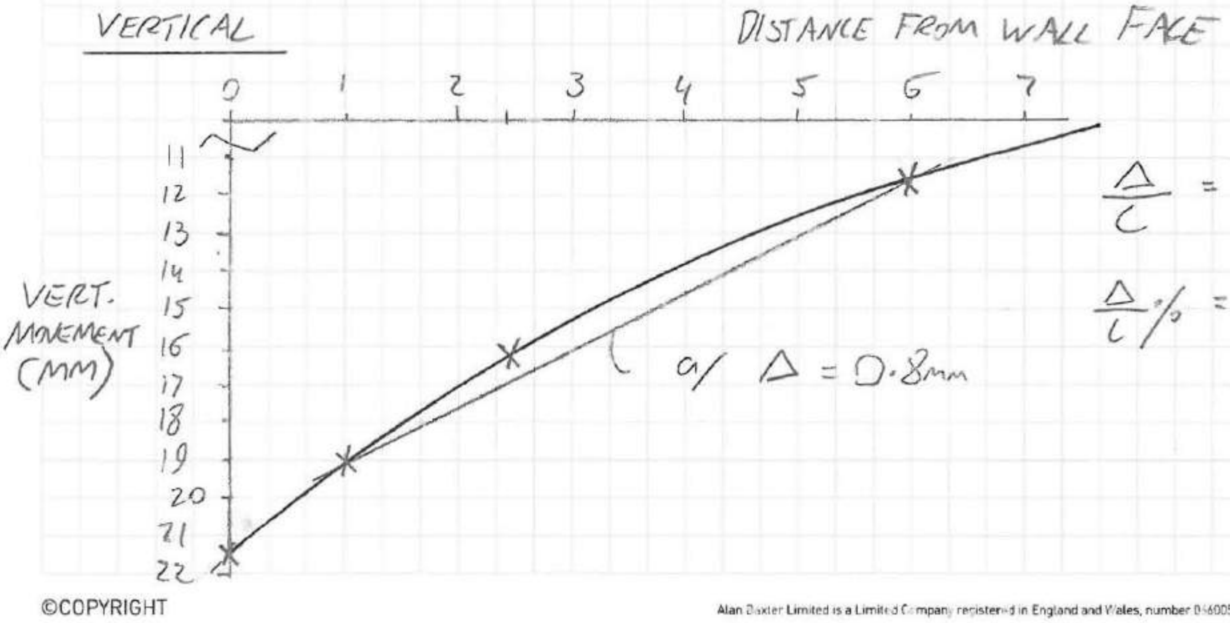
HORIZONTAL

$$\delta h_1 - \delta h_2 = 7$$

$$\varepsilon h = \frac{\delta h_1 - \delta h_2}{L} = \frac{7}{5000}$$

$$\varepsilon h \% = 0.14$$

VERTICAL



$$\frac{\Delta}{L} = \frac{0.8}{5000}$$

$$\frac{\Delta}{L}\% = 0.016$$

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 8
Engineer FKO		
Checked by SBG		
Project ELLERDALE ROAD		

LIMIT STRAIN TO 0.15% BURLAND CAT 2 = SLIGHT

$\varepsilon_{lim} = 0.15$

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

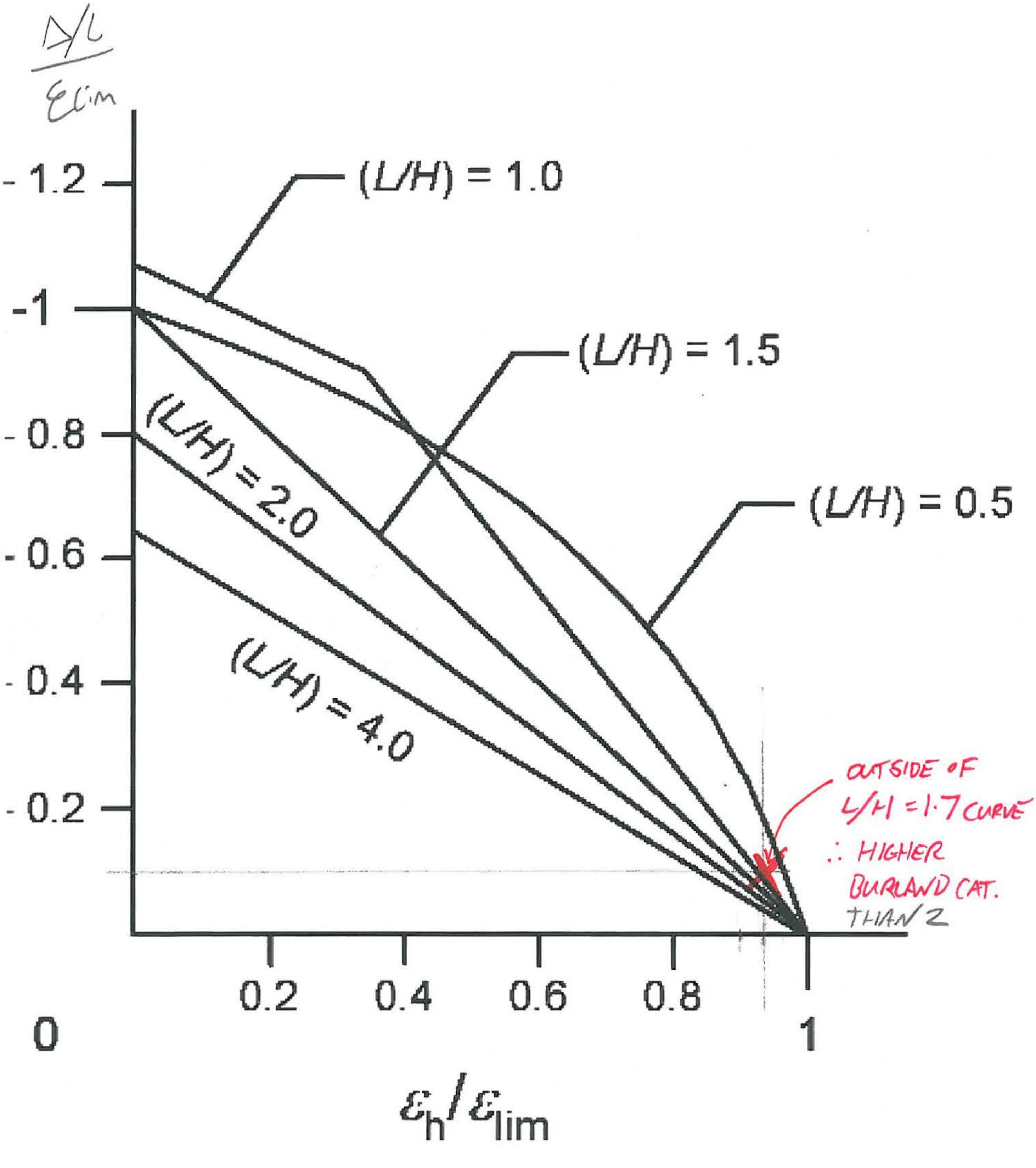
$$\frac{\varepsilon_L}{\varepsilon_{lim}} = \frac{0.14}{0.15} = 0.93$$

L/H : 1.7 FROM FIGURE 2.18 → $\boxed{\text{NOT OKAY}}$

DAMAGE CATEGORY TO EXTENSION EXCEEDS BURLAND CAT. 2

RE RUN ASSESSMENT USING TEMPORARY PROPS TO PROVIDE HIGH SUPPORT STIFNESS

ALSO TAKE INTO ACCOUNT THE RELATIVE DEPTH OF FOUNDATION TO 1 ELLERDALE ROAD AND REAR EXTENSION



Δ/L
εlim
1.2
1
0.8
0.6
0.4
0.2
0
(L/H) = 1.0
(L/H) = 1.5
(L/H) = 0.5
(L/H) = 2.0
(L/H) = 4.0
0.2
0.4
0.6
0.8
1
$\varepsilon_h / \varepsilon_{lim}$
OUTSIDE OF
L/H = 1.7 CURVE
∴ HIGHER
BURLAND CAT.
THAN 2

