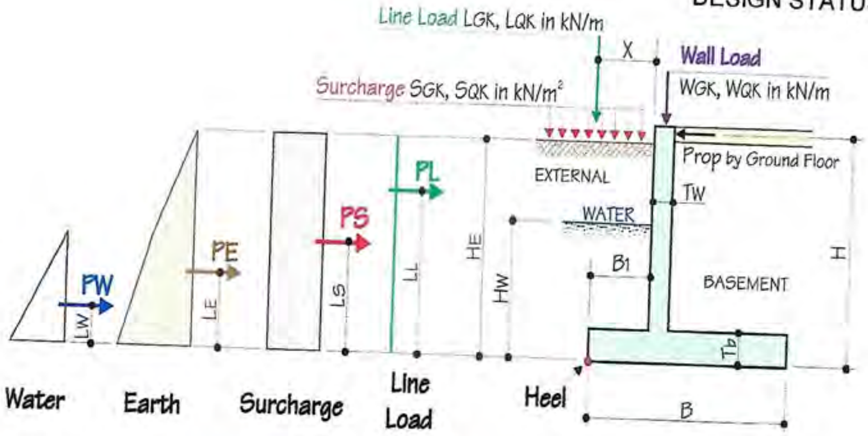
Project	21 BALDWIN GARDENS/43 LEATHER LANE	RP DESIGNS		
Client	RJ	Made by	Date	Page
Location	Gridline D E F	RN	18.11.23	502
	Basement wall design to BS8110:2005	Checked	Revision	Job No
	Originated from 'RCC61 Basement Wall.xls' v3.1 © 2006 TCC	NH	-	2198

The Concrete Centre

IDEALISED STRUCTURE and FORCE DIAGRAMS

DESIGN STATUS : VALID



DIMENSION (mm)

H =	2550	B =	3500	Tw =	330
Hw =	500	Bl =	25	Tb =	330
He =	2550				

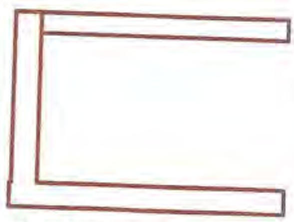
MATERIAL PROPERTIES

			steel class	A	
fcu =	35	N/mm^2	γm =	1.50	concrete
fy =	500	N/mm^2	γm =	1.15	steel

Cover to tension reinforcement (co) = 40 mm

Max. allowable design surface crack width (W) = 0.3 mm (0.2 or 0.3 mm only)

Concrete density = 24.0 kN/m3



Wall Geometry

SOIL PROPERTIES

Design angle of int'l friction of retained mat'l (Ø) =	30	degree	
Design cohesion of retained mat'l (C) =	0	kN/m2	(Only granular backfill considered, ie "C" = 0)
Density of retained mat'l (q) =	20	kN/m3	
Submerged Density of retained mat'l (qs) =	13.33	kN/m3	(default=2/3 of q), only apply when Hw >0
Design angle of int'l friction of base mat'l (Øb) =	20	degree	= 13.33
Design cohesion of base mat'l (Cb) =	20	kN/m2	
Density of base mat'l (qb) =	10	kN/m3	
Allowable gross ground bearing pressure (GBP) =	200	kN/m2	

ASSUMPTIONS

a) Wall friction is zero

b) Minimum active earth pressure = 0.25qH

c) Granular backfill

h) Design not intended for walls over 3.5 m high

i)Does **not** include check for temp or shrinkage

LOADINGS (unfactored)

Surcharge load -- live (SQK) =	0	kN/m2
Surcharge load -- dead (SGK) =	0	kN/m2
Line load -- live (LQK) =	17	kN/m
Line load -- dead (LGK) =	99	kN/m
Distance of line load from wall (X) =	350	mm
Wall load -- live (WQK) =	10	kN/m
Wall load -- Dead (WGK) =	114	kN/m

LATERAL FORCES

Ko = 0.50 default Ko = (1-SIN Ø) 0.50

Kac = 1.41 = $2Ko^{0.5}$

Force (kN)		Lever arm (m)		γ_f	Ultimate Force (kN)
PE =	32.10	LE =	0.859	1.40	44.93
PS(GK) =	0.00	LS =	1.28	1.40	0.00
PS(QK) =	0.00	LS =	1.28	1.60	0.00
PL(GK) =	49.50	LL =	2.26	1.40	69.30
PL(QK) =	8.50	LL =	2.26	1.60	13.60
PW =	1.25	LW =	0.17	1.40	1.75
Total	91.35				129.58

Project	21 BALDWIN GARDENS/43 LEATHER LANE	RP DESIGNS		
Client	RJ	Made by	Date	Page
Location	Gridline D E F	RN	18.11.23	503
	Basement wall design to BS8110:2005	Checked	Revision	Job No
	Originated from 'RCC61 Basement Wall.xls' v3.1 © 2006 TCC	NH	-	2198

The Concrete Centre

EXTERNAL STABILITY

STABILITY CHECK : OK

ANALYSIS - Assumptions & Notes

1) Wall idealised as a propped cantilever (i.e. pinned at top and fixed at base)
2) Wall is braced.
3) Maximum slenderness of wall is limited to 15, i.e [0.9*(He-Tb/2)/Tw < 15]
4) Maximum Ultimate axial load on wall is limited to 0.1fcu times the wall cross-sectional area
5) Design Span (Effective wall height) = He - (Tb/2)
6) -ve moment is hogging (i.e. tension at external face of wall)
   +ve moment is sagging (i.e. tension at internal face of wall)
7) " Wall MT. " is maximum +ve moment on the wall.
8) Estimated lateral deflections are used for checking the **P**Δ effect .

UNFACTORED LOADS AND FORCES

Lateral Force	Force (kN)	Lever arm to base (m)	Base MT. (kNm)	Wall MT. (kNm)	Reaction at Base (kN)	Reaction at Top (kN)	Estimated Elastic Deflection Δ (mm)
PE =	28.25	0.80	-9.03	4.06	22.57	5.69	0.0
PS(GK) =	0.00	1.19	0.00	#DIV/0!	0.00	0.00	0.0
PS(QK) =	0.00	1.19	0.00	0.00	0.00	0.00	0.0
PL(GK) =	49.50	2.09	-7.16	11.87	9.10	40.40	0.0
PL(QK) =	8.50	2.09	-1.23	2.04	1.56	6.94	0.0
PW =	0.56	0.11	-0.06	0.01	0.56	0.00	0.0
Total	86.81		-17.47	#DIV/0!	33.78	53.03	0.1

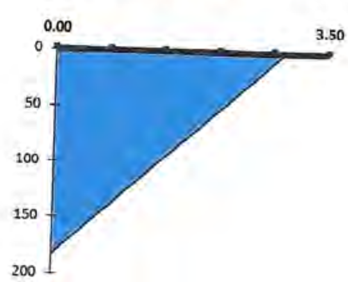
GROUND BEARING FAILURE

LOAD CASE: Wall Load **MAX**
Surcharge **MIN**

Taking moments about centre of base (anticlockwise "+")

Vertical FORCES (kN)		Lever arm (m)	Moment (kNm)
Wall load =	124	1.56	193.44
Wall (sw) =	17.58	1.56	27.43
Base =	27.72	0.00	0.00
Earth =	1.08	1.74	1.88
Water =	0.04	1.74	0.07
Surcharge =	0.00	1.74	0.00
Line load =	99.00	0.00	0.00
Σ V =	269.43	Σ Mv =	222.82

BEARING PRESSURE (kN/m²)



MOMENT due to LATERAL FORCES, Mo = -16.24 kNm

RESULTANT MOMENT, M = Mv + Mo = 206.58 kNm

ECCENTRICITY FROM BASE CENTRE, M / V = 0.77 m

MAXIMUM GROSS BEARING PRESSURE = 182.68 kN/m^2 < 200 OK

SLIDING AT BASE

(using overall factor of safety instead of partial safety fact F.O.S = 1.50

SUM of LATERAL FORCES, **P** = 33.78 kN

BASE FRICTION, $\mathbf{F_b}$ = - (**V** TANØb + B.Cb) = -168.06 kN

Factor of Safety, F_b / P = 4.97 > 1.50 OK

Project	21 BALDWIN GARDENS/43 LEATHER LA	RP DESIGNS			
Client	RJ	Made by	Date		Page
Location	Gridline D E F	RN	18.11.23		505
	Basement wall design to BS8110:2005	Checked	Revision		Job No
	Originated from 'RCC61 Basement Wall.xls' v3.1 © 2006 TCC	NH	-		2198

The Concrete Centre

OUTER BASE (per metre length) — BS8110 reference

γ_f = 1.50 (ASSUMED)

Ult. Shear = 17.17 kN (AT d from FACE of WALL)

Ult. MT. = -0.02 kNm TENSION - TOP FACE

BOTTOM REINFORCEMENT :

Min. As =	429	mm^2			Table 3.25
ϕ =	12	mm			
centres =	225	mm	< 762	OK	
As =	503	mm^2	> 429	OK	

MOMENT of RESISTANCE :

d =	284	mm			
Z =	270	mm			3.4.4.4
As' =	0	mm^2			
Mres =	58.96	kNm	> 0.02	OK	

SHEAR RESISTANCE:

100As/bd =	0.18%				
vc =	0.43	N/mm^2			Table 3.8
Vres =	122.81	kN	> 17.17	OK	3.5.5.2

CHECK CRACK WIDTH IN ACCORDANCE WITH BS8100/80(Temp & shrinkage effects not included

X = 58.43 mm ε_m = -0.00163 — BS8007

Acr = 115.54 mm W = -0.36 mm < 0.30 OK — App. B.2

NO CRACKING

INNER BASE (per metre length)

Ult. Shear = -62.03 kN (AT d from FACE of WALL)

Ult. MT. = 26.19 kNm TENSION - BOTTOM FACE

BOTTOM REINFORCEMENT :

Min. As =	429	mm^2			Table 3.25
ϕ =	12	mm			
centres =	225	mm	< 762	OK	
As =	503	mm^2	> 429	OK	

MOMENT of RESISTANCE :

d =	284	mm			
Z =	270	mm			
As' =	0	mm^2			
Mres =	58.96	kNm	> 26.19	OK	3.4.4.4

SHEAR RESISTANCE:

100As/bd =	0.18%				
vc =	0.43	N/mm^2			Table 3.8
Vres =	122.81	kN	> 62.03	OK	3.5.5.2

CHECK CRACK WIDTH IN ACCORDANCE WITH BS8100/80(Temp & shrinkage effects not included

X = 58.43 mm ε_m = -0.00084 — BS8007

Acr = 115.54 mm W = -0.19 mm < 0.30 OK — App. B.2

NO CRACKING

REINFORCEMENT SUMMARY for BASE

	Type	ϕ mm	centres mm	As mm^2	Min. As mm^2	
TOP	H	12	225	503	429	OK
BOTTOM	T	12	225	503	429	OK
TRANSVERSE	T	12	225	503	429	OK

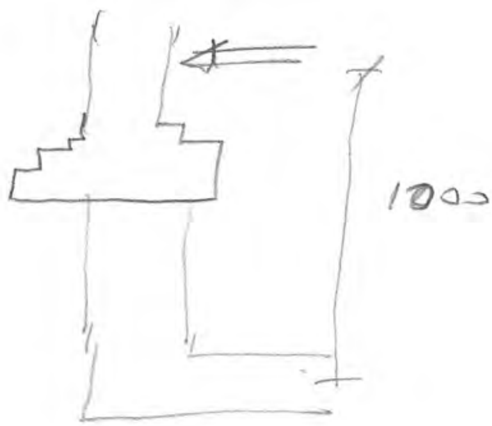
RP Designs
61 Barnes Wallis Court
Barnhill Road
HA9 9DW
Wembley
Tel 02089046856 Mob 07931745281
drawing@rp-designs.co.uk / rnara99388@hotmail.com

Calculation sheet		
Job no. 2198	Sheet no. 400	Rev
By RN	Date 1.08.23	Chkd
Site 21 BALDWIN GARDENS		

43 LEATHER LANE

propping for temporary works

UNDERPIN type 1 MASS concrete underpinning



load on the wall

WALL	(2.1×25+0.15×2)×16 = ...	90 kN/m
TIMBER floor	(1+1.5)× $\frac{4.2}{2}$ × 3 =	16 kN/m
Roof	$\frac{4.2}{2}$ × 1.5 =	3.20 kN
GF concrete floor	$\frac{4.2}{2}$ × 6.5 =	13.7 kN/m
		123

90 kN/m	13 kN/m
6.5 kN/m	1.6 kN/m
1.6 kN/m	6.3 kN/m
10.5 kN/m	
109 kN	22 kN/m

propping force 0 but use

6.1 kN/m × 2m = 12.2 kN USE

RP Designs
61 Barnes Wallis Court
Barnhill Road
HA9 9DW
Wembley
Tel 02089046856 Mob 07931745281
drawing@rp-designs.co.uk / rnara99388@hotmail.com

