It is proposed to extend the existing subterranean utility room. The proposed works are limited in nature and make only a relatively small change to the existing basement footprint. All new works are confined to the footprint of the existing building and do not extend or underpin the adjoining buildings.

The following loads have been taken for the building:

Timber roof:

Permanent (roof structure and finishes) = 1.25 kN/m^2

Variable = 0.75 kN/m^2

Timber floor:

Permanent (floor structure and finishes)= 1.5 kN/m^2

Variable = 1.5 kN/m^2

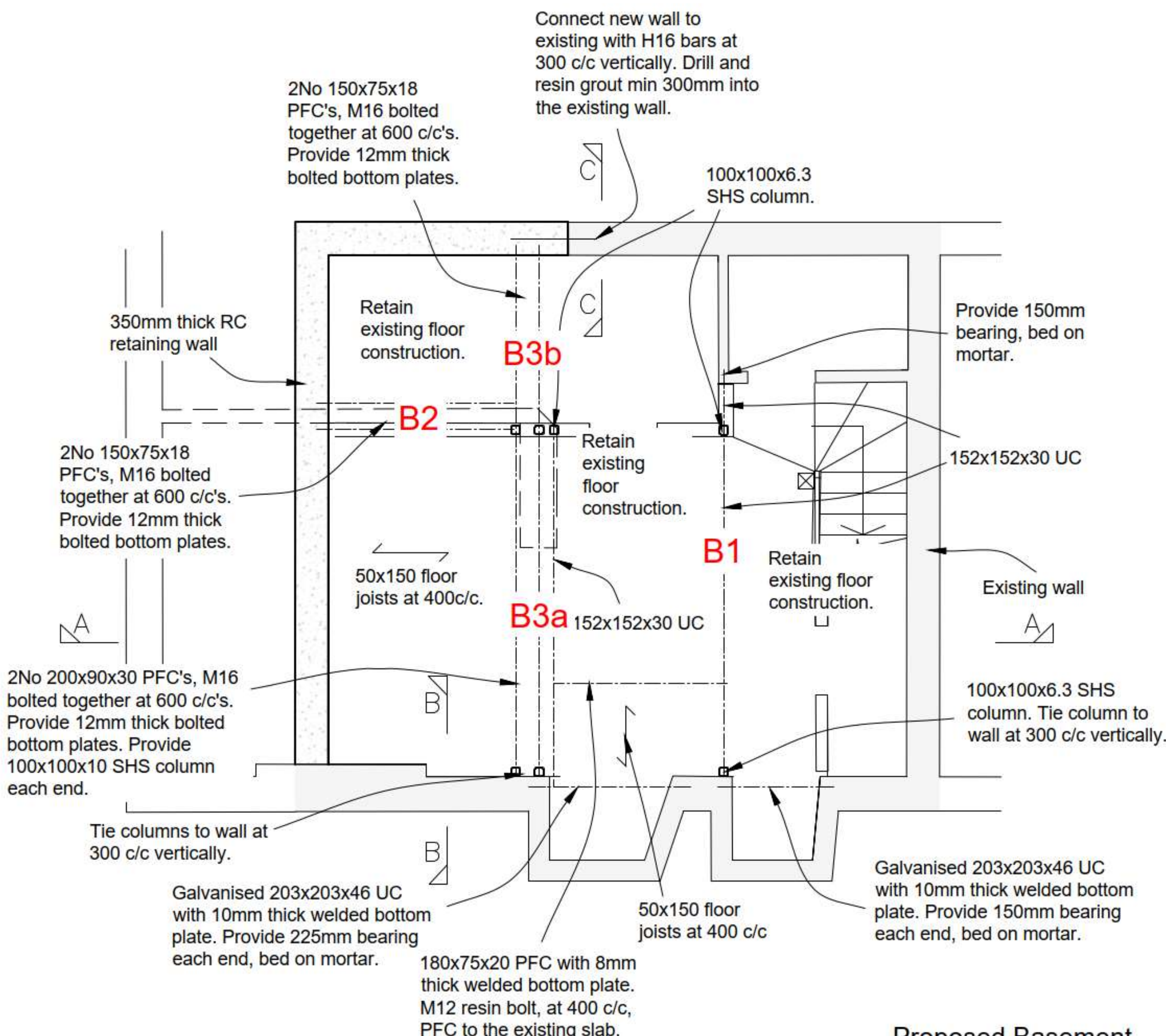
Slab over basement:

Permanent

- Finishes = 0.75 kN/m^2
- 150 slab = 3.75 kN/m^2

Total = 4.5 kN/m^2

Variable = 1.5 kN/m^2



Proposed Basement (Structure Above)
Scale 1:50@A1

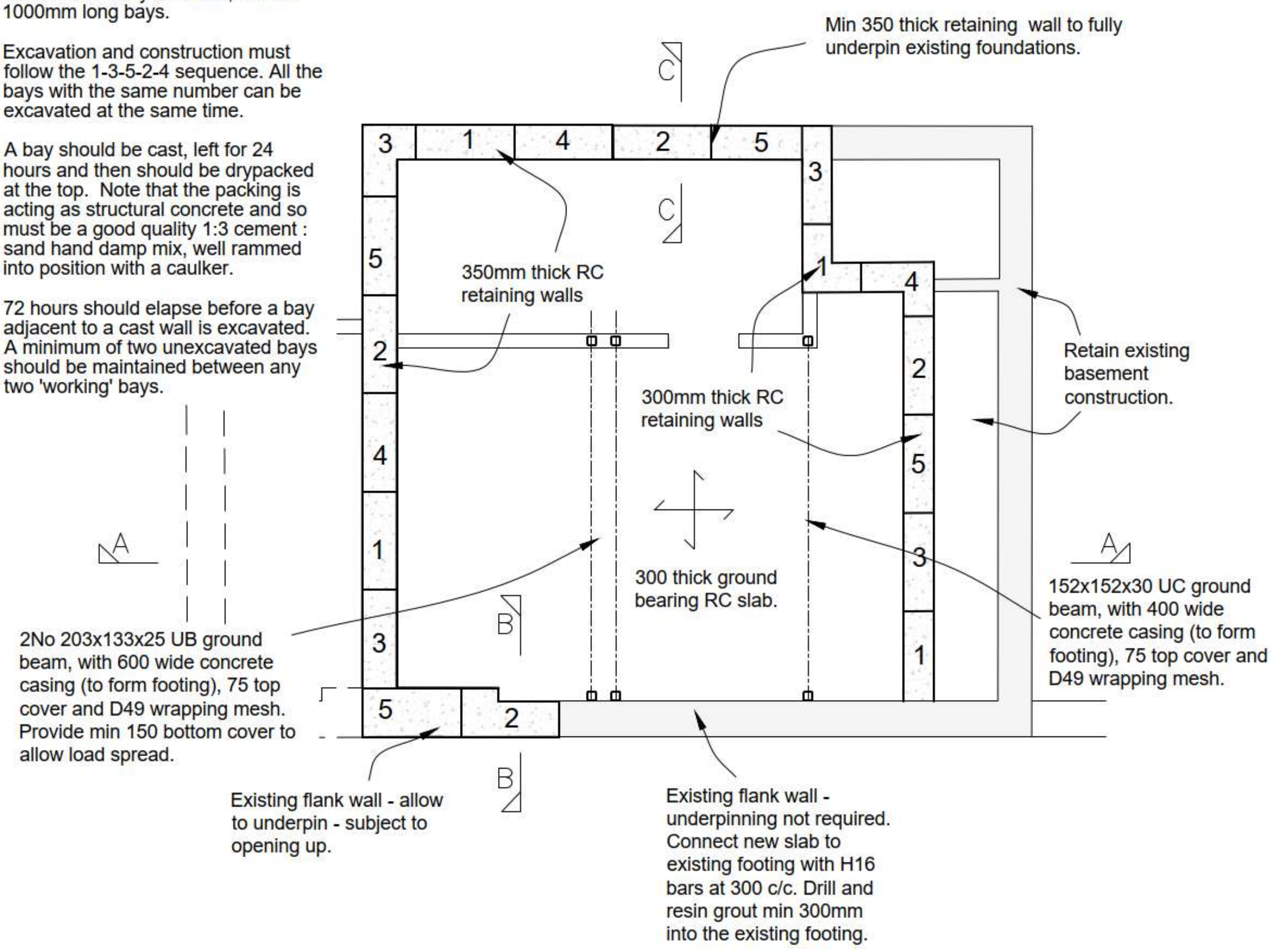
Hit & Miss sequence:

Max 20% of the wall length can be excavated at any one time, in max 1000mm long bays.