FIGURE 2.18

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 10
Engineer FGo		
Checked by SBe		
Project ELLERDALE ROAD		

GROUND MOVEMENT ESTIMATE IN ACCORDANCE WITH CIRIA C580 FOR BUILDINGS: b/, d/-g/

ESTIMATE MOVEMENTS USING EMPIRICAL RELATIONSHIPS AS PER C580- EMBEDED RETAINING WALLS

1- MOVEMENT DUE TO PILE INSTALLATION

FINAL DESIGN

BASED ON FIGURE 2.8 - C580

WALL TYPE: CONTIGUOUS PILE WALL
PILE DEPTH ≈ 12m (CONSERVATIVE)

INCLUDE HIGH STIFFNESS PROPPING

MOVEMENTS DUE TO PILE INSTALLATION
BASED ON FIG 2.8 CIRIA C580

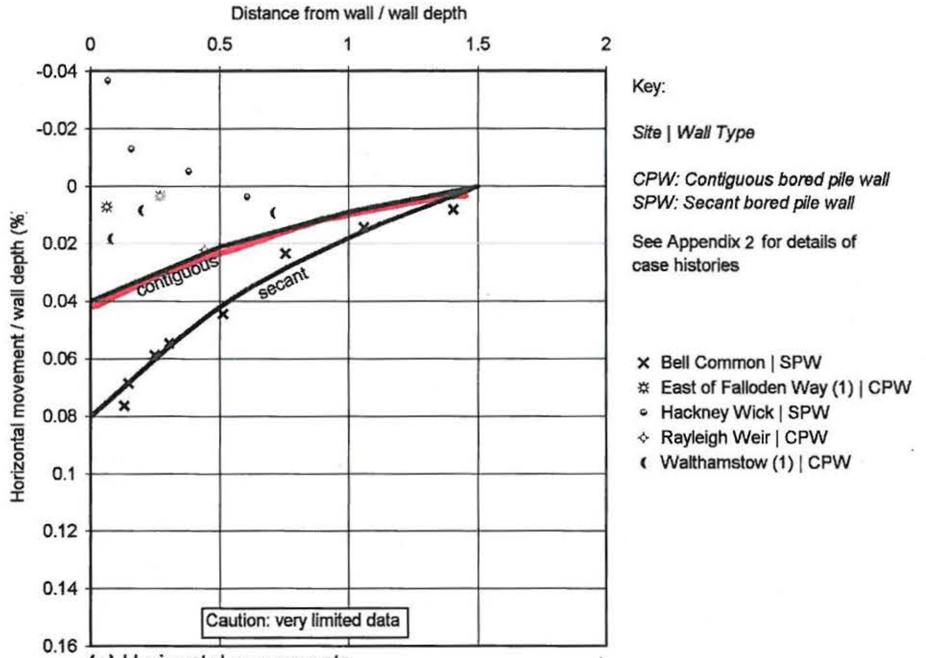
DISTANCE FROM FACE OF WALL	MOVEMENT DUE TO PILE INSTALLATION HORIZONTAL (mm)	VERTICAL (mm)
0	4.8	4.8
1	4.4	4.6
5	2.8	3.8
10	1.6	2.8
15	0.6	1.8
20	0	0.8
25	0	0

DISTANCE BEHIND WALL TO NEGLIGIBLE HORIZONTAL MOVEMENT

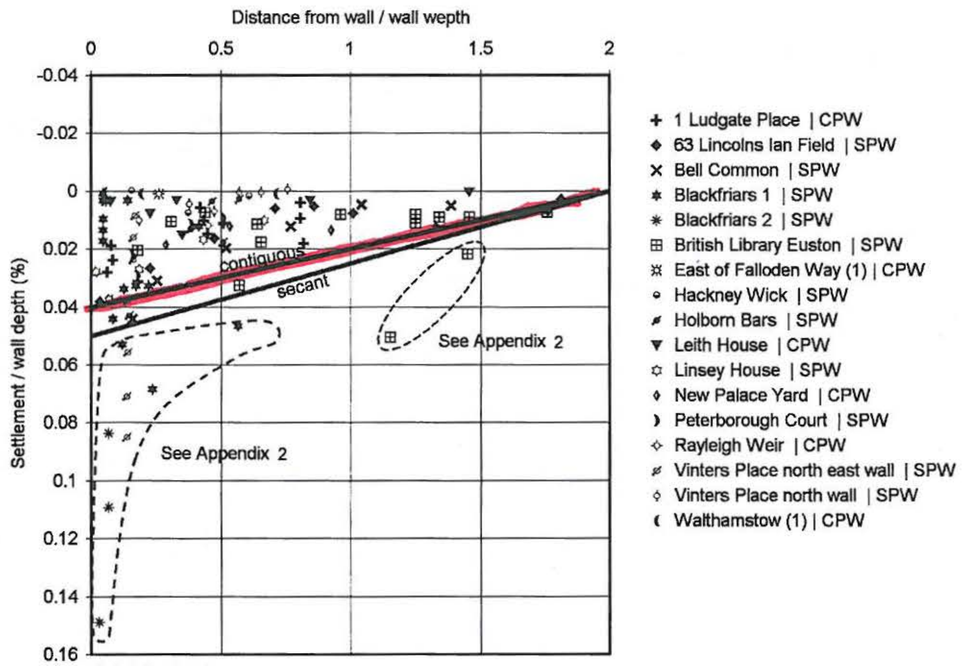
= 1.5d = 1.5 x 12 = 18m [TABLE 2.2]

DISTANCE BEHIND WALL TO NEGLIGIBLE VERTICAL MOVEMENT

= 2.0d = 2.0 x 12 = 24m [TABLE 2.2]



(a) Horizontal movements



(b) Vertical movements

Figure 2.8 *Ground surface movements due to bored pile wall installation in stiff clay*

12

Figures 2.8 and 2.9 show the combined data collated from Clough and O'Rourke (1990), Thompson (1991), Carder (1995) and Carder *et al* (1997) and can be used to estimate ground surface movements arising from the construction of bored pile and diaphragm walls embedded in stiff clays. Table 2.2 summarises the magnitude and extent of the monitored ground movements for walls installed under conditions of good workmanship. The data presented in Figures 2.8 and 2.9 are relatively limited, particularly measurements of horizontal movements for walls. Ground movement estimates based on Figures 2.8 and 2.9 and Table 2.2 should therefore be treated as indicative only. **At locations where such movements are of importance, appropriate instrumentation should be installed and the ground movements monitored accordingly.**

Table 2.2 *Ground surface movements due to bored pile and diaphragm wall installation in stiff clay*

Wall type	Horizontal movements		Vertical movements	
	Surface movement at wall (per cent of wall depth)	**Distance behind wall to negligible movement** (multiple of wall depth)	**Surface movement at wall** (per cent of wall depth)	**Distance behind wall to negligible movement** (multiple of wall depth)
Bored piles				
Contiguous	0.04	1.5	0.04	2
Secant	0.08	1.5	0.05	2
Diaphragm walls				
Planar	0.05	1.5	0.05	1.5
Counterfort	0.1	1.5	0.05	1.5

Notes

1. Maximum surface movement occurs close to the wall and is calculated as a percentage of the pile depth/diaphragm wall trench depth, as appropriate.
2. Extent of movement is calculated non-dimensionally by dividing by the pile depth/diaphragm wall trench depth, as appropriate

Ground movements arising from excavation in front of wall

Ground movements associated with excavations comprise "global" and "local" movements. Global movements are caused by elastic movements in the ground, whereas local movements are concentrated and plastic and arise as the soil approaches its limiting strength. Movements induced by the excavation are made up of the response to the removal of lateral support to the sides of the wall and the response to the removal of the vertical load at the base of the excavation.

Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no.	Sheet
Engineer FGo	1706/02	13
Checked by SBe		
Project ELLERDALE ROAD		

2 - MOVEMENT DUE TO EXCAVATION

ASSUME HIGH SUPPORT STIFFNESS AS TEMPORARY PROPS INSTALLED BEFORE PERMANENT PROPS (SLABS) AT HIGH LEVEL

[TABLE 2.3]

DEPTH OF EXCAVATION = 4.8 m   BASED ON FIGURE 2.11

DISTANCE FROM FACE OF WALL m	MOVEMENT DUE TO EXCAVATION HORIZONTAL (mm)	VERTICAL (mm)
0	7.2	2.0
1	6.8	2.7
5	5.3	3.2
10	3.4	1.7
15	1.6	0.5
20	0	0
25	0	0
30	0	0
35	0	0

DISTANCE BEHIND WALL TO NEGLIGIBLE HORIZONTAL MOVEMENT

$= 4 \times 4.8 = 19.2$ m

DISTANCE BEHIND WALL TO NEGLIGIBLE VERTICAL MOVEMENT

$= 3.5 \times 4.8 = 16.8$ m

14

Table 2.3 *Support stiffness categories (Carder, 1995)*

Support stiffness	Description/examples
High	*Top-down construction*, temporary props installed before permanent props at *high* level
Moderate	Temporary props of high stiffness installed before permanent props at *low* level
Low	*Cantilever walls*, temporary props of *low* stiffness or temporary props installed at low level

Table 2.4 summarises the magnitude and extent of the monitored ground surface movements due to excavation in front of bored pile, diaphragm and sheet pile walls wholly embedded in stiff clay under conditions of good workmanship. The case history data, upon which Table 2.4 is based, relate to excavations that range in depth from 8 m to 31 m, have a factor of safety against base heave in excess of 3 and where walls are wholly embedded in stiff clay.

Table 2.4 *Ground surface movements due to excavation in front of bored pile, diaphragm wall and sheet pile walls wholly embedded in stiff clays*

Movement type	High support stiffness (high propped wall, top-down construction)		Low support stiffness (cantilever or low-stiffness temporary props or temporary props installed at low level)	
	Surface movement at wall (per cent of max excavation depth)	**Distance behind wall to negligible movement** (multiple of max excavation depth)	**Surface movement at wall** (per cent of max excavation depth)	**Distance behind wall to negligible movement** (multiple of max excavation depth)
Horizontal	0.15	4	0.4	4
Vertical	0.1	3.5	0.35	4

Notes

1. Maximum surface movement occurs close to the wall and is expressed as a percentage of maximum excavation depth in front of the wall.
2. Extent of movement is calculated non-dimensionally by dividing by maximum excavation depth.
3. Movements exclude those arising from wall installation effects.
4. Movements correspond to good workmanship and to walls wholly embedded in stiff clays retaining stiff clays or competent soils.
5. Movements will be greater where soft soils are encountered at formation level; see Appendix 2.

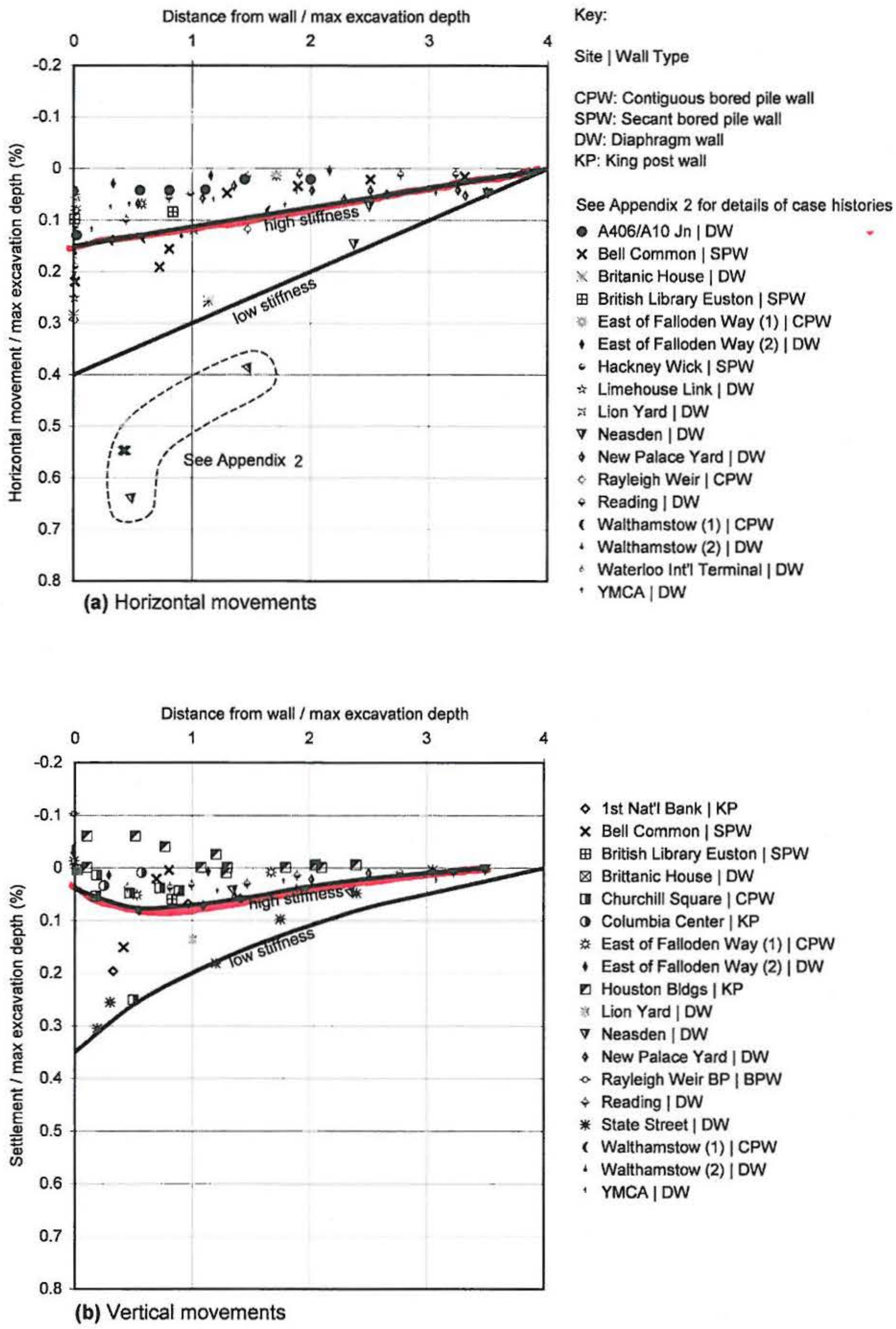


Figure 2.11 *Ground surface movements due to excavation in front of wall in stiff clay*

Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no.	Sheet
Engineer FGo	1706/02	16
Checked by SBa		
Project ELLERDALE ROAD		