Calculation sheet

Job no. 2198	Sheet no. 49	Rev
By RN	Date 1.08.23	Chkd
Site 21 BALDWIN GARDENS		

43 LEATHEM LANE

Propping force is 0.0

but passive resistance impact of

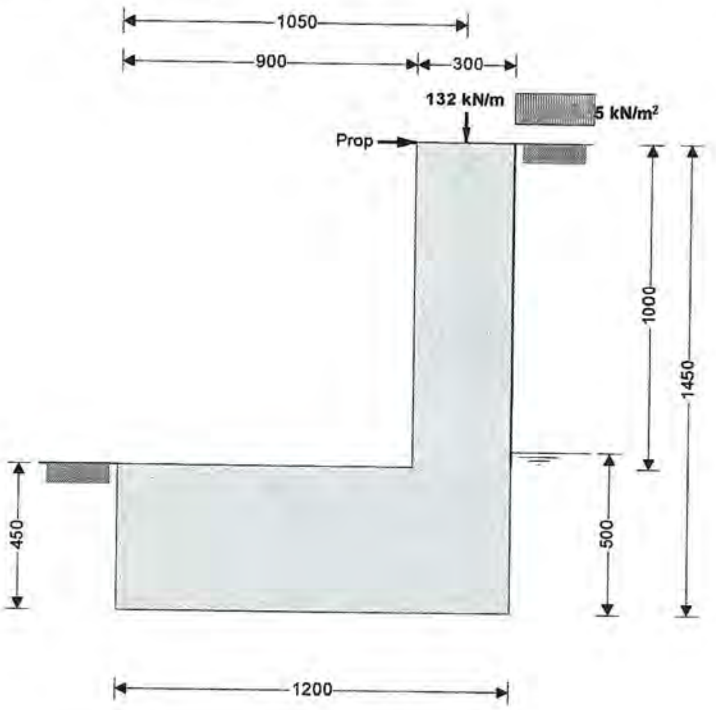
wall 6.1 kN/m prop length 4000

capacity of prop size 4 = 10 kN

USE every 1500mm c/c

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at top**
Height of retaining wall stem	h_{stem} = **1000** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **900** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1200** mm
Thickness of base	t_{base} = **450** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **750** mm
Thickness of downstand	t_{ds} = **450** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **1450** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **500** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **50** mm
Density of wall construction	γ_{wall} = **23.6** kN/m³
Density of base construction	γ_{base} = **23.6** kN/m³
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **1450** mm

Retained material details

Mobilisation factor	M = **1.5**
Moist density of retained material	γ_m = **18.0** kN/m³
Saturated density of retained material	γ_s = **21.0** kN/m³
Design shear strength	ϕ' = **24.2** deg
Angle of wall friction	δ = **0.0** deg

Base material details

Moist density	γ_{mb} = **18.0** kN/m³
Design shear strength	ϕ'_b = **24.2** deg
Design base friction	δ_b = **18.6** deg
Allowable bearing pressure	$P_{bearing}$ = **150** kN/m²

Using Rankine theory

Active pressure coefficient for retained material $K_a = (\cos(\beta) - \sqrt{[(\cos(\beta))^2 - (\cos(\phi'))^2]}) / (\cos(\beta) + \sqrt{[(\cos(\beta))^2 - (\cos(\phi'))^2]})$

K_a = **0.419**

Passive pressure coefficient for base material $K_p = (1 + \sqrt{[1 - (\cos(\phi'_b))^2]}) / (1 - \sqrt{[1 - (\cos(\phi'_b))^2]})$ = **2.389**

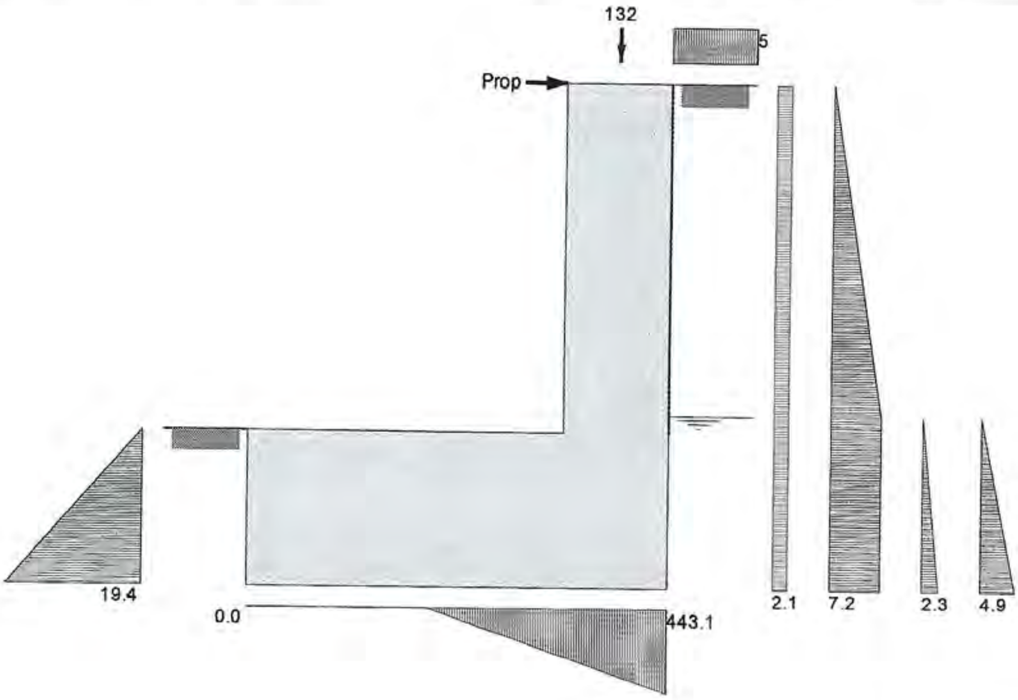
At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

Surcharge load on plan	Surcharge = **5.0** kN/m²
Applied vertical dead load on wall	W_{dead} = **110.0** kN/m
Applied vertical live load on wall	W_{live} = **22.0** kN/m
Position of applied vertical load on wall	l_{load} = **1050** mm
Applied horizontal dead load on wall	F_{dead} = **0.0** kN/m
Applied horizontal live load on wall	F_{live} = **0.0** kN/m
Height of applied horizontal load on wall	h_{load} = **0** mm

Tekla Tedds RP DESIGNS 61 BARNES WALLIS COURT HA9 9DW	Project: 21 Baldwin Gradens/43 Leather Lane			Job no. 2198	
	Calcs for: Propping force in unerpin type 1			Start page no./Revision 404	
	Calcs by RN	Calcs date 02/08/2023	Checked by NH	Checked date	Approved by	Approved date



Loads shown in kN/m, pressures shown in kN/m²

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **7.1** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **12.7** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **132** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **151.8** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **3** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **3.4** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **3.6** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **0.6** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **1.2** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **11.8** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.4** kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0\ \text{kN/m})$

F_{prop} = **0.0** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **2.2** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **2.8** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **0.9** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.1** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.2** kNm/m

RP Designs
61 Barnes Wallis Court
Barnhill Road
HA9 9DW
Wembley
Tel 02089046856 Mob 07931745281
drawing@rp-designs.co.uk / rnara99388@hotmail.com

Calculation sheet

Job no. 219 B	Sheet no. 405	Rev
By RW	Date 1.08.23	Chkd
Site 21 BALDIN GARDENS		

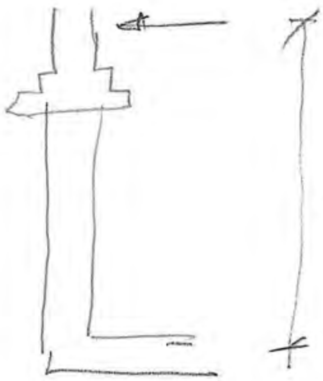
UNDERPIN type 2. Reinforced concrete retaining WALL

load on the WALL

WALL (2.1 x 2.5 + 0.15) x 4.5 = 25 KN/m

TIMBER floor. FLAT ROOF ($\frac{3}{2}$ x 1; 1.5) x 3 = 5 KN/m 7 KN/m

30 KN/m 7 KN/m



propping force 10 KN/m

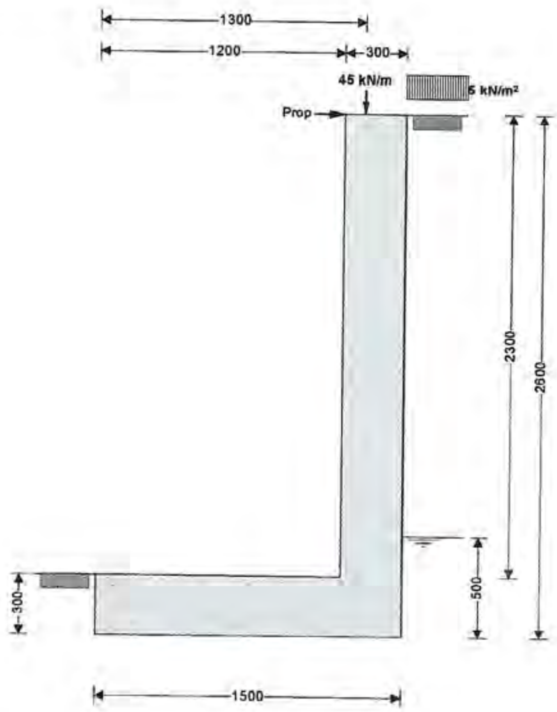
WALLER DESIGN

DESIGN BENDING MOMENT USE
5.2 KNm ≤ 55 KNm

shear 13.8 KN ≤ 40 KN
USE
MABEY
System 160
or 152 UC
23 SR
355

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at top**
Height of retaining wall stem	h_{stem} = **2300** mm
Thickness of wall stem	t_{wall} = **300** mm
Length of toe	l_{toe} = **1200** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1500** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **750** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2600** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **500** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **200** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2600** mm

Retained material details

Mobilisation factor $M = \mathbf{1.5}$

Moist density of retained material $\gamma_m = \mathbf{18.0}$ kN/m³

Saturated density of retained material $\gamma_s = \mathbf{21.0}$ kN/m³

Design shear strength $\phi' = \mathbf{24.2}$ deg

Angle of wall friction $\delta = \mathbf{0.0}$ deg

Base material details

Moist density $\gamma_{mb} = \mathbf{18.0}$ kN/m³

Design shear strength $\phi'_b = \mathbf{24.2}$ deg

Design base friction $\delta_b = \mathbf{18.6}$ deg

Allowable bearing pressure $P_{bearing} = \mathbf{150}$ kN/m²

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi') = \mathbf{0.590}$

Loading details

Surcharge load on plan Surcharge = **5.0** kN/m²

Applied vertical dead load on wall $W_{dead} = \mathbf{35.0}$ kN/m

Applied vertical live load on wall $W_{live} = \mathbf{10.0}$ kN/m

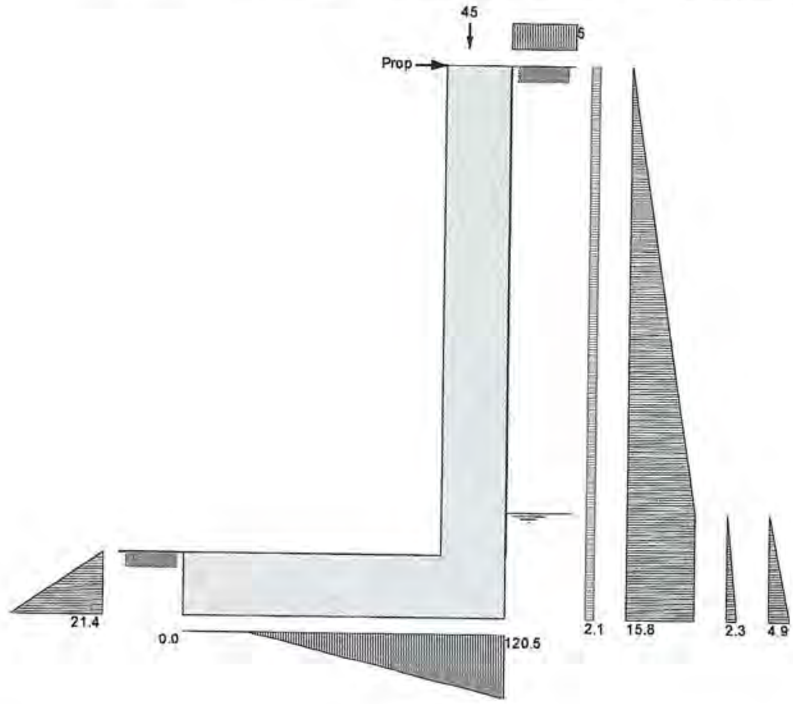
Position of applied vertical load on wall $l_{load} = \mathbf{1300}$ mm

Applied horizontal dead load on wall $F_{dead} = \mathbf{0.0}$ kN/m

Applied horizontal live load on wall $F_{live} = \mathbf{0.0}$ kN/m

Height of applied horizontal load on wall $h_{load} = \mathbf{0}$ mm

Tekla Tedds RP DESIGNS 61 BARNES WALLIS COURT HA9 9DW	Project 21 Baldwin Gradens/43 Leather Lane				Job no. 2198	
	Calcs for Propping force in underpin type 2				Start page no./Revision 408	
	Calcs by RN	Calcs date 03/08/2023	Checked by NH	Checked date	Approved by	Approved date



Loads shown in kN/m, pressures shown in kN/m²

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **16.3** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **10.6** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **45** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **71.9** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **5.4** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **16.6** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **7.9** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **0.6** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **1.2** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **31.8** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **7.7** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **7.1** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **19.9** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **2** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.1** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **0.2** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water} = \mathbf{29.3}$ kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2) = \mathbf{22}$ kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2 = \mathbf{8}$ kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load} = \mathbf{45.5}$ kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead} = \mathbf{75.4}$ kNm/m

Check bearing pressure

Propping force $M_{prop} = F_{prop} \times (h_{wall} - d_{ds}) = \mathbf{20.1}$ kNm/m

Design vertical live load $M_{live} = W_{live} \times l_{load} = \mathbf{13}$ kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{prop} + M_{live} = \mathbf{79.3}$ kNm/m

Total vertical reaction $R = W_{total} = \mathbf{71.9}$ kN/m

Distance to reaction $x_{bar} = M_{total} / R = \mathbf{1102}$ mm

Eccentricity of reaction $e = abs((l_{base} / 2) - x_{bar}) = \mathbf{352}$ mm

Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe} = 0 \text{ kN/m}^2 = \mathbf{0}$ kN/m²