Excavation and construction must follow the 1-3-5-2-4 sequence. All the bays with the same number can be excavated at the same time.

A bay should be cast, left for 24 hours and then should be drypacked at the top. Note that the packing is acting as structural concrete and so must be a good quality 1:3 cement : sand hand damp mix, well rammed into position with a caulker.

72 hours should elapse before a bay adjacent to a cast wall is excavated. A minimum of two unexcavated bays should be maintained between any two 'working' bays.



Proposed Basement (Structure Below)

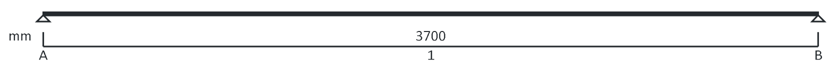
Scale 1:50@A1

B1

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Timber floor 1.5kN/m2x2m/2 - Permanent full UDL 1.5 kN/m
	Concrete slab 4.5kN/m2×2m/2 - Permanent full UDL 4.5 kN/m
	Grd floor 1.5kN/m2×4m/2 - Variable full UDL 3 kN/m

Analysis results

Maximum moment;	M_{max} = **22.2** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **24** kN;	V_{min} = **-24** kN
Deflection;	δ_{max} = **6.2** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **24** kN;	R_{A_min} = **24** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **11.6** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **5.6** kN	
Maximum reaction at support B;	R_{B_max} = **24** kN;	R_{B_min} = **24** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **11.6** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **5.5** kN	

Section details

Section type;	**UC 152x152x30 (BS4-1)**;	Steel grade;	**S355**
Section classification;	**Class 1**		

Check shear - Section 6.2.6

Design shear force; V_{Ed} = **24** kN; Design shear resistance; $V_{c,Rd}$ = **236.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; M_{Ed} = **22.2** kNm; Des.bending resist.moment; $M_{c,Rd}$ = **87.9** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT}$ = **0.779**; Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd}$ = **75.1** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **10.3** mm; Maximum deflection; δ = **6.179** mm

PASS - Maximum deflection does not exceed deflection limit

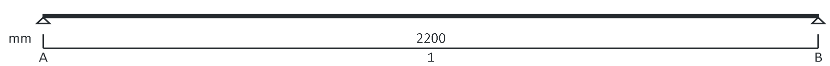
;

B2

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Masonry 3.3mht×2kN/m2 - Permanent full UDL 6.6 kN/m
	1st & 2nd Floors 2×5.6/2×1.5kN/m2 - Permanent full UDL 8.4 kN/m
	1st & 2nd Floors 2×5.6/2×1.5kN/m2 - Variable full UDL 8.4 kN/m
	Grd 1.6m/2×1.5kN/m2 - Permanent full UDL 1.2 kN/m
	Grd 1.6m/2×1.5kN/m2 - Variable full UDL 1.2 kN/m

Analysis results

Maximum moment;	M_{max} = **22.2** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **40.4** kN;	V_{min} = **-40.4** kN
Deflection;	δ_{max} = **2.2** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **40.4** kN;	R_{A_min} = **40.4** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **18.2** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **10.6** kN	
Maximum reaction at support B;	R_{B_max} = **40.4** kN;	R_{B_min} = **40.4** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **18.2** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **10.6** kN	

Section details

Section type; **2 x PFC 150x75x18 (BS4-1)**; Steel grade; **S355**

Section classification; **Class 1**

Check shear - Section 6.2.6

Design shear force; V_{Ed} = **40** kN; Design shear resistance; $V_{c,Rd}$ = **390.2** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; M_{Ed} = **22.2** kNm; Des.bending resist.moment; $M_{c,Rd}$ = **93.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT}$ = **0.783**; Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd}$ = **67.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **6.1** mm; Maximum deflection; δ = **2.206** mm

PASS - Maximum deflection does not exceed deflection limit

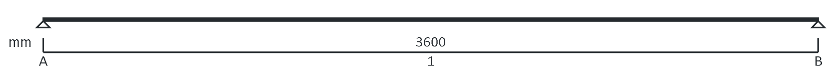
;