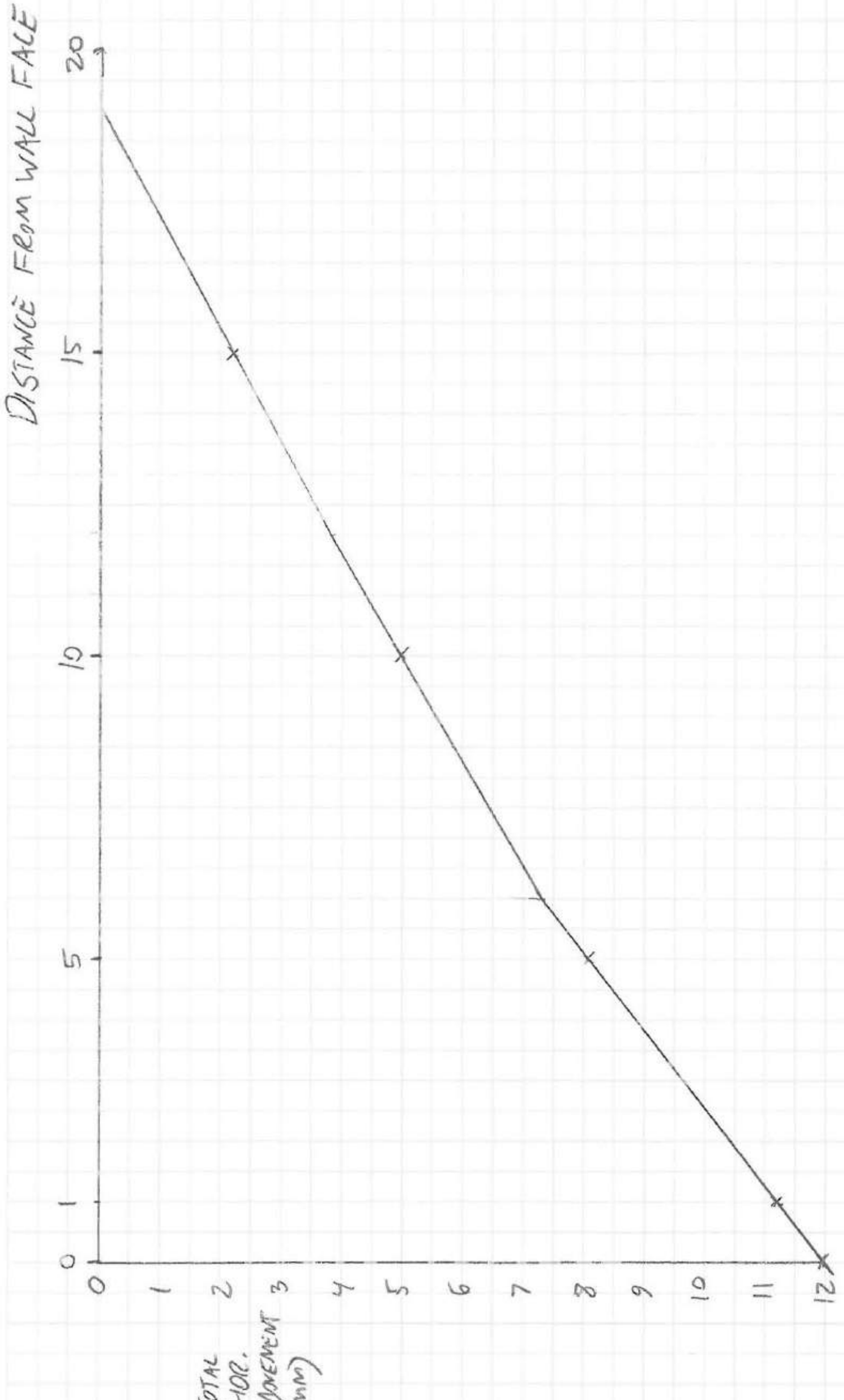
TOTAL GROUND SURFACE MOVEMENTS

ADD MOVEMENTS DUE TO PILE INSTALLATION AND EXCAVATION

DISTANCE FROM WALL FACE (M)	TOTAL MOVEMENT HORIZONTAL (MM)	VERTICAL (MM)
0	12	6.8
1	11.2	7.3
5	8.1	7.0
10	5	4.5
15	2.2	2.3
20	0	0.8
25	0	0
30	0	0
35	0	0

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

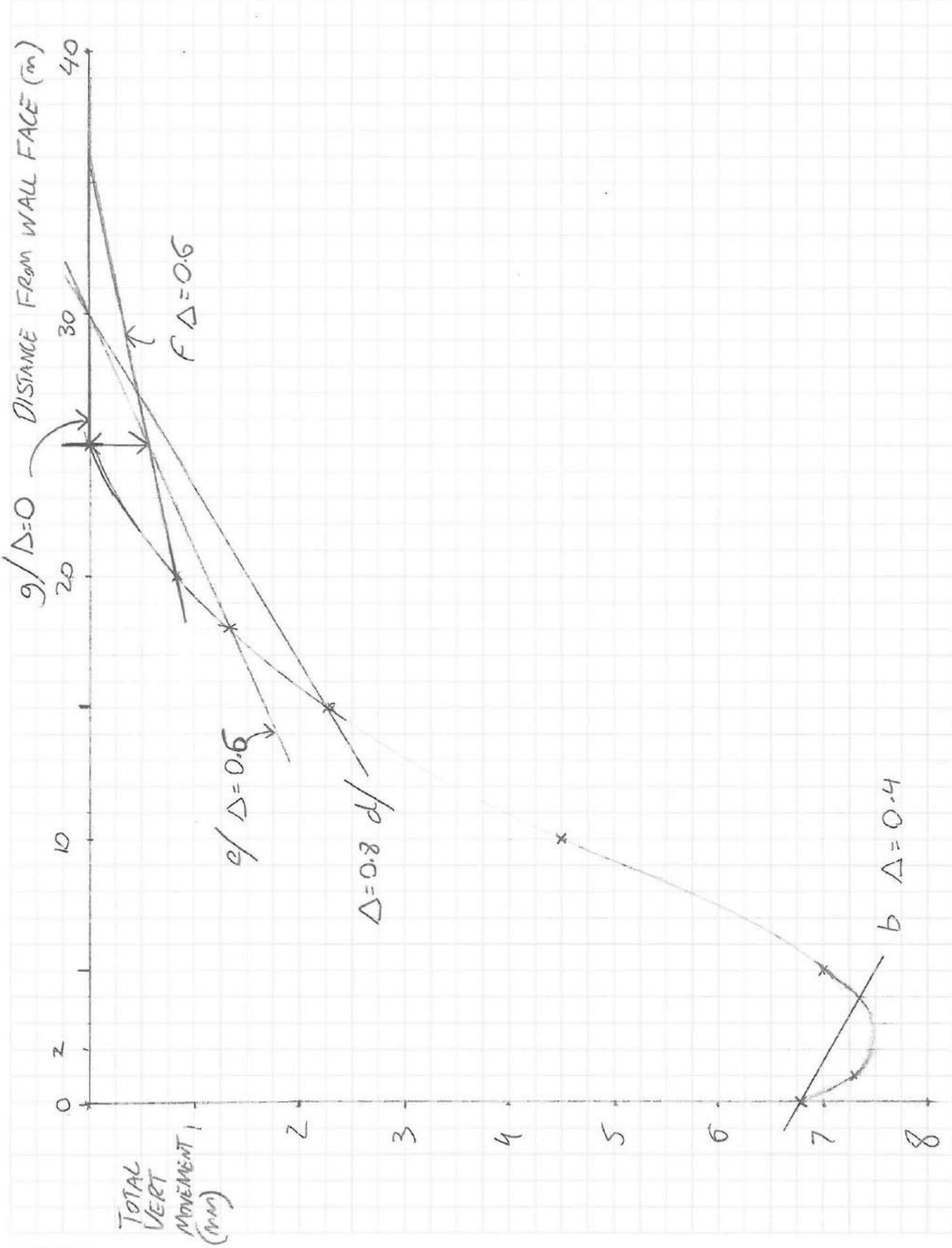
Date NOV 15	Job no. 1706/02	Sheet 17
Engineer FGo		
Checked by SPx		
Project ELLERDALE ROAD		



Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 18
Engineer FGO		
Checked by SPB		
Project ELLERDALE ROAD		



Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

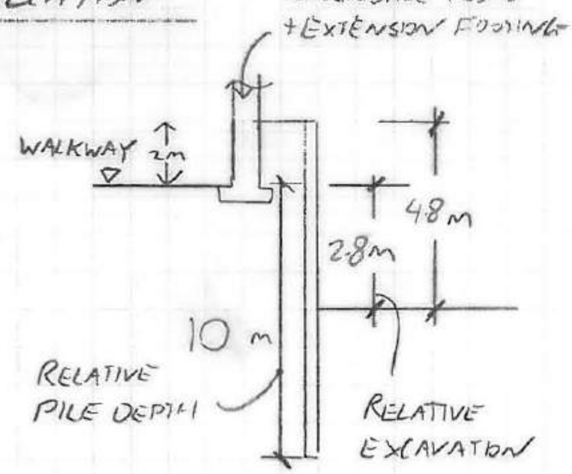
Date NOV 2015	Job no. 1706/02	Sheet 19
Engineer FGo		
Checked by SBa		
Project ELLERDALE ROAD		