Bearing pressure at heel $p_{heel} = R / (1.5 \times (l_{base} - x_{bar})) = \mathbf{120.5}$ kN/m²

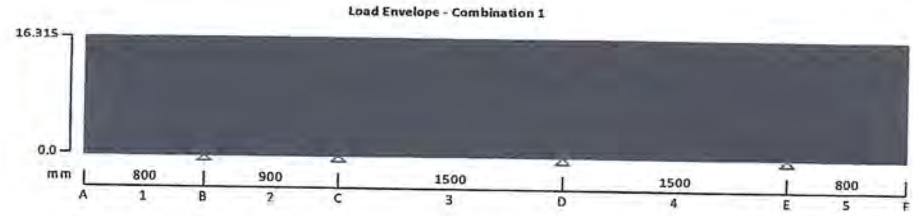
PASS - Maximum bearing pressure is less than allowable bearing pressure

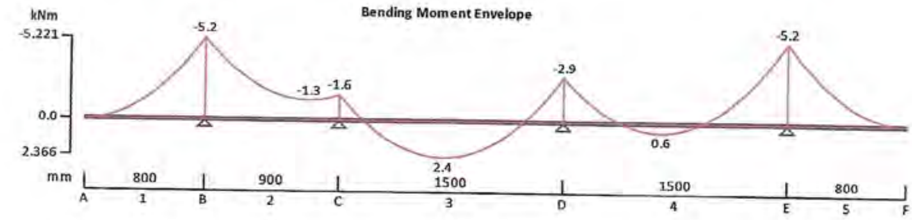
Tekla Tedds RP DESIGNS 61 BARNES WALLIS COURT HA9 9DW	Project 21 Baldwin Gradens/43 Leather Lane				Job no. 2198	
	Calcs for Waller Design				Start page no./Revision 410	
	Calcs by RN	Calcs date 03/08/2023	Checked by NH	Checked date	Approved by	Approved date

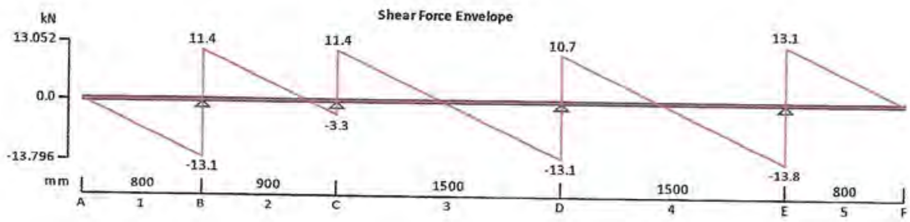
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.05







Support conditions

Support A	Vertically free
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free
Support E	Vertically restrained
	Rotationally free
Support F	Vertically free
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Imposed full UDL 10 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support E	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support F	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **2.4** kNm	M_{min} = **-5.2** kNm
Maximum moment span 1	M_{s1_max} = **0** kNm	M_{s1_min} = **-5.2** kNm
Maximum moment span 2	M_{s2_max} = **-1.3** kNm	M_{s2_min} = **-5.2** kNm
Maximum moment span 3	M_{s3_max} = **2.4** kNm	M_{s3_min} = **-2.9** kNm
Maximum moment span 4	M_{s4_max} = **0.6** kNm	M_{s4_min} = **-5.2** kNm
Maximum moment span 5	M_{s5_max} = **0** kNm	M_{s5_min} = **-5.2** kNm
Maximum shear	V_{max} = **13.1** kN	V_{min} = **-13.8** kN
Maximum shear span 1	V_{s1_max} = **0** kN	V_{s1_min} = **-13.1** kN
Maximum shear span 2	V_{s2_max} = **11.4** kN	V_{s2_min} = **-3.3** kN
Maximum shear span 3	V_{s3_max} = **11.4** kN	V_{s3_min} = **-13.1** kN
Maximum shear span 4	V_{s4_max} = **10.7** kN	V_{s4_min} = **-13.8** kN
Maximum shear span 5	V_{s5_max} = **13.1** kN	V_{s5_min} = **0** kN
Deflection	δ_{max} = **0.5** mm	δ_{min} = **0.1** mm
Deflection span 1	δ_{s1_max} = **0.5** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0** mm	δ_{s2_min} = **0.1** mm
Deflection span 3	δ_{s3_max} = **0.1** mm	δ_{s3_min} = **0** mm
Deflection span 4	δ_{s4_max} = **0** mm	δ_{s4_min} = **0** mm
Deflection span 5	δ_{s5_max} = **0.4** mm	δ_{s5_min} = **0** mm

Maximum reaction at support A R_{A_max} = **0** kN R_{A_min} = **0** kN

Maximum reaction at support B R_{B_max} = **24.4** kN R_{B_min} = **24.4** kN

Unfactored dead load reaction at support B R_{B_Dead} = **0.3** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **15** kN

Maximum reaction at support C R_{C_max} = **14.7** kN R_{C_min} = **14.7** kN

Unfactored dead load reaction at support C R_{C_Dead} = **0.2** kN

Unfactored imposed load reaction at support C $R_{C\_Imposed}$ = **9** kN

Maximum reaction at support D R_{D_max} = **23.8** kN R_{D_min} = **23.8** kN

Unfactored dead load reaction at support D R_{D_Dead} = **0.3** kN

Unfactored imposed load reaction at support D $R_{D\_Imposed}$ = **14.6** kN

Maximum reaction at support E R_{E_max} = **26.8** kN R_{E_min} = **26.8** kN

Unfactored dead load reaction at support E R_{E_Dead} = **0.4** kN

Unfactored imposed load reaction at support E $R_{E\_Imposed}$ = **16.5** kN

Maximum reaction at support F R_{F_max} = **0** kN R_{F_min} = **0** kN

Section details

Section type **UC 152x152x23 (BS4-1)** Steel grade **S275**

Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.00** Section classification **Semi-compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **13.8** kN Design shear resistance P_v = **145.8** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 - Section 4.2.5

Design bending moment M = **5.2** kNm Moment capacity low shear M_c = **48.5** kNm

Equivalent uniform moment factor - Section 4.3.6.6

Equiv uniform mnt factor LTB m_{LT} = **1.000**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **32.3** kNm M_b / m_{LT} = **32.3** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **2.222** mm Maximum deflection δ = **0.461** mm

PASS - Maximum deflection does not exceed deflection limit

2.2 Shutter Beams

2.2.1 Details & Section Properties

The Mabey Hire steel Shutter Beam is equivalent to a 9" x 3" SC4 timber waler.

Shutter Beam Properties	
Second Moment of Area	122 cm^4
Section Modulus	19 cm^3
Cross-Sectional Area	8.28 cm^2
Maximum Resisting Moment	5.5 kNm
Maximum Permissible Shear Capacity	90.0 kN
EI Value (E = 208,000N/mm^2)	250.0 kNm^2

Shutter Beam Lengths and Weights		
Code Number	Length (mm)	Weight (kg)
S3/2/0.46	460	3.00
S3/2/0.7	775	7.10
S3/2/1.55	1550	13.45
S3/2/2.3	2325	21.20
S3/2/3.1	3100	27.30
S3/2/3.8	3875	33.90
S3/2/4.6	4650	40.70
S3/2/6.2	6200	54.60

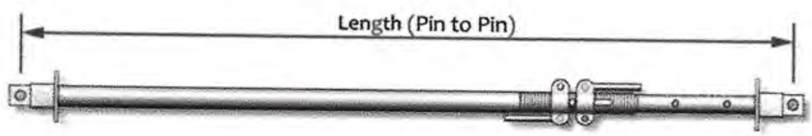
2.3 Plumbing Prop

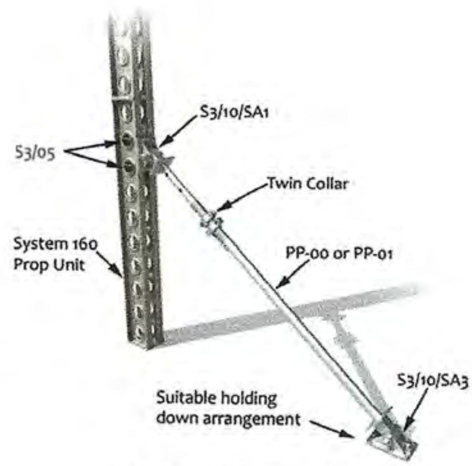
2.3.1 Details & Properties

The main application for this item is as a lightweight plumbing prop.

PP-00
S.W.L. at Min. length = 35 kN
S.W.L. at Max. length = 35 kN

PP-01
S.W.L. at Min. length = 35 kN
S.W.L. at Max. length = 17 kN





Typical Plumbing Prop Arrangement
(See table for alternative connection details)

Prop Connection Details		Prop Kit Codes	
Head of Prop	Base of Prop	PP-00	PP-01
MK3 Soldier	Concrete	PP/00A	PP/01A
Concrete	Concrete	PP/00C	PP/01C
MK3 Soldier	MK3 Soldier	PP/00D	PP/01D

Code	Length (mm)		Weight (kg)
	Min	Max	
PP-00	1270	2020	20
PP-01	1950	3320	26

Note: S3/10/SA1 or SA3 at each end adds 130mm to the Prop min & max lengths.

RP Designs
61 Barnes Wallis Court
Barnhill Road
HA9 9DW
Wembley
Tel 02089046856 Mob 07931745281
drawing@rp-designs.co.uk / rnara99388@hotmail.com

Calculation sheet		
Job no. 2198	Sheet no. 4/3	Rev
By Rr	Date 1.08 23	Chkd
Site 21 Baldwin Gardens		

propping Axial

MAXIMUM Axial 30kN

length 4000 mm

MAXIMUM CAPACITY 110kN USK

MAY 3K 160

OK

2.1.3 General Prop Unit Capacities

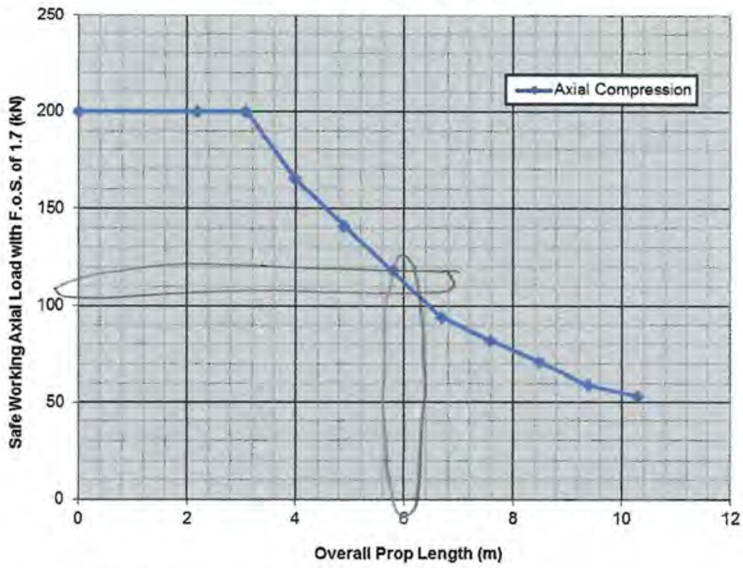
Major Axis (X-X) Capacities:-	Maximum bending moment suitably restrained assuming no local point load	60 kNm
	Maximum bending moment suitably restrained with 160 kN point load	30 kNm
	Maximum moment of resistance at joint (6 No. M16 bolts)	25 kNm
	Maximum shear capacity	120 kN
	Maximum point load when distributed evenly over an area of 150 x 150mm	160 kN

Minor Axis Bending Capacity:-	Maximum bending moment about Y-Y axis	7.5 kNm *

* **Note on Y-Y properties:** Performance in the Y-Y direction is relatively complex and values are based on testing and finite element analysis. Refer to Mabey Hire engineers if further information is required.

2.1.4 Load Capacity of Prop Units - Axial

Prop Loading Chart



Notes:

1. A factor of safety of 1.7 for axial prop loads was used.
2. When used in tension the Safe Working Load is 120 kN.
3. Values assume that the load is applied concentrically along the centre of the prop i.e no eccentricity.
4. In horizontal applications, the capacity above may be used providing that the props are used with the soldier webs / lightening holes in the vertical plane.
5. Longer props can be braced to reduce the effective length and improve the capacity.
6. All bolt joints require 6 No. S3/22 bolts.
7. Straightness requirement: When the prop is assembled, the maximum deviation from straight should be 2mm per metre length e.g. 10mm for a 5m prop. Should a prop fall outside this requirement, components should be checked for damage.
8. Limitations of End Fittings: Values may be limited by the type of end fittings used. In the absence of a standard end fitting, the load must be applied through a stiff surface such as a steel plate or uniform concrete surface.
9. Further information should be obtained from Mabey Hire engineers.

RP DESIGNS
21 BALDWIN GARDENS/43 LEATHER LANE
LONDON
EC1 7UY

RP Designs

61 Barnes Wallis Court
Barnhill Road
HA9 9DW
Wembley

Tel 02089046856 Mob 07931745281
drawing@rp-designs.co.uk / rnara99388@hotmail.com

.

CONSTRUCTION METHOD STATEMENT FINAL VERSION

FOR

PROPOSED BASEMENT EXTENSION
AT
BALDWIN GARDENS/43 LEATHER LANE
LONDON
EC1N 7UY

DATE: 18/12/2023

The following document is prepared for RP Designs on the instruction of, and for the sole use and benefit of the client.

The client is the main Contractor. For the above site: JV BUILDERS LIMITED. THEY ARE THE SOLELY RESPONSIBLE FOR THE CONSTRUCTION AND MANAGEMNT OF THE SITE.

Structural Summary

The property is an existing Town House containing an existing basement, ground and four floors above situated at the corner of the above address and is being refurbished to modern standards and includes a new loft floor above and new modified roof, Lower the existing basement in the footprint of the property and a rear storey extension.