B3A

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Masonry 9mht ×2kN/m2 - Permanent full UDL 18 kN/m
	Roof 4m/2×1.25kN/m2 - Permanent full UDL 2.5 kN/m
	Roof 4m/2×1.25kN/m2 - Variable full UDL 1.5 kN/m
	1st&2nd 2×4m/2×1.5kN/m2 - Permanent full UDL 6 kN/m
	1st&2nd 2×4m/2×1.5kN/m2 - Variable full UDL 6 kN/m
	Grd 2m/2×4.5kN/m2 - Permanent full UDL 4.5 kN/m
	Grd 2.2m/2×1.5kN/m2 - Permanent full UDL 1.6 kN/m
	Grd 4.2m/2×1.5kN/m2 - Variable full UDL 3.1 kN/m

Analysis results

Maximum moment; M_{max} = **98.3** kNm; M_{min} = **0** kNm

Maximum shear; V_{max} = **109.3** kN; V_{min} = **-109.3** kN

Deflection; δ_{max} = **9** mm; δ_{min} = **0** mm

Maximum reaction at support A; R_{A_max} = **109.3** kN; R_{A_min} = **109.3** kN

Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = **59.7** kN

Unfactored variable load reaction at support A; $R_{A\_Variable}$ = **19.1** kN

Maximum reaction at support B; R_{B_max} = **109.3** kN; R_{B_min} = **109.3** kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = **59.7** kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = **19.1** kN

Section details

Section type; **2 x PFC 200x90x30 (British Steel Section Range 2022 (BS4-1))**; Steel grade; **S355**

Section classification; **Class 1**

Check shear - Section 6.2.6

Design shear force; V_{Ed} = **109** kN; Design shear resistance; $V_{c,Rd}$ = **627.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; M_{Ed} = **98.3** kNm; Des.bending resist.moment; $M_{c,Rd}$ = **206.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT}$ = **0.951**; Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd}$ = **125.4** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **10** mm; Maximum deflection; δ = **9.035** mm

PASS - Maximum deflection does not exceed deflection limit

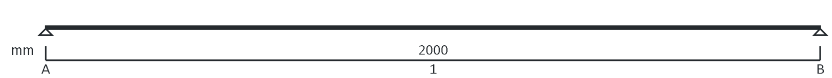
;

B3B

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

Support A Vertically restrained

	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Permanent self weight of beam × 1
	Masonry 9mht ×2kN/m2 - Permanent full UDL 18 kN/m
	Roof 4m/2×1.25kN/m2 - Permanent full UDL 2.5 kN/m
	Roof 4m/2×1.25kN/m2 - Variable full UDL 1.5 kN/m
	1st&2nd 2×4m/2×1.5kN/m2 - Permanent full UDL 6 kN/m
	1st&2nd 2×4m/2×1.5kN/m2 - Variable full UDL 1 kN/m
	Grd 2m/2×4.5kN/m2 - Permanent full UDL 4.5 kN/m
	Grd 2.2m/2×1.5kN/m2 - Permanent full UDL 1.6 kN/m
	Grd 4.2m/2×1.5kN/m2 - Variable full UDL 3.1 kN/m
	B2 - Permanent point load 18.2 kN at 300 mm
	B2 - Variable point load 10.6 kN at 300 mm

Analysis results

Maximum moment;	M_{max} = **32.9** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **87.3** kN;	V_{min} = **-59** kN
Deflection;	δ_{max} = **2.8** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **87.3** kN;	R_{A_min} = **87.3** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **48.4** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **14.6** kN	
Maximum reaction at support B;	R_{B_max} = **59** kN;	R_{B_min} = **59** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **35.7** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **7.2** kN	

Section details

Section type; **2 x PFC 150x75x18 (British Steel Section Range 2022 (BS4-1))**; Steel grade; **S355**

Section classification; **Class 1**

Check shear - Section 6.2.6

Design shear force; V_{Ed} = **87** kN; Design shear resistance; $V_{c,Rd}$ = **390.2** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; M_{Ed} = **32.9** kNm; Des.bending resist.moment; $M_{c,Rd}$ = **93.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio; $\bar{\lambda}_{LT}$ = **0.736**; Limiting slenderness ratio; $\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ > $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment; $M_{b,Rd}$ = **70.9** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **5.6** mm; Maximum deflection; δ = **2.804** mm