GROUND MOVEMENT ESTIMATE IN ACCORDANCE WITH CIRIA C580 FOR BUILDINGS a/ + c/

ESTIMATE MOVEMENTS USING EMPIRICAL RELATIONSHIPS AS PER C580 - EMBEDED RETAINING WALLS. BUILDINGS a/ AND c/ ARE BEING CONSIDERED SEPARATELY AS THEY ARE KNOWN TO BE FOUNDED 2M BELOW GROUND LEVEL

1- MOVEMENT DUE TO PILE INSTALLATION

BASED ON FIGURE 2.8 C580



DISTANCE FROM FACE OF WALL	MOVEMENT DUE TO PILE INSTALLATION HORIZONTAL (mm)	VERTICAL (mm)
0	4	4
2	3.2	3.6
4	2.4	3.2
6	1.8	2.8
8	1.3	2.4
10	0.9	2.0
12	0.5	1.6
14	0.2	1.2
20	0	0

DISTANCE BEHIND WALL TO NEGLIGIBLE HORIZONTAL MOVEMENT

$= 1.5d = 1.5 \times 10 = 15m$

DISTANCE BEHIND WALL TO NEGLIGIBLE VERTICAL MOVEMENT

$= 2.0d = 2.0 \times 10 = 20m$

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no.	Sheet
Engineer FGo	1706/02	20
Checked by SBo		
Project ELLERDALE ROAD		

2- MOVEMENT DUE TO EXCAVATION

ASSUME HIGH SUPPORT STIFFNESS AS TEMPORARY PROPS INSTALLED BEFORE PERMANENT PROPS (SLABS) AT HIGH LEVEL

[TABLE 2.3]

DEPTH OF EXCAVATION - RELATIVE TO 1 ELLERDALE ROAD = 2.8m
BASED ON FIGURE 2.11

DISTANCE FROM FACE OF WALL (m)	MOVEMENT DUE TO EXCAVATION HORIZONTAL (mm)	VERTICAL (mm)
0	4.2	1.1
2	3.5	2.1
4	2.7	1.6
6	2.0	1.0
8	1.2	0.5
10	0.4	0
12	0	0
14	0	0

DISTANCE BEHIND WALL TO NEGLIGIBLE HORIZONTAL MOVEMENT

$= 4.0 \times d = 4.0 \times 2.8 = 11.2$ m

DISTANCE BEHIND WALL TO NEGLIGIBLE VERTICAL MOVEMENT

$= 3.5 \times d = 3.5 \times 2.8 = 9.8$ m

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date	Job no.	Sheet
NOV 2015	1706/02	21
Engineer FG		
Checked by SG		
Project ELLERDALE ROAD		

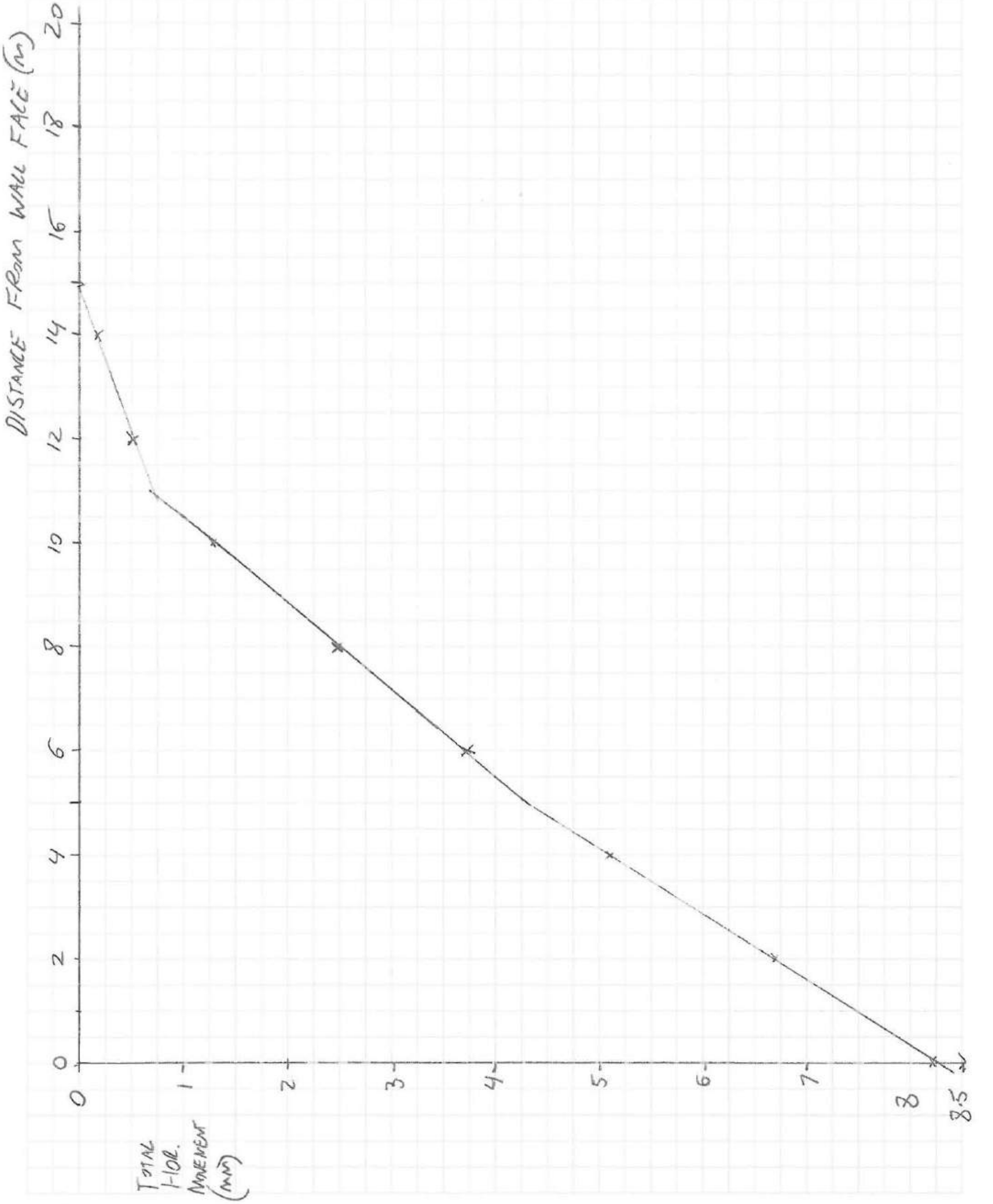
TOTAL GROUND SURFACE MOVEMENTS

DISTANCE FROM WALL FACE (M)	TOTAL MOVEMENT HORIZONTAL (MM)	VERTICAL (MM)
0	8.2	5.1
2	6.7	5.7
4	5.1	4.8
6	3.8	3.8
8	2.5	2.9
10	1.3	2.0
12	0.5	1.6
14	0.2	1.2
20	0	0

Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

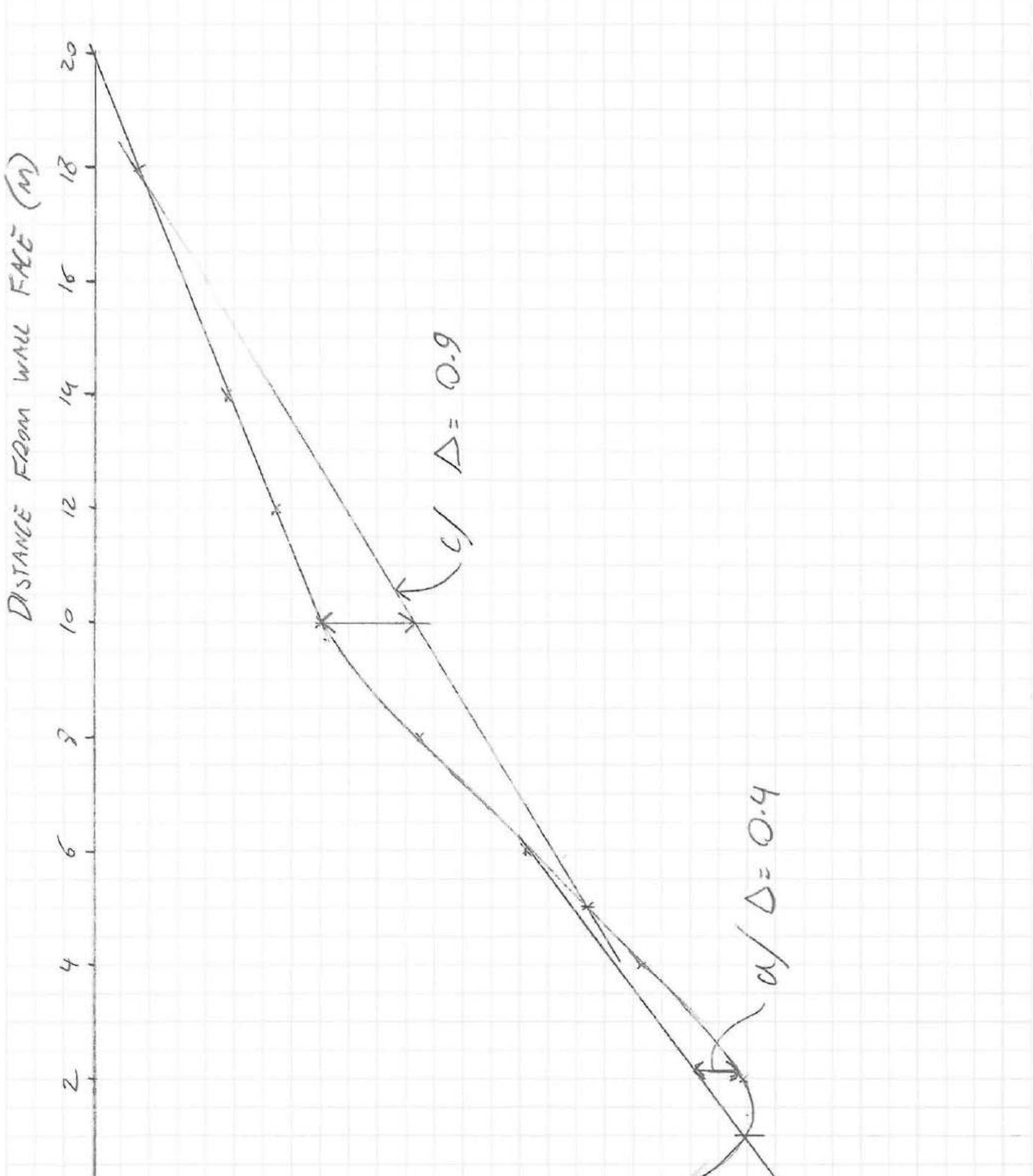
Date	NOV 2015
Engineer	FGo
Checked by	SBe
Job no.	1706/02
Sheet	22
Project	ELLERDALE ROAD

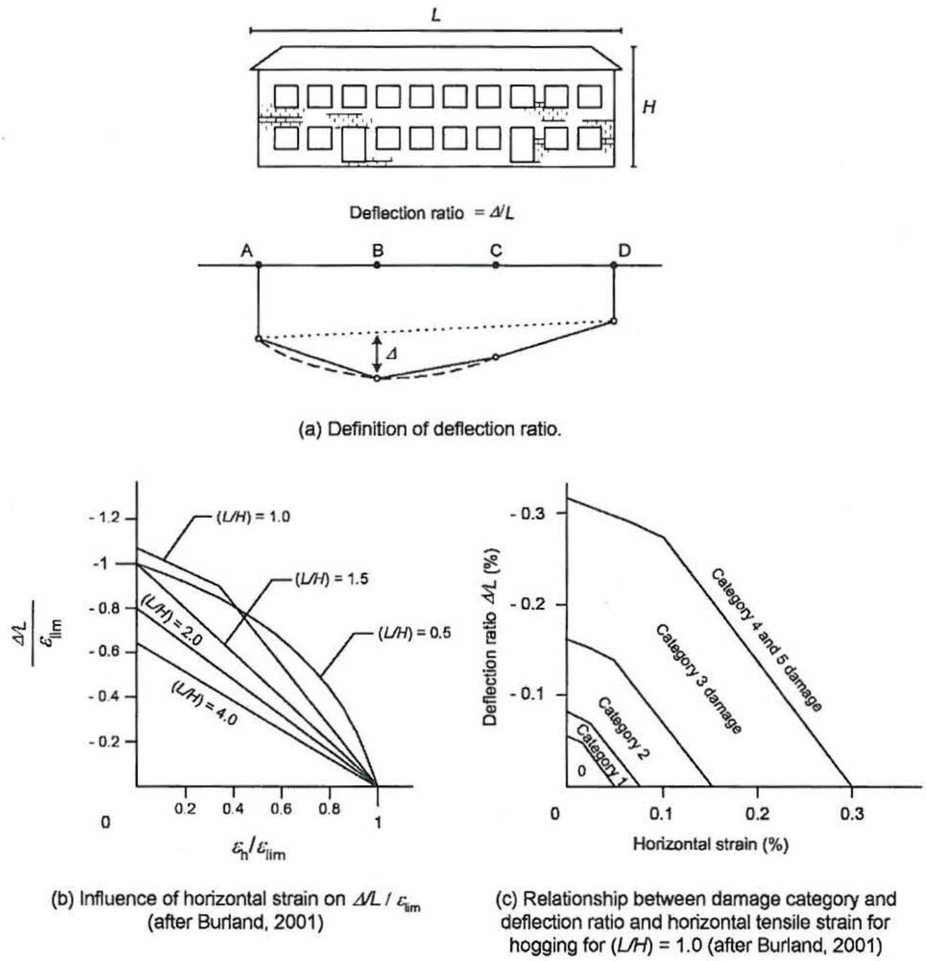


Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 23
Engineer FGo		
Checked by Sbe		
Project ELLERDALE ROAD		





(a) Definition of deflection ratio.

(b) Influence of horizontal strain on Δ/L / ε_{lim} (after Burland, 2001)

(c) Relationship between damage category and deflection ratio and horizontal tensile strain for hogging for $(L/H) = 1.0$ (after Burland, 2001)

By adopling values of ε_{lim} associated with the various damage calgories given in Table 2.5, Figure (b) can be developed into an interaction diagram showing the relationship between Δ/L and ε_h for a particular value of L/H Figure (c) shows such a diagram for $(L/H) = 1.0$

Figure 2.18 *Relationship between damage category, deflection ratio and horizontal tensile strain (after Burland, 2001)*

Reinforced concrete-framed structures are more flexible in shear than are masonry structures and are consequently less susceptible to damage. Nevertheless, for the purposes of a stage 2 assessment of potential damage, all structures should be treated as masonry structures.

Box 2.5 *Procedure for stage 2 damage category assessment*

The following steps should be undertaken in making a stage 2 assessment of the damage to a structure:

(i) establish L and H for the structure (see Figure 2.18(a) for definitions of L and H)

(ii) determine (L/H)

(iii) determine relationship between (Δ/L) and ε_h for the required (L/H) from Figure 2.18(b) for ε_{lim} values from Table 2.5

(iv) estimate vertical and horizontal ground surface movements in the vicinity of the structure from Figure 2.14

(v) determine (Δ/L) and ε_h $(= \delta_h/L)$ where δ_h is the horizontal movement

(vi) estimate damage category from the relationship between (Δ/L) and ε_h established from step (iii) above.