The present document solely refers to the construction of the retro- fit basement in the site.

RP Designs is acting on behalf of the client JV Builders limited.

To be observed by all staff at all times, any deviation from these control procedures must be authorised by the site foreman or safety representative.

General comments

Communication with Other Workers on Site.

All staff will report to the site office for induction on arrival at the site. The site manager will inform staff of any hazards that are present on site.
Staff will inform the site manager of the work to be carried out and how it could affect other trades working on the site.
Where necessary notices will be posted advising of any hazards present during the works.
Where contractor activities cross, the senior person must liaise with the other trades to ensure safe operation.

First Aid

It is the responsibility of the company to ensure adequate First Aid provision for its staff. Adequate means provision of a trained first aider, suitable first aid equipment and/or the provision of an appointed person at the minimum.
A trained First Aider will be a suitable person who has attended an HSE approved course of at least three days duration.
An Appointed Person is a person provided by the employer to take charge of the situation (e.g., to call an ambulance) if a serious injury/illness.
occurs in the absence of a First Aider. The Appointed Person can render emergency First Aid if trained to do so. All staff when inducted will be made aware of the location of the First Aid kit.

Manual Handling

All staff and contractors have been instructed on the potential dangers of manual handling and have received manual handling training.
Equipment provided to reduce manual handling must be used where provided.
Staff and contractors will not lift items of tools or equipment that.
are beyond their capabilities.
Heavy or awkward items will be split into smaller units where possible or dual lifted where this is not possible. It is the responsibility of the site foreman/employer to identify and control manual handling activities.
and control manual handling activities as they occur on site on a day-to-day basis.

Material Handling

All materials required for site will be unloaded to a designated unloading and storage area which will be away from the work area as far as is practicable. This area will be kept tidy to minimise trip hazards. Materials as and when required will be collected from the storage area and transferred to the work area.
All staff will take care when handling materials and will use mechanical aids. wherever possible. When stacking materials particular care must be taken to ensure that the stack is secure, and that the product does not get damaged.

Personal Protective Equipment (PPE).

PPE will be provided as a last form of protection against a hazard. Staff will use the appropriate PPE for the task as identified in the risk assessment.
All site workers will wear Safety boots, Hi Visibility Vests, Hard Hats and protective clothing at all times, other items of PPE such as eye protection, hearing protection and gloves are available to be worn as and when necessary and as determined by the risk assessment.

Preparation & Induction.

A risk assessment will be carried out for all tasks which will be discussed with members of staff and the sub-contractors, any queries or concerns will be raised with the contract manager who will ensure it is dealt with.
Staff and sub-contractors will be inducted onto site in order to understand. the hazards present on site and the tasks that are to take place. Staff will also be advised of other site activities that could impact on their work and be made aware of any liaison that needs to take place between different trades. Staff will follow all site rules and safety procedures.

Staff and Training

The task will be carried out by staff from JV Builders Ltd, all staff are qualified, experienced, receive ongoing training, and hold suitable qualifications. Apprentices are under constant supervision by experienced members of staff. Any sub-contractors appointed by us have been assessed for their ability and suitability to carry out the tasks allocated to them.

Tools and Electrical Equipment

All tools and equipment will be visually inspected on a regular basis, defective or damaged equipment will be removed from service.
Electrical tools will be 110V or battery operated where possible. Sub-contractors will not be allowed to bring on to site any damaged or defective tools,
the site foreman is responsible for ensuring that all tools and equipment. allowed on the site are fit for purpose. Any portable electrical equipment taken on to site must be PAT tested every 3 months when used on construction sites, 6 monthly for heavy use activities and annually for other activities. A risk assessment will determine if inspection periods need to be varied.

Welfare

The principal contractor is responsible for providing adequate washing, toilet, drying and refreshment facilities for staff and sub-contractors,
staff and contractors are responsible for ensuring that such welfare facilities are maintained in a clean and wholesome manner.
This will be your responsibility when you are the principal contractor, it may be necessary occasionally for your company to identify suitable local amenities.

Contractor and Visitor Safety

JV Builders Ltd will liaise with other contractors’ staff on a day-to-day basis and ensure they are aware of the risks present during the works. Staff and contractors will not leave any area of work in a dangerous condition or with risks to themselves, other contractors, tenants, or visitors, all tools and equipment will be cleared to secure storage at the completion of each shift.
Heavy plant, scaffold, ladders, and any other access to height will be made inaccessible.

Digging Out (Excavations)

The area of excavations will first be checked for live services using cat scanners and plans where possible. Any existing services will be protected or moved as required.
The holes will be either dug using an excavator or hand dug by two men. using pick and shovel; the excavations will be dug to the required width and depth according to the plans.
To prevent the potential for subsidence or collapse coffer dams will be used to support the sides where the depth exceeds1.5 metres, a distance of approx 1.2 metres is left in situ and another section is dug out, the intermediate sections are not dug out until the back fill or concrete in the original excavations has been set for 48 hours.

Excavation Safety

On site dig permits may be required, refer to the site manager where applicable. No machines are allowed near the excavations to minimise the risk of collapse, all trenches will be protected from collapse with braced timber shuttering and sandbags, or coffer dams; the site foreman will ensure the safety of each excavation prior to allowing work. Barriers will be erected around all excavations. Access and egress to trenches will be via short timber ladders, all excavations will have a banksman on duty and an evacuation hoist on site whilst the trench is being worked. Occasionally water enters the excavations, and this must be cleared out by hand bailing, severe flooding may require the use of a pump. JV Builders Ltd staff are aware of the dangers of standing water and will take precautions to ensure contaminated water does not get onto the skin or enter the body. Good standards of hygiene will be maintained prior to eating, drinking, and clearing site. A fact sheet on Leptospirosis is provided and will be read by all staff and sub-contractors under our control.
Excavation inspections should be made in accordance with the HSE 'Construction information sheet No 47 (rev1)'.

Ladders

Ladders will only be used as an access to scaffold or for carrying out light work of short duration. Ladders must be tied and or footed.
Ladders must only be set at a ratio of 1 metre out at the base for every 4 metres in height.
All staff and sub-contractors are required to read and understand HSE leaflet INDG402 the Safe use of Ladders & Stepladders.
Ladders will be removed or boarded off at the end of each shift to prevent unauthorised access, damaged or unsuitable ladders will be removed from site immediately.
Ladders must be stored in such a way that they cannot be damaged by other objects or by the elements.
Class 3 ladders or step ladders must not be used on site.

Machine Tools

Machine tools will only be operated by competent persons. Apprentices will be allowed to operate machinery if under the direct supervision of a competent person.
Machine tools will be isolated when not in use and under no circumstances will. they be left unattended. All machine tools will be PAT tested on at least an annual basis and visually inspected on a daily basis by the competent person. Construction sites require that PAT tests are carried out every 3 months. Any tools found to be damaged will be removed from site immediately until a repair or replacement can be affected.

Portland Cement

Portland cement is a light grey powder that poses little immediate hazard. A single short-term exposure to the dry powder is not likely to cause serious harm.
However, exposure of sufficient duration to wet Portland cement can cause serious, potentially irreversible tissue (skin or eye) destruction in the form of chemical (caustic) burns. The same type of tissue destruction can occur if wet or moist areas of the body are exposed for sufficient duration to dry Portland cement.

Dust masks should be worn when exposed to dry cement powder as repeated. exposure could cause chronic bronchitis.
All persons exposed to Portland cement must ensure that they have sufficient protection from the caustic effects of cement.
Staff will wear impervious gloves overalls and wellington boots, eye protection must be used during mixing, working, and laying of Portland cement and its products.

Site Access and Egress

The principal contractor is responsible for providing safe access and egress to the site, JV Builders Ltd staff will ensure safe access and egress is maintained for themselves and other contractors in the area they are working in, good standards of housekeeping will be maintained.
JV Builders Ltd will be responsible for safe access and egress.
when you are the principal contractor. Access routes will be sign posted and barriers will be put in place where necessary.

<u>Proposed Works</u>

The proposed work involves lowering the existing basement level by 600 to 2450mm in some areas and new reinforced retaining wall all the heigh needed to be creating a new liveable basement with proper water proofing system.

1.0 CONSTRUCTION METHOD STATEMENT:

1. Underpinning formation Construction Method.

1.1 The construction method statement provides an approach which will allow the basement to be correctly considered during construction. All the construction is based on drawings by the structural Engineer of the project RP Designs drawings 2198-0 Rev B, 2198-1 Rev B,2198-2 Rev B,2198-3 Rev B, 2198-4 Rev A, 2198-5 Rev E, 2198-6 ,2198-7 ,2198-8 Rev E, 2198-9 Rev E ,2198-10 Rev A,2198-11 Rev A,2198-12 Rev A,2198-13 Rev D and 2198-14 Rev B.
All their drawings must be fully followed without any deviations. If any deviation is needed RP Designs must be inform and follow their advice.

1.2 The construction method statement for 21 Baldwin Gardens/ 43 Leather Lane EC1N has been writing and approved by a Chartered Engineer and is in accordance with the recommendations by CIRIA Report 97, British standards, and Eurocodes. The sequencing has been developed considering guidance from Association of Specialist Underpinning Contractors.

1.3 The Approach followed in this design is sequentially cast mass concrete under pin faced with mas concrete cantilever retaining walls or reinforced concrete cantilever retaining walls. Following fully pins sequence showing in drawings by RP Designs 2198-0.

1.4 The temporary works of the construction pins must be done following.
RP Designs drawings 2198-8 and 2198-9, where the pins are being number to follow sequence of construction any deviation must be consulted prior starting the works with RP Designs.

1.5 As the excavation is deeper than 1.2 m adequate support must be installed timely manner and ahead of excavation as far as possible.

Maximum depth unsupported must be 1.2 m if the soil is self-supported.
In this case the soil must be supported.

2. Enabling works

2.1 The site is to be hoarded with at least ply sheet 2.2 m to prevent unauthorised public access.

2.2 Licenses for skips, conveyors, and Main contractor to be posted on hoarding.

2.3 If Propping of the existing structure is required, is to be fully installed prior to the commencement of any excavation and /or underpinning. Follow temporary works Engineer drawings RP Designs 2198-8 and 2198-9

2.4 Before installing propping mark horizontal level on the existing wall on top of the proposed level of propping on order to take weekly measurement to ensure that the wall is not moving. See Target measure document.

2.5 Exposed brickwork at least one metre above the existing ground floor to verify the condition of the wall.

2.6 Follow sequence proposed on RP Designs drawing 2182-0.

2.8 Walls where the props are to be installed, use ply-faced timber to protect the wall.

2.9 Mark points in existing wall to take horizontal and vertical measurements weekly when underpin is being done and every 5 days meanwhile the construction is being finished after done the basement.

2.10 Installed diagonal props supporting the ply as shown in sequence detail.

2.11 Demolish the existing flooring and take measurements to make sure that the wall is not moving horizontally or and vertically.

2.12 Install horizontal props MABEY S3/System160 as tight as possible to front wall to ensure that the existing wall has a horizontal restrained to equal the horizontal restrained done by the existing floor.

3. Underpinning

3.1 Excavate first base no more than 1000 mm wide and 2000 mm long depending on the type of soil found of if soft spots found back prop with pre-cast lintels. The lintels are to be cut into the soil by 150 mm either side of the pin.
A site stock of minimum 10 lintels to be present to be present for emergency use.
JV Contractors to follow drawings

3.2 Provide propping to local brickwork if wall cannot support itself props to be sacrificial and cast into the retaining wall without interfering with new reinforcement.

3.3 If require provide propping to existing floor where necessary.

3.4 Excavate base without over digging and clear underside of existing footing.

3.5 Local Authority or Building Control to inspect for approval of excavation base.

3.6 Start installing the trench sheeting as deep as possible but a minimum of 600 mm into the ground excavates if required and fill and compact and start forming the trench 1000x2000 using ply boarding for ease of removal to have reinforcement in a continuous manner.
.

3.7 Cast concrete to be premixed and bought in by a reputable concrete supplier as specified by RP Designs specification for underpin base providing the same bearing area as that provided by original wall and fix reinforcement for retaining wall stem and base as specified. Site Supervisor to inspect and sign off works prior to proceed to next stage. Identify in paperwork date, pin number and time. To be available by Engineer and party wall Surveyors if required.

3.8 Concrete supplier to supply two cubes of concrete and store for testing. Test at 14 and 28 days and provide results to client and design team as they became available.

3.9 Horizontal temporary prop to wall to be inserted. Alternative cast base against soil. According to existing boreholes shows that depth where the underpin is done a Soil made of sand / Gravel will be found. But this must be confirmed in situ. If conditions change at the time of construction RP Designs Associates must be informed immediately.

3.10 Place shuttering and pour concrete for retaining wall Stop a minimum of 75 mm from underside of existing footing. Take two cubes of concrete and submit for testing.

3.11 Rammed in dry pack between underpinning and existing masonry (24 hours after pouring the concrete pin the gap shall be filled using a dry pack mortar).

3.12 Using light hand tools trim back existing concrete footing and masonry corbel on internal face. Avoiding excessive vibration.2 No sheets of 1200 gauge polythene required between underpin and retaining wall if required.

3.13 Start shuttering for wall pin and pour concrete as specified.

3.14 Excavation for the next numbered sections of underpinning shall not commence until at least 24 hours after drypacking.

3.15 Follow sequence as shown in drawing by RP Designs 2198-0 Rev B.

3.16 Excavation must be always covered to avoid rainwater when no work is being done in the trench. Edge protection according to CDM regulations must be provided. Follow CDM Coordinator advice.