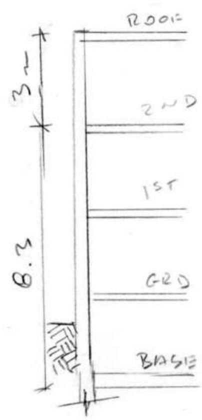
PASS - Maximum deflection does not exceed deflection limit

;

Tekla Tedds	Project				Job no.	
Cooper Associates 6 Bartholomew Place London EC1A 7HH	17A Chesterford Gardens, NW3 7DD				CA6669	
	Calcs for				Start page no./Revision	
	Basement				9/B	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	DD	03/11/2023				

LOAD TAKE DOWN – FLANK WALL

FLANK WALL:



		DL [kN/m]	IL [kN/m]
ROOF	$3.8m/2 \times 1.25\,kN/m^2\ (0.75\,kN/m^2)$ =	2.4	1.4
2ND	$3.8m/2 \times 1.5\,kN/m^2\ (1.5\,kN/m^2)$ =	2.8	2.8
1ST	—— " —— =	2.8	2.8
GRD	—— " —— =	2.8	2.8

GROUND BEARING

345 WALL: $0.345m \times 8.3m \times 20\,kN/m^3 \times 0.8 = 45.8$

OPENINGS

225 WALL: $0.225 \times 3m \times 20\,kN/m^3 \times 0.8 = 10.8$

67.4   9.8

TOTAL 77.2 kN/m

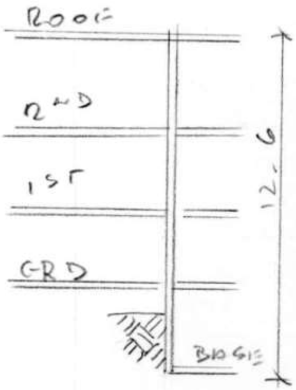
EXISTING BEARING PRESSURE WITHOUT LATERAL PRESSURE OF SOIL

$$\frac{77.2\,kN/m}{0.745m} = 104\,kN/m^2$$

Tekla. Tedds Cooper Associates 6 Bartholomew Place London EC1A 7HH	Project 17A Chesterford Gardens, NW3 7DD				Job no. CA6669	
	Calcs for Basement				Start page no./Revision 10/B	
	Calcs by DD	Calcs date 03/11/2023	Checked by	Checked date	Approved by	Approved date

LOAD TAKE DOWN – CORRIDOR WALL

COMMUNAL CORRIDOR WALL



		DL [kN/m]	LL [kN/m]
ROOF	$6.6m/2 \times 1.25\,kN/m^2\,(0.75)$ =	4.1	2.5
2ND	$6.6m/2 \times 1.5\,kN/m^2\,(1.5)$ =	5.0	5.0
1ST	—— " —— =	5.0	5.0
GRD	—— " —— =	5.0	5.0

100 WALL : $0.1m \times 12.6m \times 20\,kN/m^3 \times 0.9 = 22.7$

OPENINGS

41.8   17.5

TOTAL = $59.3\,kN/m$

EXISTING BEARING PRESSURE WITHOUT LATERAL PRESSURE OF SOIL

$$\frac{59.3\,kN/m}{0.745m} = 80\,kN/m^2$$

BEARING CAPACITY

Bearing capacity of 100kPa assumed. The bearing capacity is based on the visual assessment of soil in trial pits and experience in the area. Stiff clay is normally accepted as 100 kN/m2 and often has a higher GBC. The bearing pressure below existing walls is bigger than 100kPa.

RETAINING WALL ANALYSIS & DESIGN (EN1992)

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.22

Retaining wall details

Stem type; Cantilever

Stem height; h_{stem} = **2500** mm

Stem thickness; t_{stem} = **350** mm

Angle to rear face of stem; α = **90** deg

Stem density; γ_{stem} = **25** kN/m^3

Toe length; l_{toe} = **1800** mm

Base thickness; t_{base} = **300** mm

Base density; γ_{base} = **25** kN/m^3

Height of retained soil; h_{ret} = **2500** mm; Angle of soil surface; β = **0** deg