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	<u>Fine cracks that can easily be treated during normal decoration.</u> Perhaps isolated slight fracture in building. Cracks in external brickwork visible on inspection.	< 1	0.05–0.075
2 Slight	<u>Cracks easily filled. Redecoration probably required.</u> Several slight fractures showing inside of building. Cracks are visible externally and <u>some repointing may be required externally</u> to ensure weathertightness. Doors and windows may stick slightly.	< 5	0.075–0.15
3 Moderate	<u>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.</u> 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	<u>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.</u> 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	<u>This requires a major repair involving partial or complete rebuilding.</u> 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.

Alan Baxter

75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 26
Engineer FGo		
Checked by Sbe		
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	7.4	6	3.75	3.65	5	0.4	0.073	0.008
b/	0	12	4	8.8	3.2	4	0.4	0.08	0.01
c/	5	4.3	18	0	4.3	13	0.9	0.03	0.007
d/	15	2.2	30	0	2.2	15	0.8	0.015	0.005
e/	18	0	30	0	0	13	0.6	0	0.005
f/	20	0	35	0	0	15	0.6	0	0.005
g/	25	0	50	0	0	20	0	0	0

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

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

DAMAGE ASSESSMENT

THE HORIZONTAL AND VERTICAL STRAIN WILL BE USED TO ASSESS THE BURLAND 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 0.15% (BURLAND CAT 2 = SLIGHT)

STRUCTURE

a/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.008}{0.15} = 0.053$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.073}{0.15} = 0.49$$

$L/H = 1.7$ FROM FIGURE 2.18 → OKAY

b/

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

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.08}{0.15} = 0.53$$

$L/H = 2.7$ FROM FIGURE 2.18 → OKAY

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

c/

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

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

$L/H = 1.1$ FROM FIGURE 2.18 → OKAY

Alan Baxter
75 Cowcross Street
London EC1M 6EL
tel 020 7250 1555
email aba@alanbaxter.co.uk
www.alanbaxter.co.uk

Date NOV 2015	Job no. 1706/02	Sheet 28
Engineer FGo		
Checked by Slo		
Project ELLERDALE ROAD		

STRUCTURE

d/

$$\frac{\Delta/L}{\varepsilon_{lim}} = \frac{0.005}{0.05} = 0.1$$

$$\frac{\varepsilon_h}{\varepsilon_{lim}} = \frac{0.015}{0.05} = 0.3$$

$L/H = 1.3$ FROM FIGURE 2.18 → OKAY.

SUMMARY

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

$\left.\begin{matrix} e/ \\ f/ \\ g/ \end{matrix}\right\}$ BY INSPECTION

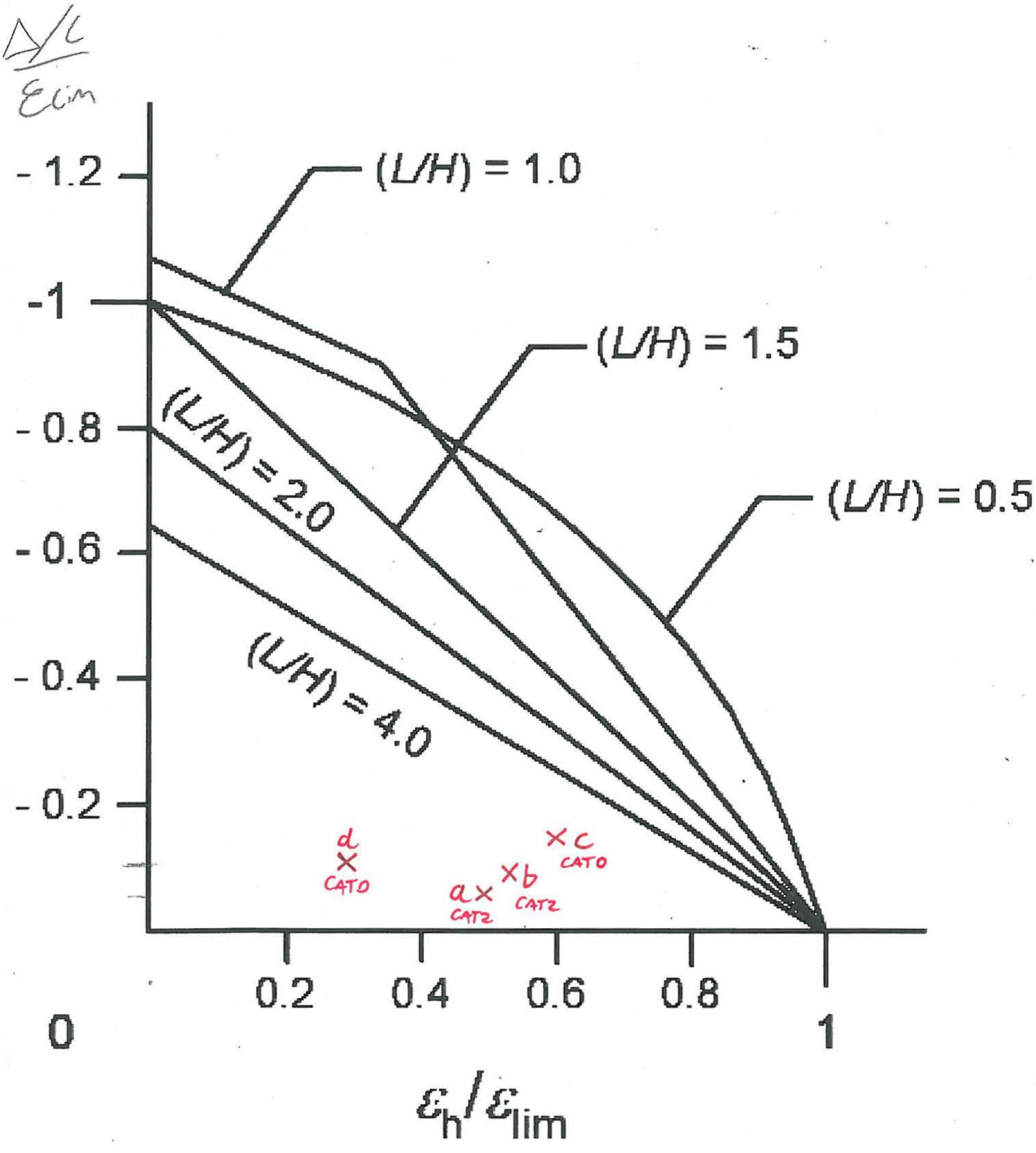


FIGURE 2.18 · C580

Appendix L – Ground Engineering 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