3.17 If ground water is found Contractor must inform the main Engineer at once. Water must be pump out as soon as possible. The area is low risk of for surface water flooding.
<u>The dewatering system to be used will stop any loss of fines from beneath the party wall and the AO's property /foundations.</u>
<u>**Provide at least two sump water pump before starting construction.**</u>

3.18 <u>**Any horizontally or vertically movement in the pin wall must be inform at once to the main Engineer RP Designs and party wall surveyors.**</u>

3.19 Temporary works must be maintained after finishing the underpinning and installing the new and/or existing ground floor level with the recommendations by 3.21 as shown by Drawings. After the concrete as gained enough strength and after the ground floor is reinstated and strapped to wall,
high level prop can be reduced to a lower half level where the retaining wall has been cast to build the bearing concrete basement slab.

3.20 Residual of the excavation must be disposed of a proper manner.
Probably using conveyor belt system, which will discharge directly into road licenced vehicles in Baldwin Gardens in front of the site. Parking suspensions

will be required.

3.21 If internal bearing walls need to be underpinned, the wall above together with the joists must be supported by temporary needles.

3.22 Needles are to be spaced to prevent the brickwork above “saw toothing”. Where brickwork is good, needles must be placed at a maximum of 1100 mm centres. Lighter needles or strong boys should be placed at tighter centres under door thresholds.

3.23 Props are to be placed on proper sleepers on firm ground or, if necessary, temporary footings will be cast.

3.24 Adhere to drawings 2198-8 Rev D and 2198-9 Rev D for sequence of works .

4.0 Approval.

4.1 Building control officer/approved inspector to inspect every pin base and reinforcement prior to casting concrete.

4.2 LV Builders   to keep a register of inspection and casting dates for all pins. To be available for inspection by Engineer and party wall surveyors.

4.3 One month after work completed JV Builders Ltd is to contact adjacent party wall surveyor to attend site and complete final condition survey and sign off works.

4.4 Every 2 weeks or earlier RP Designs to inspect pins and work done to the progress of the construction.

4.5 the Temporary Works Engineer (RP Designs) will visit site to inspect and approve in writing the Temporary Works installation at the critical stages e.g., loading/unloading stages, etc., and must provide a copy of these written approvals in Contractor’s site progress reports which are to be issued to the party wall surveyors monthly or on request.

5. Other items

5.1 Health and safety

JV Builders Ltd to appoint a CDM Co-ordinator and if no must inform the Health and Safety Executive of the works by filling in and submitting an F10 form prior to starting work on site.

5.2 Setting out.

A) JV Builders Ltd will check or agree the existing dimensions shown on the drawings and must give written notice to the Architect if he is not satisfied with their accuracy.

B) JV Builders Ltd is responsible for all necessary setting out and datum lines and levels to enable the works to be set out to the requirements and accuracies of this specification.

C) Unless otherwise indicated on the drawings the setting out dimensions and levels of the finished works shall be within the maximum tolerances given below.

Description	Maximum Tolerance
All dimensions of 3m and over	+/- 5mm
All dimensions less than 3m	+/- 3mm

5.3 JV Builders Ltd must always follow Health and safety procedures.
Any incident in situ must be recorded, when possible, to do so.

5.4 Contractor to keep record of any deviation and inform RP Designs of any issue.

5.5 It is recommended to have progress meetings in a weekly basis with all involved in the construction and Design team.

2.0 Conclusion

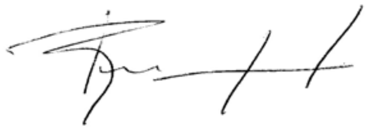
The proposed works when executed correctly in accordance with this structural methodology, management plans as well as relevant plans and specifications will not pose any significant threat to the structural stability of adjoining properties.

The above method is similar to that frequently employed for the formation of basements in highly density urban environments and is entirely appropriate for the proposed works.

Prepared By

Rommel Naranjo Pg (D) and

Frank Bartal BSc MIStructE FB..



On Behalf of RP Designs
61 Barnes wallis Court Barnhill Road
HA9 9 DW
02089046856
0793145281

RP DESIGNS
21 BALDWIN GARDENS/43 LEATHER LANE
LONDON
EC1 7UY

DRAWINGS FROM RP DESIGNS LATEST REVISION TO BE FOLLOWED AND TO BE ADHERED TO.
ANY DEVIATION OF THE DRAWINGS MUST BE INFORMED IMMEDIATELY TO RP DESIGNS.
DRAWINGS:

2198-1 Rev B.
2198-5 Rev E.
2198-8 Rev F.
2198-9 Rev E.
2198-10 Rev A.
2198-11 Rev A.
2198-12 Rev A.
2198-13 Rev D.
2198-14 Rev B.

I , Mr Jasbinder Virdi on Behalf of JV Builders limited we will adhere to the above drawings by RP Designs . If any deviation of the above drawings would happen We will inform RP Designs and The party walls surveyors and agree among the changes if needed.

- J.S. VIRDI

Signed by Mr Virdi on behalf of JV Builders.

Signed by Mr Virdi on behalf of JV Builders.

RP DESIGNS
21 BALDWIN GARDENS/43 LEATHER LANE
LONDON
EC1 7UY

COMMENTS:

APPENDIX 3

RSA GEOTECHNICS LTD

geotechnical and environmental consultants

ground&water

Ground Movement Assessment

21 Baldwin Gardens & 43 Leather Lane, Holburn, London EC1N 7TJ

On behalf of RSA Geotechnics Limited

Report Reference: GWPR5984/GMA/April 2024			Status: Final
Issue	Prepared By	Checked By	Checked & Verified By
V1.03			
	Rob Terrell BSc (Hons) MSc FGS Senior Engineer	Miltiadis Mellios MSc(Eng) GMICE FGS MIEnvSc Principal Engineer	Francis Williams MGeol (Hons) FGS CEnv CGeol Director

Site Investigations | Environmental Consultants | Geotechnical Engineers

2 The Long Barn, Norton Farm, Selborne Road, Alton, Hampshire GU34 3NB
0333 600 1221 enquiries@groundandwater.co.uk **groundandwater.co.uk**

Registered Office: Kineton House, 31 Horse Fair, Banbury, Oxfordshire OX16 0AE Registered in England No. 07032001

1.0 INTRODUCTION

1.1 General

Ground and Water Limited were instructed by RSA Geotechnics Limited Engineers on the 17th October 2022 to conduct a standalone Ground Movement Assessment on the site referred to as 21 Baldwin Gardens & 43 Leather Lane, Holburn, London EC1N 7TJ. The scope of the investigation was detailed within the Ground and Water Limited fee proposal (reference: QU-0314).

1.2 Aims of the Investigation

The aim of the investigation was understood to be to supply the client and their designers with information regarding the ground movements throughout and following the basement development, as well as assessing nearby assets. This report aims to assist them in preparing an appropriate scheme for development.

A Land Stability Report For Basement Impact Assessment, produced by RSA Geotechnics Limited in February 2024, was used for this assessment. Ground and Water Limited are not to be held accountable for shortcomings within this report.

1.3 Conditions and Limitations

This report has been prepared based on the terms, conditions and limitations outlined within Appendix A.

1.4 Technical Glossary

Generic technical terms can be viewed within the glossary provided within Appendix B.

2.0 SITE SETTING

2.1 Site Location

The site is located north-west of the road junction between Leather Lane and Baldwin's Gardens about 350m west of Farringdon Station in the London Borough of Camden at National Grid Reference TQ 312 818.

2.2 Site Description

The property at 21 Baldwin's Gardens/43 Leather Lane, Camden, EC1N 7TJ comprises a four-storey terraced building with a partial basement. The site extended over an irregular rectangular shaped area ~16.00m long from east to west and varying in width from ~5.00 – 6.50m. The current basement layout is shown on drawing number 16336SI/1.

The former retail space at ground floor and basement level were unoccupied at the time of the site investigation fieldwork undertaken on 6 September 2023. The headroom in the existing basement was ~1.90 – 2.00m and was supported by brickwork retaining walls.

In places the basement slab (at about 17.60m AOD) had previously been removed by the building contractor to expose the strip foundations of the perimeter walls bearing upon the Hackney Gravel Member. Details of the existing foundations are shown on sections on RP Designs drawing number 2198-7. The foundations took the form of 300mm thick corbelled brickwork with outstands extending 220 – 250mm from the internal face of the basement walls. The undersides of the brickwork foundations were typically ~0.50m below the basement floor between ~16.90 – 17.10m AOD, ~2.50 – 2.70m below the external ground level on Leather Lane.

2.3 Nearby Assets and Subterranean Developments

The surrounding area consists of similar properties, comprising four and five-storey properties built of traditional loadbearing masonry construction. Two nearby properties within 14m of the site have single level basements/lower ground floors.

2.4 Proposed Development

The proposed development involves extending the first, second and third floors towards the rear (above the existing ground floor), adding a fourth floor and enlarging the basement towards the rear so that it extends beneath the whole building footprint. The proposed basement finished floor level at 17.33m AOD will be 2.662m below the ground-floor level. The existing and proposed basement perimeter walls are to be underpinned to a depth 1.40m below the proposed basement floor level at 15.933m AOD. The basement will be designed to support the existing primary structure above with RC walls built in sequential underpins. Alterations to the superstructure fall outside the scope of this report. One lift of underpinning is to be undertaken throughout the construction sequence.

2.5 Levels

To aid the assessment, a summary of the existing and proposed elevations of various levels across the site can be seen in the following table.

Summary of On-Site Levels and Changes in Levels		
Level	**Existing (m AOD)**	**Proposed (m AOD)**
Ground Floor	~19.500	~19.500
Top of Basement Slab	17.600	17.333
Base of Basement Foundation	16.900 – 17.100	15.933

Additionally, the following table highlights changes in levels across various sections of the site.

Summary of On-Site Levels and Changes in Levels	
Level	**Lowering (m)**
Lowering from the ground floor to basement formation level	4.062
Lowering from the lower ground floor to basement formation level	0.967 – 1.167

3.0 SUMMARY OF PREVIOUS GROUND INVESTIGATION

3.1 Site Works

RGS Geotechnics Limited undertook site works on 6th September 2023 which comprised the drilling of 2No. Window Sampler Boreholes. WS1 was drilled to 0.60m bgl (from 17.31m AOD to 16.71m AOD), whereas WS1A was drilled to 1.30m bgl (from 17.31m AOD to 16.01m AOD), where both boreholes refused on dense granular soils. A groundwater monitoring standpipe was installed within WS2, with a response zone between 0.30 – 1.30m bgl/17.01 – 16.01m AOD.

3.2 Encountered Ground Conditions

The ground conditions encountered within the trial holes constructed on the site did generally conform to that anticipated from examination of the geology map. A 0.30 – 0.80m capping of Made Ground was noted to overlie the granular Hackney Gravel Member. The London Clay Formation was not encountered; however, the London Clay Formation was inferred to be encountered from 12.50m AOD to ~-0.50m AOD, based on nearby BGS boreholes. A summary of the ground conditions encountered has been provided in the following table.

Summary of On-Site Levels and Changes in Levels		
Level	**Top Depth (m AOD)**	**Base Depth (m AOD)**
Made Ground	19.50	17.01 – 16.51
Hackney Gravel Member	17.01 – 16.51	12.50 – 8.50
London Clay Formation*	12.50 – 8.50	~-0.50
***Based upon nearest BGS boreholes**		

3.3 Groundwater Conditions

No groundwater was encountered during drilling; however, groundwater strikes may have been obscured by the drilling process. A groundwater monitoring reading was taken on 06/09/2023, within the 1.30m deep standpipe installed within WS1A, where no groundwater was recorded; it should be noted that groundwater levels may increase at other times of the year, specifically after periods of rainfall associated with winter.

3.4 Geotechnical Laboratory Testing

A particle size distribution test, undertaken on a sample of the Hackney Gravel Member found 12% silt and clay, 31% sand and 57% gravel.

4.0 GEOTECHNICAL ANALYSIS

4.1 Settlement and Heave Analysis

This section of the report states suitable geotechnical parameters for the soils encountered as well as analysis the bearing capacity of the soils. A settlement/heave analysis was also undertaken following the construction of the proposed development using Pdisp from Oasys.

4.1.1 Geotechnical Parameters for Modelling

Following a literature review from well documented publications, the short-term and long-term Young's Modulus (E short term and E') has been produced.

Geological Strata

A summary of the ground conditions encountered has been provided in the following table.

Summary of On-Site Levels and Changes in Levels		
Level	**Top Depth (m AOD)**	**Base Depth (m AOD)**
Made Ground	19.50	17.01 – 16.51
Hackney Gravel Member	17.01 – 16.51	12.50 – 8.50
London Clay Formation*	12.50 – 8.50	~-0.50
***Based upon nearest BGS boreholes**		

Made Ground

Made Ground was modelled between the ground level (19.50m AOD) and 16.51m AOD.

A short-term and long-term Young Modulus (Eu and E') of 10MPa was suitable and on the conservative side, regarding Made Ground encountered on site. For Made Ground, it was considered suitable for E' and Eu to be equal, given that these soils are more permeable and to limit the level of anticipated Young Modulus at a representative value.

A Poisson's ratio of 0.45 was given to these soils based on their variable nature for short term conditions, decreasing to 0.20 for long term conditions.

Hackney Gravel Member

The Hackney Gravel Member was modelled between 16.51m AOD to 8.50m AOD.

As no SPTs were undertaken within the Hackney Gravel Member, an SPT N Value of 8 was used for the modelling. This may be considered conservative given drilling refused on the soils thus they must be dense; however, the potential for groundwater flushing out fine material from these soils and weakening them means that a SPT N Value of 8 was considered suitable and realistically conservative.