Depth of cover; d_{cover} = **0** mm

Retained soil properties

Soil type; Stiff clay

Moist density; γ_{mr} = **19** kN/m^3

Saturated density; γ_{sr} = **19** kN/m^3

Characteristic effective shear resistance angle; $\phi'_{r.k}$ = **18** deg

Characteristic wall friction angle; $\delta_{r.k}$ = **9** deg

Base soil properties

Soil type; Stiff clay

Soil density; γ_b = **19** kN/m^3

Characteristic effective shear resistance angle; $\phi'_{b.k}$ = **18** deg

Characteristic wall friction angle; $\delta_{b.k}$ = **9** deg

Characteristic base friction angle; $\delta_{bb.k}$ = **12** deg

Presumed bearing capacity; $P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load; Surcharge$_Q$ = **5** kN/m^2

Calculate retaining wall geometry

Base length; l_{base} = **2150** mm

Moist soil height; h_{moist} = **2500** mm

Length of surcharge load; l_{sur} = **0** mm

Vertical distance; x_{sur_v} = **2150** mm

Effective height of wall; h_{eff} = **2800** mm

Horizontal distance; x_{sur_h} = **1400** mm

Area of wall stem; A_{stem} = **0.875** m^2; Vertical distance; x_{stem} = **1975** mm

Area of wall base; A_{base} = **0.645** m^2; Vertical distance; x_{base} = **1075** mm

Using Coulomb theory

Active pressure coefficient; K_A = **0.483**; Passive pressure coefficient; K_P = **2.359**

Bearing pressure check

Vertical forces on wall

Total; $F_{total\_v} = F_{stem} + F_{base}$ = **38** kN/m

Horizontal forces on wall

Total; $F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{pass\_h} =$ **40.2** kN/m

Moments on wall

Total; $M_{total} = M_{stem} + M_{base} + M_{sur} + M_{moist} =$ **18** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} =$ **40.2** kN/m

Bearing pressure at toe; $q_{toe} =$ **53.4** kN/m^2; Bearing pressure at heel; $q_{heel} =$ **0** kN/m^2

Factor of safety; $FoS_{bp} =$ **1.872**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.22

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C32/40

Char.comp.cylinder strength; $f_{ck} =$ **32** N/mm^2; Mean axial tensile strength; $f_{ctm} =$ **3.0** N/mm^2

Secant modulus of elasticity; $E_{cm} =$ **33346** N/mm^2; Maximum aggregate size; $h_{agg} =$ **20** mm

Design comp.concrete strength; $f_{cd} =$ **18.1** N/mm^2; Partial factor; $\gamma_C =$ **1.50**

Reinforcement details

Characteristic yield strength; $f_{yk} =$ **500** N/mm^2; Modulus of elasticity; $E_s =$ **200000** N/mm^2

Design yield strength; $f_{yd} =$ **435** N/mm^2; Partial factor; $\gamma_S =$ **1.15**

Cover to reinforcement

Front face of stem; $c_{sf} =$ **30** mm; Rear face of stem; $c_{sr} =$ **75** mm

Top face of base; $c_{bt} =$ **30** mm; Bottom face of base; $c_{bb} =$ **50** mm

Check stem design at base of stem

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment; M = **43.1** kNm/m; K = **0.019**; K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required; $A_{sr.req} =$ **390** mm^2/m

Tens.reinforcement provided; 16 dia.bars @ 200 c/c; Tens.reinforcement provided; $A_{sr.prov} =$ **1005** mm^2/m

Min.area of reinforcement; $A_{sr.min} =$ **420** mm^2/m; Max.area of reinforcement; $A_{sr.max} =$ **14000** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single summary

Deflection control - Section 7.4

Limiting span to depth ratio; 16 Actual span to depth ratio; 9.4

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} =$ **0.3** mm; Maximum crack width; $w_k =$ **0.164** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **47.2** kN/m

Rectangular section in shear - Section 6.2

Design shear force; V = **47.2** kN/m; Design shear resistance; $V_{Rd.c} =$ **137** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement; $A_{sx.req}$ = **350** mm^2/m; Max.spacing of reinforcement; s_{sx_max} = **400** mm

Trans.reinforcement provided; 10 dia.bars @ 200 c/c; Trans.reinforcement provided; $A_{sx.prov}$ = **393** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment; M = **44.2** kNm/m; K = **0.023**; K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required; $A_{bb.req}$ = **439** mm^2/m