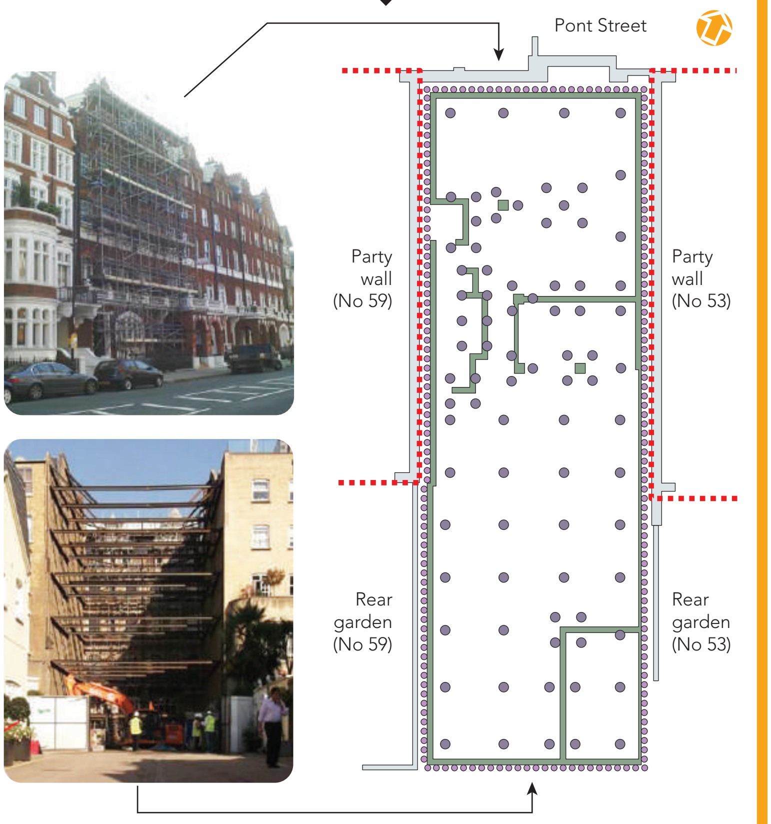


Figure 1 Site views and plan layout

FIGURE 2

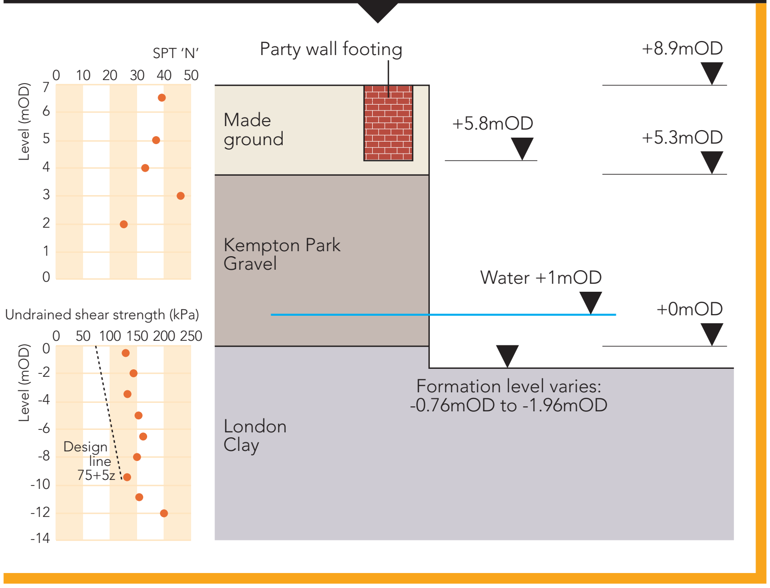


Figure 2 Conceptual site model

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/m^3)	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

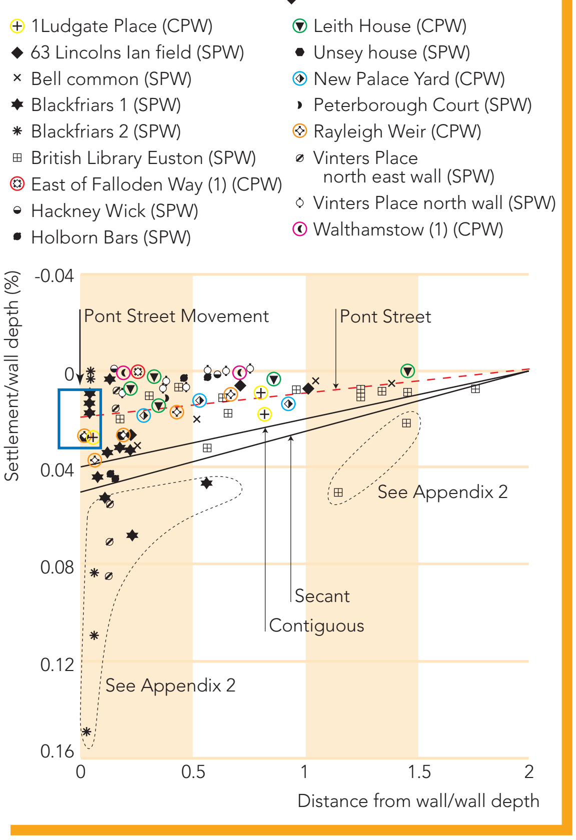


Figure 5 Case study data CIRIA C580 – with authors' rationalisation

FIGURE 6

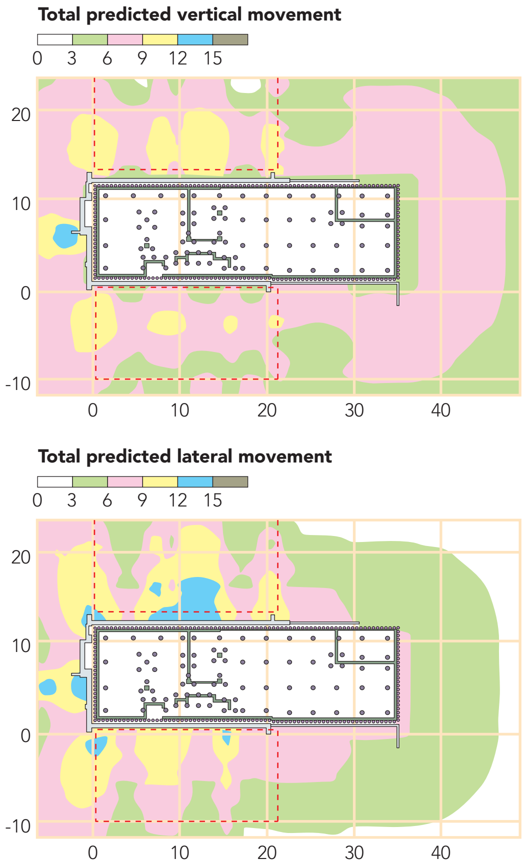
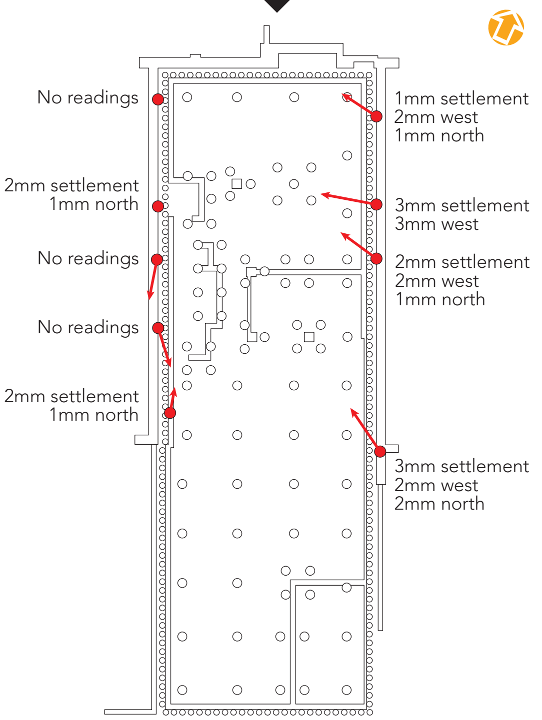


Figure 6 Predicted ground movements (total)

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]

Acknowledgements

The authors would like to thank their colleagues in Galliford Try and CGL together with the other members of the design and construction teams who have assisted in the collation of this data and the successful outcome of the project.

References

Gaba et al (2003). Ciria C580, Embedded retaining walls – guidance for economic design

Fernie et al (2003). Movement and deep basement provision at Knightsbridge Crown Court, Harrods, London, in Ciria Special Publication 199, Response of buildings to excavation-induced ground movements.

Peck, RB; Hanson, WE; and Thornburn, TH (1967) Foundation Engineering, 2nd edition. John Wiley, New York, p310.

Patel, D (1992). Interpretation of results of pile tests in London Clay, Proceedings of the conference of piling in Europe. Institution of Civil Engineers.

Burland, JB; Standing, JR; Jardine, FM (2001). Building response to tunnelling, case studies from construction of the Jubilee Line Extension, London, Volume 1: Projects and Methods, Ciria special publication 200

Burland, JB;Wroth, CP (1974). Settlement of buildings and associated damage, State of the art review. Conf on Settlement of Structures, Cambridge, Pentech Press, London, pp611-654

Boscardin, MD; Cording, EG (1989). Building response to excavation induced settlement. J Geotech Eng, ASCE, 115 (1); pp 1-21.

Bowles, JE (1988). Foundation Analysis and Design, 4th edition, McGraw-Hill International