Given the granular soils are permeable, no significant long-term draining of the soil was anticipated to occur and therefore the short and long-term modulus was considered sensible to remain the same. The widely accepted relationship between recorded SPTs within granular soils and E values of 2000* SPT "N" values was used for this consideration. The value was cross-referenced with representative published data (Obrzud & Truty 2012), showing a range of between 50 – 320MPa for the Young Modulus for dense sands and gravels. This also aligns with the drained modulus (30 – 160MPa) for

River Terrace Gravels included in *"Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200".*

For the Hackney Gravel Member, it was considered suitable for E' and Eu to be equal, given that these soils are more permeable and to limit the level of anticipated Young Modulus at a representative value.

A Poisson's ratio of 0.30 was given to these soils based on their granular nature for short term conditions, decreasing to 0.20 for long term conditions.

London Clay Formation

Cohesive soils of the London Clay Formation were inferred between 8.50 – 0.50m AOD based on BGS borehole records.

A design line was taken from "*Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200*". The equation was undrained shear strength = (depth into the LCF x 8) + 50.

The relationship between Eu and Cu is generally dependent on strain levels. For small strains, a ratio of 400 can be adopted based on well documented publications. This is also reflected for the London Clay Formation, after extensive research, within graphs depicting strains and Eu/Cu ratios included in *"Burland JB, Standing, JR, and Jardine, FM (2001) Building response to tunnelling, case studies from construction of the Jubilee Line Extension CIRIA Special Publication 200".*

A ratio of E' to Eu of ~0.75 was considered a sensible approach for this stage in the design, for cohesive soils of the London Clay Formation.

A Poisson's ratio of 0.45 was given to these soils based on their cohesive nature for short term conditions, decreasing to 0.20 for long term conditions.

Summary

A summary of the above has been provided in the following table.

Summary of Geotechnical Parameters								
Geological Strata	Depth (m AOD)		Short-term Young's Modulus (Eu short term) (kPa)		Long-term, Young's Modulus (E' long term) (kPa)		Poisson's Ratio	
	Top	Base	Top	Base	Top	Base	ST	LT
MG	19.50	16.51	10,000.00	10,000.00	10,000.00	10,000.00	0.45	0.20
HGM	16.51	8.50	16,000.00	16,000.00	16,000.00	16,000.00	0.30	0.20
LCF	8.50	-0.50	20,000.00	48,800.00	15,000.00	36,600.00	0.45	0.20

4.1.2 Bearing Capacity Assessment

The bearing capacity at the founding depth was stated within the Land Stability Report For Basement Impact Assessment, produced by RSA Geotechnics Limited in February 2024. It was stated within this report that proposed loadings will be below the bearing capacity.

It should be noted that no in-situ strength testing was undertaken during the site investigation. In-situ strength testing is recommended to be undertaken to confirm the strength/density of the soils underlying the site, and thus confirming geotechnical parameters and bearing capacities.

4.1.3 Settlement/Heave Analysis

Analyses of vertical ground movements, using the Mindlin analysis method within Pdisp software, was undertaken to assess the potential movements resulting from changes of net vertical pressure changes. Geotechnical parameters noted in the previous section of this report were used for the model. A rigid boundary at depth was considered at -0.50m AOD, for calculation purposes.

The borehole was excavated from 19.50m AOD, and therefore that is the surface of the model. Displacement lines for the surrounding walls were placed at 19.50m AOD.

Five representative stages of construction, in terms of the net change in vertical pressure, have been modelled. These were considered to adequately approximate the movements rising from the basement construction.

- **Stage 1:** Excavation of the retaining wall voids, with short-term conditions;
- **Stage 2:** Loads associated with the construction of the retaining walls, with short-term conditions;
- **Stage 3:** Stage 2 loads, as well as loads associated with the mass excavation of the basement footprint, with short-term conditions;
- **Stage 4:** Stage 3 loads/load removals, as well as loads associated with the construction of the basement slab, with short term conditions. The basement is now fully constructed;
- **Stage 5:** Stage 4 previous loads/load removals, for long-term conditions.

As the proposed development did not comprise the demolition of the existing building, the existing loads of the property were not anticipated to change throughout the development.

The overburden pressure release following the excavation and removal of soils was based on a specific weight of soil of 19kN/m. Overburden pressure releases are shown below:

- The existing ground floor was being excavated from 19.50m AOD to 15.933m AOD, equating to a change of 4.062m, which calculates to an overburden pressure release of 77.178kN/m^2.
- The existing lower ground floor was being excavated from 17.100m AOD to 15.933m AOD, equating to a change of 1.167m, which calculates to an overburden pressure release of 22.173kN/m^2.

The final loads were based on the drawing below provided within the Land Stability Report For Basement Impact Assessment, produced by RSA Geotechnics Limited in February 2024. All loads were to be placed at the proposed basement formation level (15.933m AOD). Loads were proposed to be between 120 – 150kPa, spread across a 1.00m wide footing. For conservatism, the highest load (150.00kPa) was modelled, in order to increase settlement for the ground movement analysis.

A summary of all applied loads, at each stage/model, can be viewed in the following table.

Summary of Net Bearing Pressure Changes for PDisp Analysis					
Description	Level (m AOD)	Applied Load (+ive)/ Load Removal (-ive) (kN/m^2)			
		Stage 1	Stage 2	Stage 3	Stage 4 & 5
Underpinning from EGFL to PBFL	15.933	-77.178			
Underpinning from ELGFL to PBFL	15.933	-22.173			
Construction (Stage 1 -3) and Final Loading (Stage 4 and 5) of Retaining Walls	15.933		30.00	30.00	150.00
Mass Excavation from ELGFL to PBFL	15.933			-77.178	-77.178
Mass Excavation from ELGFL to PBFL	15.933			-22.173	-22.173
Construction of Basement Slab	15.933				10.00
EGFL = Existing Ground Floor Level **ELGFL = Existing Lower Ground Floor Level** **PBFL = Proposed Basement Floor Level** **PLGFL = Proposed Lower Ground Floor Level**					

The method stated above was considered to comprise a comprehensive and reasonably conservative approach, in order to calculate a realistic maximum potential heave and settlement.

A tabulated summary concluding the amount of soil displacement shown within the contour plots can be viewed below. It should be noted that the soil displacement between models are not cumulative values; therefore, the amount of soil displacement between models should not be added together as each model shows each construction stage individually.

Settlement/Heave Analysis	
Model	Soil Displacement
	Proposed Basement Formation Level (15.933m AOD)
Model 1	Maximum heave of 9.32mm below the retaining wall voids
Model 2	Maximum settlement of 3.656mm below the retaining walls
Model 3	Maximum heave of 12.10mm in the centre of the rear section of the basement. Maximum settlement of 2.54mm below the western retaining wall.
Model 4	Maximum heave of 2.76mm in the centre of the rear section of the basement. Maximum settlement of 17.60mm below the western retaining wall.
Model 5	Maximum heave of 1.32mm in the centre of the rear section of the basement. Maximum settlement of 23.20mm below the western retaining wall.
Please note that the above figures should not be added together (or be superimposed) and that they represent anticipated movements at different accumulated stages of construction, in order to approach and test all expected combinations of loading regimes (models).	

The maximum amount of heave throughout construction was noted following the mass excavation of the basement voids (Model 3). Once constructed, the maximum amount of heave increased for long term conditions (Model 5), when compared to short term conditions (Model 4); therefore, the highest risk of movement will likely occur during the construction of the basement and later through long-term heave of the constructed basement.

The inputs and outputs of this analysis can be provided on request.

4.2 Ground Movement Analysis

According to CIRIA C760 estimating ground movements in the vicinity of excavations is very complex due to the variety of factors involved. It is also mentioned that ground movements around the excavation can be controlled and minimised by adopting specific measures, which are discussed at the end of this section. The ground movement assessment and resulting damage assessment can be viewed within this section.

Full inputs and outputs of this analysis can be provided by request.

4.2.1 Models

One model was created to assess the horizontal and vertical soil displacement and the resulting damage to surrounding assets from the loading of the new development, the heave from overburden pressure release, excavations and underpinning.

Displacement lines were imported from Pdisp to ensure consistency between both the Pdisp and Xdisp models. Along imported displacement lines, the walls of the corresponding buildings were modelled. An individual wall/face approach was carried out for each property. The height of each wall was added manually whereas default structural properties were deemed appropriate. The segments for each line were combined in order to properly assess each wall/structural feature. The structural features were given approximate heights for the damage categorisation. All walls of these properties were modelled at existing ground floor level (19.50m AOD along the z-axis). For the roads and pavements surrounding the site, walls were not suitable to be modelled; and therefore, the amount of soil displacement and deflection was assessed using the displacement lines and contour plots.

The ensure the model was realistic and conservative, whilst not overcomplicated, one excavation was modelled based on the maximum excavation depth of 4.062m. This was modelled between the ground floor level (19.500m AOD) and the proposed basement founding level (15.933m AOD).

As there were no official ground movement curves from CIRIA C760 for soil displacement from underpinned foundations, user generated curves were created for both excavations, where a maximum of 5mm horizontal and vertical soil displacement was predicted at the face of the excavation. For horizontal ground movement, the distance to 0.00mm ground movement was 4 x depth of the excavation. For vertical ground movement, the distance to 0.00mm ground movement was 2 x depth of the excavation.

To ensure that the effect of the loading of the proposed foundations, Pdisp Model 4 vertical displacements were imported into the model. To ensure the model was conservative in estimating the settlement away from the basement, only positive loads (settlement inducing) were imported from the Pdisp model.

Once the analysis was undertaken, Xdisp segments areas of hogging, sagging and negligible movement along each wall, and gave each segment a category of damage; however, as the wall was thought to act as one structurally, these segments were combined and a damage category for the wall as a whole was given. For any walls where the segments were not possible to be combined, it was considered that the software was not sensitive to capture the damage, therefore instead of 'negligible', the un-combined results were taken into account, for conservatism.

4.2.2 Analyses

The following parameters have been used to inform all assessments.

- Given limitations of the software, a conservative assessment was undertaken assuming that all properties and levels were relative to the ground level.
- The method of basement construction is understood to be traditional underpinning;
- A high wall stiffness has been assumed;
- In the permanent case the wall should be propped at high level;

In terms of damage assessment, the widely accepted Burland et al, 1977 method was used for combined segments along structural features. It was considered that the construction design from the structural engineer will account for any damage resulting from predicted soil displacement on the actual building, as advised in this report. Any walls noted to extend directly away from the basement were noted to be 0.001m away from the basement at their closest point(s), in order to avoid computational twists.

4.2.3 Discussion of Results

Based on the modelling above, it was considered that all walls surrounding the proposed development will result in negligible (category 0) to very slight (category 1) damage, based on a conservative and realistic approach. This is an acceptable amount of damage based on LBC Guidance.

The following was also not accounted for within the modelling, which may lessen ground movement and the resulting structural damage to walls:

- **It is likely that adjoining front and rear walls of adjoining buildings may act structurally as one wall, where connected, which may lessen structural damage.**
- **It is known that two basements are existing within close proximity to the proposed basement. These sub-surface structures may offer rigidity to the surrounding ground and reduce ground movement resulting from the excavation and construction of the basement, lessening structural damage on overlying walls.**

4.2.4 Assessment of Roads and Utilities

As the underpinned lower ground floor is adjacent to the highway/footpath of Leather Lane, it is likely that soil displacement will occur affecting the highway/footpath; however, given the length of the roads, the minimal deflection expected was not anticipated to cause damage. Monitoring is recommended as good practice.

4.2.5 Additional Comments

It should be noted that using stiff clay data in some movements in this assessment could be argued to have produced less conservative results. This however is countered by the following, which make the results more conservative.

- The size of the developments used to provide the case histories for C760 are significantly greater than the scale of works proposed. In practice the range of ground movements (relative to the excavation depth and the building dimensions) is therefore likely to be much smaller for this development.

Should the following precautions be included in the Construction Method Statement, as well as best practice and good construction techniques are utilised by a reputable contractor, then ground movements due to underpinning will be limited. In the permanent case the wall will/should always be propped at high level. **It is recommended that monitoring is undertaken as good practice.**

It will be important that the building contractor is closely supervised and is experienced in this type of construction. It will be critical to prevent exposed faces from collapse or significant ground loss into the new excavation and temporary face support should be maintained where practicable. The adequacy of temporary support will be critical in limiting ground movements. A number of factors will assist in limiting ground movements:

- Most ground movement will occur during excavation and construction so the adequacy of temporary support will be critical in limiting ground movements;
- The speed of propping and support is key to limiting ground movements;
- Good workmanship will contribute to minimising ground movements;
- The assessment assumes the wall is in competent clay;
- Larger movements will be expected where soft soils are encountered at, above and below formation;
- The adequacy of temporary support will be critical in limiting ground movements;

CIRIA C760 advises that ground movements are influenced by the quality of workmanship. The party wall act will apply to this development and will re-enforce good workmanship. The act provides an effective mechanism for ensuring that structural integrity of the neighbouring property is maintained throughout the construction phase. Examples of this can be viewed below.

- Ensuring that adequate propping is in place at all times during construction;
- Minimise deterioration of the central soil mass by the use of blinding/covering with a waterproof membrane;
- Installation of the first (stiff) support quickly and early in the construction sequence for each excavation panel;
- Control dewatering to minimise fines removal and drawdown;
- Avoid overbreak.
- Avoid leaving ground unsupported.

4.3 Structural Monitoring

As stated within the previous section, it is recommended that structural monitoring is undertaken to ensure the movements remain within acceptable limits and to enable mitigation to be effectively implemented in the event of trigger values for movement being exceeded. Trigger values should be adopted based on this assessment and the comments by the structural engineers.

The final extent of the structural monitoring will be a matter for the agreement with the neighbours as part of the Party Wall Agreements.

Monitoring positions should be located at the front and rear elevations of the neighbouring properties. The targets should be set at both a low and high level and a minimum of four targets should be installed at each elevation (two targets near party wall and two targets at the far end of the elevation). Precise survey equipment should be used to record all vertical and horizontal components of movement (in three perpendicular dimensions) to a minimum accuracy of 1mm.

Before any excavation or construction works commence, monitoring over a period of at least a month should be undertaken in order to establish a baseline situation and record any seasonal movement trends that may also affect measurements during the development.

During all underpinning works and basement excavation works, monitoring should be undertaken daily at the start and end of every work shift. At other times, monitoring should be undertaken weekly to cover a period prior to commencement of any works and ceasing after completion of the works, by agreement of all interested parties.

The cumulative movements in any direction of any monitoring point are to be compared with the predicted movements at any stage, using the following table. The proposed contingency actions are also noted.

Summary of Monitoring Criteria and Trigger Values		
Ground Movement	**Status**	**Assessment**
Total differential movement <3mm in any direction	Green	None
Total differential movement >=3mm and <5mm in any direction	Amber	• Notify Structural Engineer and Party Wall Surveyor
Total differential movement >=5mm in any direction	Red	• Immediately Cease any excavation work • Immediately notify Structural Engineer and Party Wall Surveyor • Provide additional support/backfill excavation pending review by structural engineer.

The trigger values noted above were concluded based on a manual sensitivity analysis into ground movements and resulting vertical deflection and horizontal strain, from various excavation depths, for site specific walls.

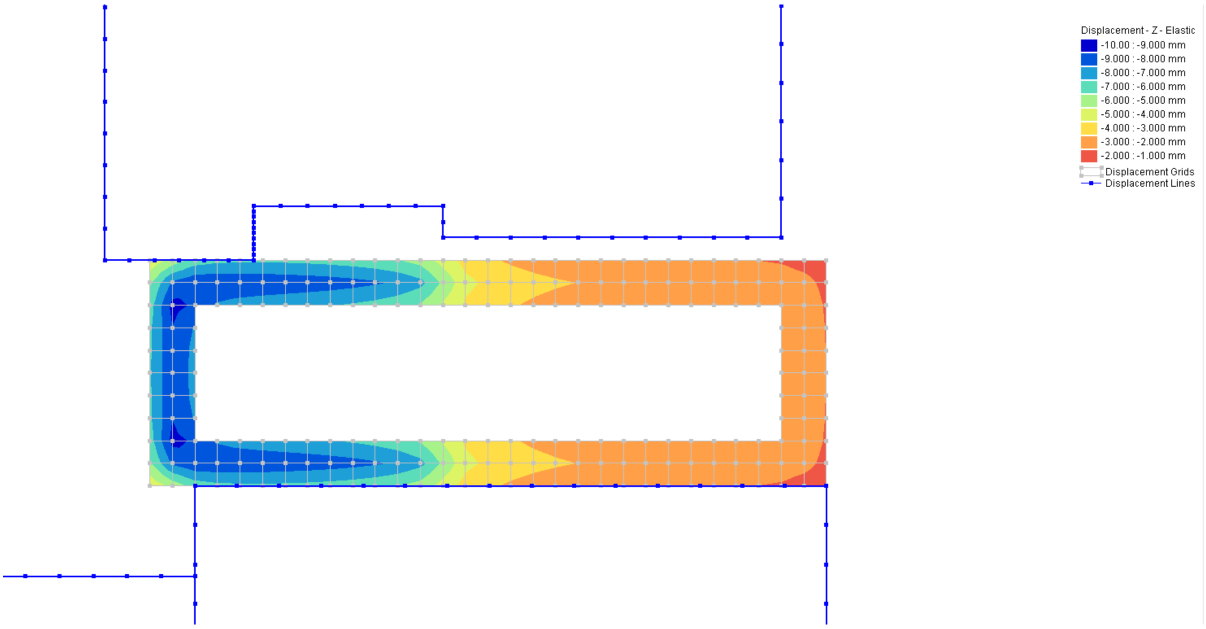
geotechnical and environmental consultants

ground&water

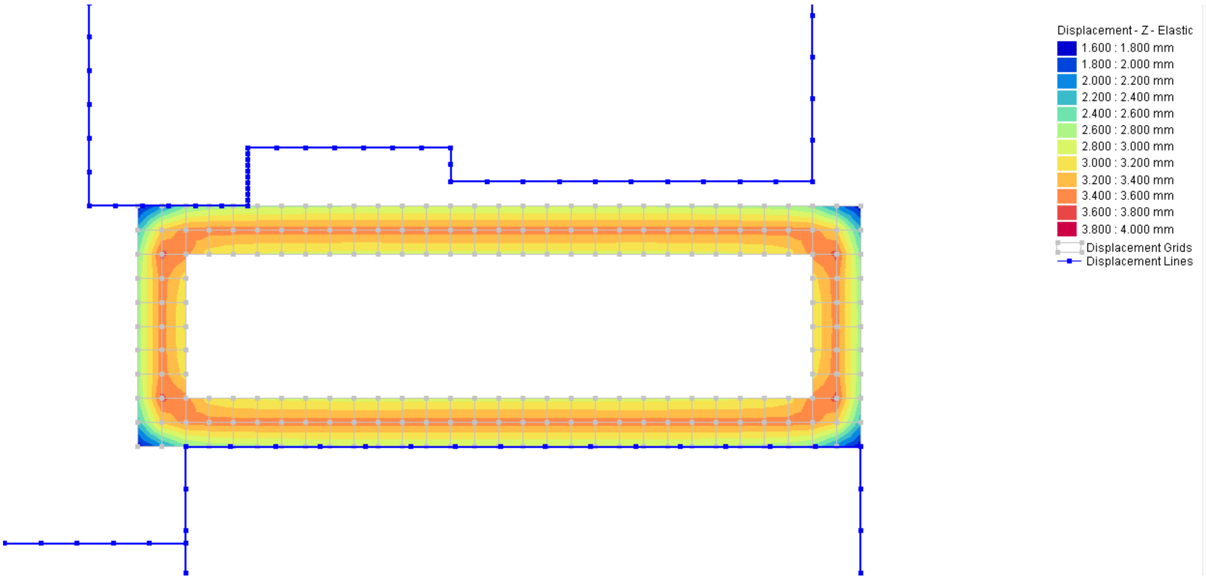
FIGURES AND GRAPHS

SETTLEMENT AND HEAVE ASSESSMENT

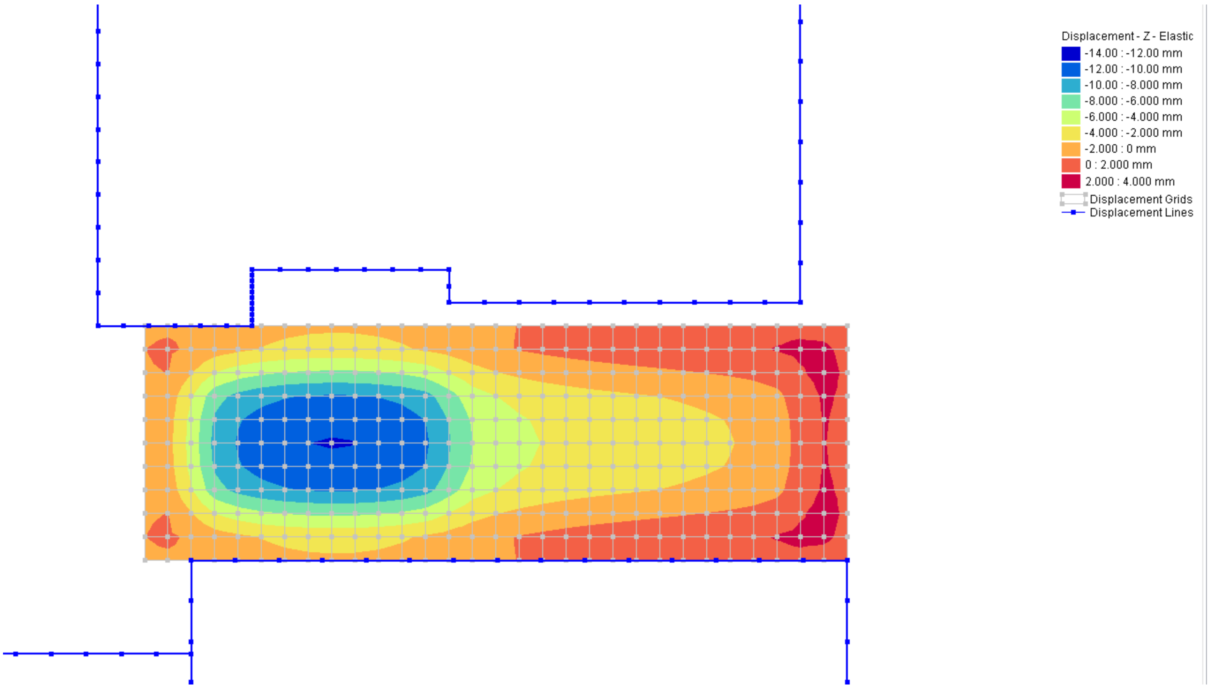
Model 1 Output



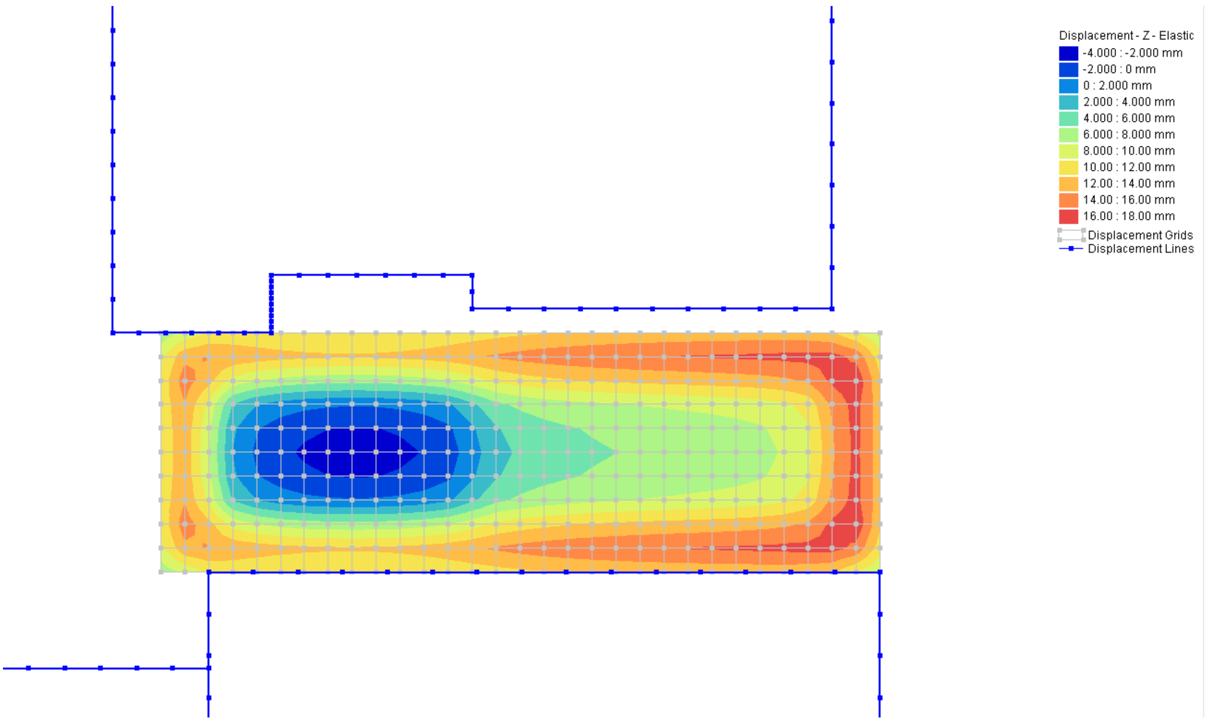
Model 2 Output



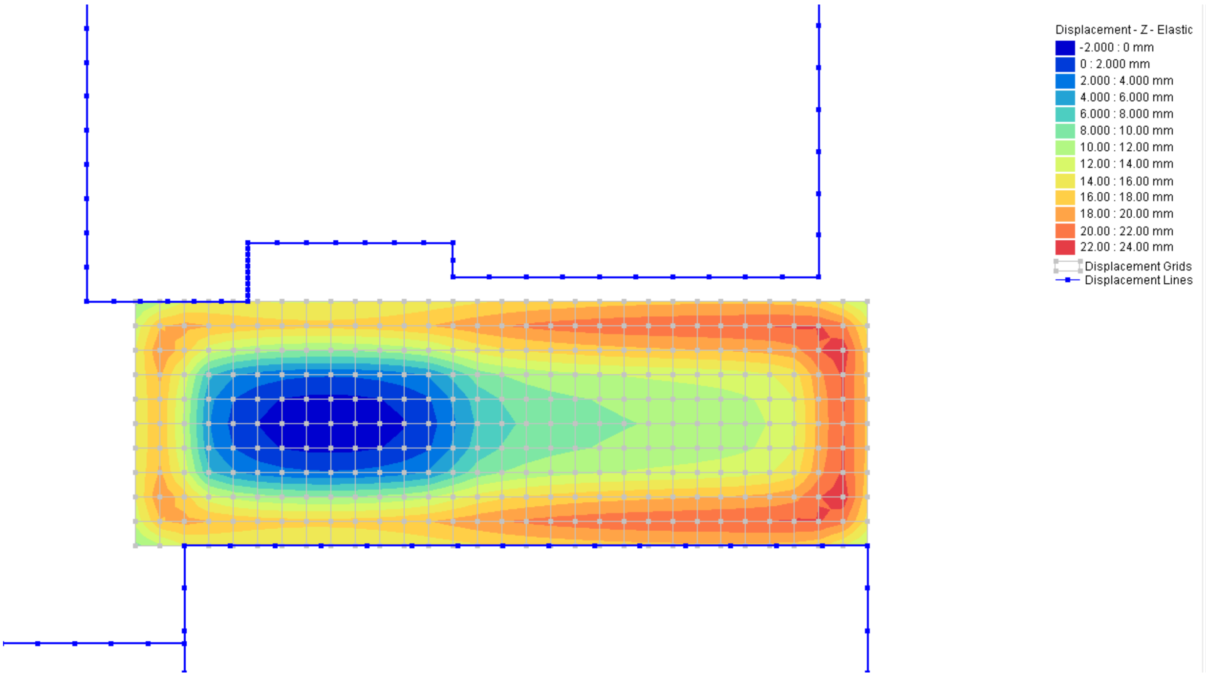
Model 3 Output



Model 4 Output

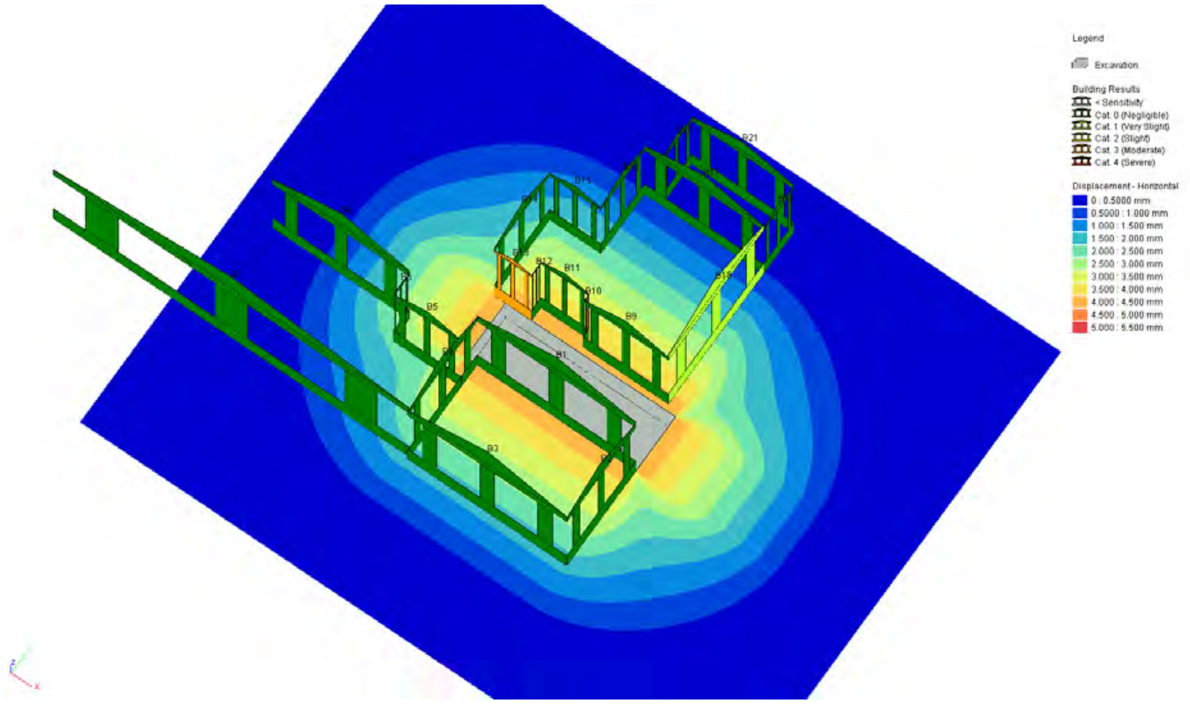


Model 5 Output

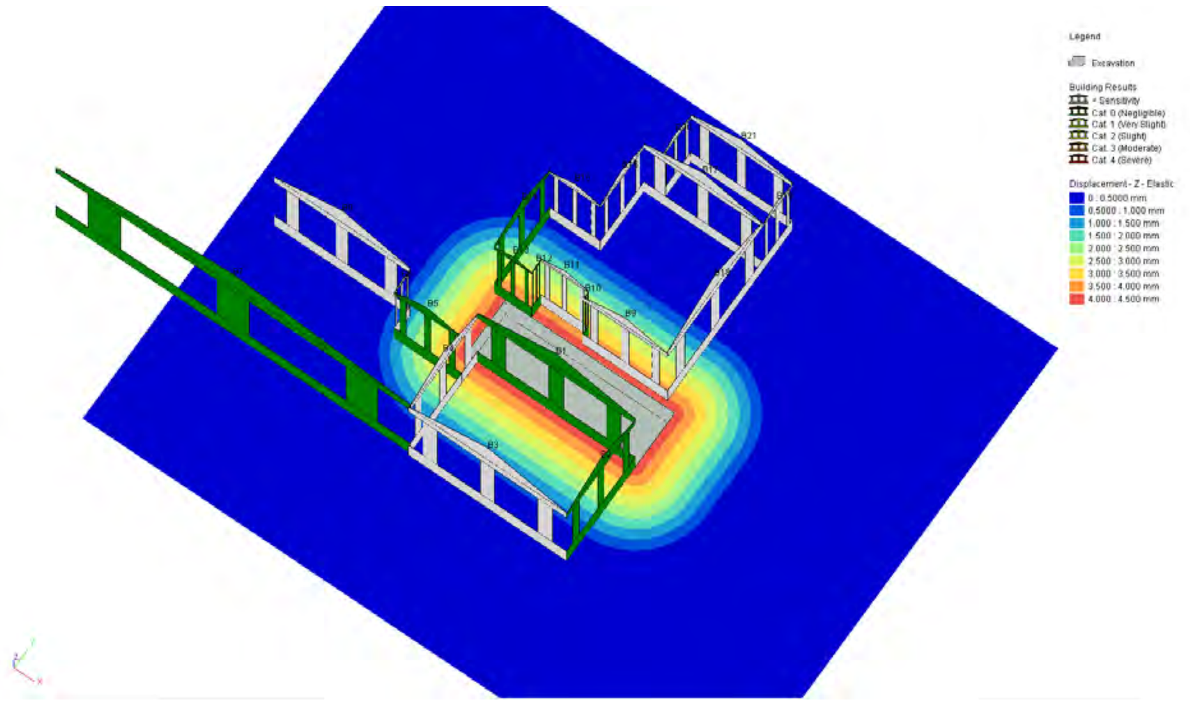


GROUND MOVEMENT ASSESSMENT

Horizontal Movement Contours and Non-Combined Segment Damage Assessment



Vertical Movement Contours and Combined Segment Damage Assessment



APPENDIX A: Conditions and Limitations

2 The Long Barn, Norton Farm, Selborne Road, Alton, Hampshire GU34 3NB
0333 600 1221 enquiries@groundandwater.co.uk **groundandwater.co.uk**

The ground is a product of continuing natural and artificial processes. As a result, the ground will exhibit a variety of characteristics that vary from place to place across a site, and also with time. Whilst a ground investigation will mitigate to a greater or lesser degree against the resulting risk from variation, the risks cannot be eliminated.

The report has been prepared on the basis of information, data and materials which were available at the time of writing. Accordingly, any conclusions, opinions or judgements made in the report should not be regarded as definitive or relied upon to the exclusion of other information, opinions and judgements.

The investigation, interpretations, and recommendations given in this report were prepared for the sole benefit of the client in accordance with their brief; as such these do not necessarily address all aspects of ground behaviour at the site. No liability is accepted for any reliance placed on it by others unless specifically agreed in writing.

Any decisions made by you, or by any organisation, agency or person who has read, received or been provided with information contained in the report ("you" or "the Recipient") are decisions of the Recipient and we will not make, or be deemed to make, any decisions on behalf of any Recipient. We will not be liable for the consequences of any such decisions.

Current regulations and good practice were used in the preparation of this report. An appropriately qualified person must review the recommendations given in this report at the time of preparation of the scheme design to ensure that any recommendations given remain valid in light of changes in regulation and practice, or additional information obtained regarding the site.

Any Recipient must take into account any other factors apart from the Report of which they and their experts and advisers are or should be aware. The information, data, conclusions, opinions and judgements set out in the report may relate to certain contexts and may not be suitable in other contexts. It is your responsibility to ensure that you do not use the information we provide in the wrong context.

This report is based on readily available geological records, the recorded physical investigation, the strata observed in the works, together with the results of completed site and laboratory tests. Whilst skill and care has been taken to interpret these conditions likely between or below investigation points, the possibility of other characteristics not revealed cannot be discounted, for which no liability can be accepted. The impact of our assessment on other aspects of the development required evaluation by other involved parties.

The opinions expressed cannot be absolute due to the limitations of time and resources within the context of the agreed brief and the possibility of unrecorded previous in ground activities. The ground conditions have been sampled or monitored in recorded locations and tests for some of the more common chemicals generally expected. Other concentrations of types of chemicals may exist. It was not part of the scope of this report to comment on environment/contaminated land considerations.

The conclusions and recommendations relate to 21 Baldwin Gardens & 43 Leather Lane, Holburn, London EC1N 7TJ.

Trial hole is a generic term used to describe a method of direct investigation. The term trial pit, borehole or window sampler borehole implies the specific technique used to produce a trial hole.

The depth to roots and/or of desiccation may vary from that found during the investigation. The client is responsible for establishing the depth to roots and/or of desiccation on a plot-by-plot basis prior to the construction of foundations. Where trees are mentioned in the text this means existing trees, recently removed trees (approximately 15 years to full recovery on cohesive soils) and those planned as part of the site landscaping.

Ownership of copyright of all printed material including reports, laboratory test results, trial pit and borehole log sheets, including drillers log sheets, remain with Ground and Water Limited. Licence is for the sole use of the client and may not be assigned, transferred or given to a third party.

Only our client may rely on this report and should this report or any information contained in it be provided to any third party we accept no responsibility to the third party for the contents of this report save to the extent expressly outlined by us in writing in a reliance letter addressed from us to the third party.

Recipients are not permitted to publish this report outside of their organisation without our express written consent.

---

2 The Long Barn, Norton Farm, Selborne Road, Alton, Hampshire GU34 3NB
0333 600 1221 enquiries@groundandwater.co.uk **groundandwater.co.uk**

APPENDIX B: Technical Glossary

2 The Long Barn, Norton Farm, Selborne Road, Alton, Hampshire GU34 3NB
0333 600 1221 enquiries@groundandwater.co.uk **groundandwater.co.uk**

TECHNICAL GLOSSARY

The list of possible definitions within the report may be seen below. Please note that some definitions may not be relevant to this report.

HYDROGEOLOGY:

A **Principal Aquifer** is a layer of rock or drift deposits that have high intergranular and/or fracture permeability - meaning they usually provide a high level of water storage. They may support water supply and/or river base flow on a strategic scale. In most cases, principal aquifers are aquifers previously designated as major aquifer.

Secondary (A) Aquifers consist of deposits with permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers. These are generally aquifers formerly classified as Minor Aquifers.

Secondary (B) Aquifers consist of deposits with predominantly lower permeability layers with may stoke and yield limited amounts of groundwater due to localised features such as fissures, think permeable horizons and weathering. These are generally the water-bearing parts of the former non-aquifers.

Secondary Aquifers (Undifferentiated) are assigned in cases where it has not been possible to attribute either category A or B to a rock type. In most cases, this means that the layer in question has previously been designated as both a minor aquifer and non-aquifer in different locations due to the variable characteristics of the rock type.

Unproductive Strata are rock layers with low permeability that have negligible significance for water supply or river base flow. These were formerly classified as non-aquifers.

FLOOD ZONES:

Environment Agency Flood Zone 2, defined as; land having between a 1 in 100 and 1 in 1,000 annual probability of river flooding; or land having between a 1 in 200 and 1 in 1,000 annual probability of sea flooding.

Environment Agency Flood Zone 3 shows the extent of a river flood with a 1 in 100 (1%0 or greater chance of occurring in any year or a sea flood with a 1 in 200 (0.5%) or greater chance of occurring in any year.

Environment Agency Flood Zone 3 area that benefits from flood defences, defined as; land and property in this flood zone would have a high probability of flooding without the local flood defences. These protect the area against a river flood with a 1% chance of happening each year, or a flood from the sea with a 0.5% chance of happening each year.

GROUNDWATER SOURCE PROTECTION ZONES (SPZS):

Inner Zone (SPZ1): This zone is 50 day travel time of pollutant to source with a 50 metres default minimum radius.

Outer Zone (SPZ2): This zone is 400 day travel time of pollutant to source. This has a 250 or 500 metres minimum radius around the source depending on the amount of water taken.

Total Catchment (SPZ3): This is the area around a supply source within which all the groundwater ends up at the abstraction point. This is the point from where the water is taken. This could extend some distance from the source point.

Zone of Special Interest (SPZ4): This zone is where local conditions require additional protection.

IN-SITU STRENGTH GEOTECHNICAL TESTING:

Windowless Sample and/or Cable Percussion and/or Rotary Boreholes provide samples of the ground for assessment but they do not give any engineering data. The standard penetration test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The test uses a thick-walled sample tube, with an outside diameter of 50mm and an inside diameter of 35mm, and a length of around 650mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5kg falling through a distance of 760mm. The sample tube is driven 150mm into the ground and then the number of blows needed for the tube to penetrate each 75mm up to a depth of 450mm is recorded. The sum of the number of blows is termed the "standard penetration resistance" or the "N-value".

Dynamic Probing involves the driving of a metal cone into the ground via a series of steel rods. These rods are driven from the surface by a hammer system that lifts and drops a 63.5kg (SHDP) hammer onto the top of the rods through a set height, thus ensuring a consistent energy input. The number of hammer blows that are required to drive the cone down by each 100mm increment are recorded. These blow counts then provide a comparative assessment from which correlations have been published, based on dynamic energy, which permits engineering parameters to be generated. (The Dynamic Probe 'Super Heavy' (SHDP) Tests were conducted in accordance with BS 1377; 1990; Part 9, Clause 3.2).