Tens.reinforcement provided; 12 dia.bars @ 200 c/c; Tens.reinforcement provided; $A_{bb.prov}$ = **565** mm^2/m

Min.area of reinforcement; $A_{bb.min}$ = **384** mm^2/m; Max.area of reinforcement; $A_{bb.max}$ = **12000** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single summary

Crack control - Section 7.3

Limiting crack width; w_{max} = **0.3** mm; Maximum crack width; w_k = **0.297** mm

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **33** kN/m

Rectangular section in shear - Section 6.2

Design shear force; V = **33** kN/m; Design shear resistance; $V_{Rd.c}$ = **127.1** kN/m

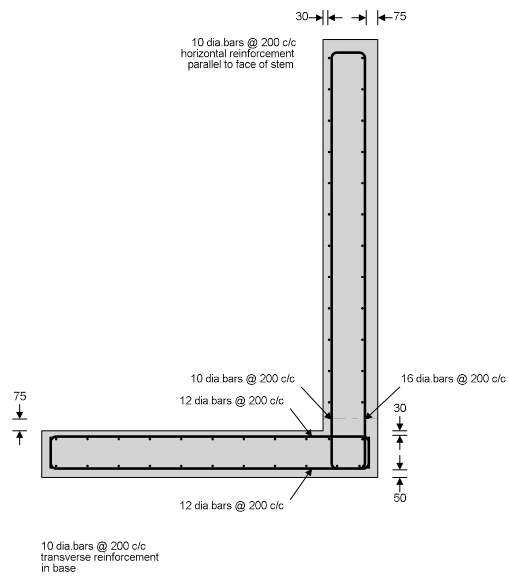
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Min.area of reinforcement; $A_{bx.req}$ = **113** mm^2/m; Max.spacing of reinforcement; s_{bx_max} = **450** mm

Trans.reinforcement provided; 10 dia.bars @ 200 c/c; Trans.reinforcement provided; $A_{bx.prov}$ = **393** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

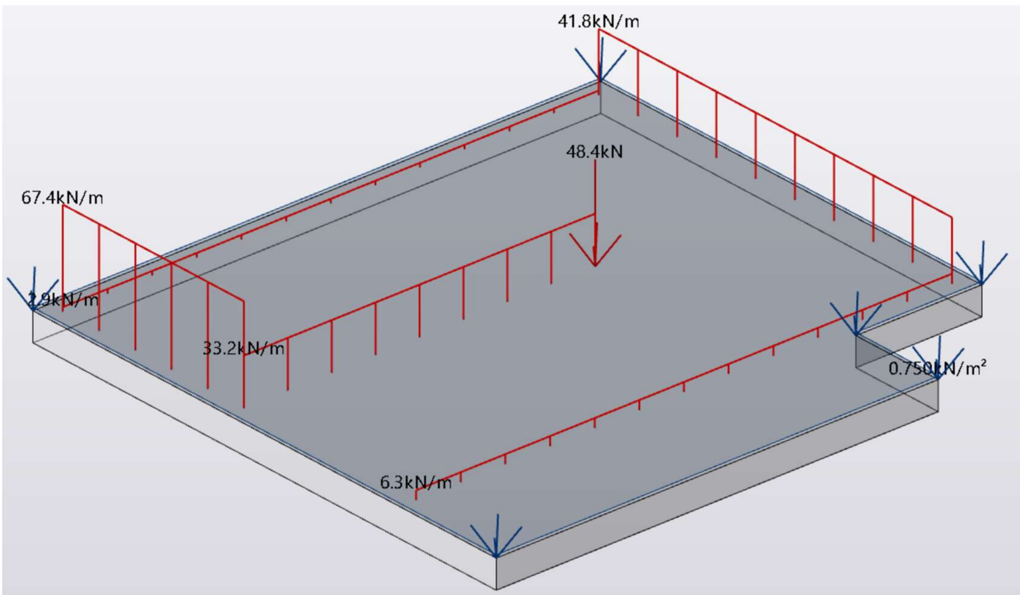
;

Tekla Tedds	Project				Job no.	
Cooper Associates 6 Bartholomew Place London EC1A 7HH	17A Chesterford Gardens, NW3 7DD				CA6669	
	Calcs for				Start page no./Revision	
	Basement				15/B	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	DD	03/11/2023				

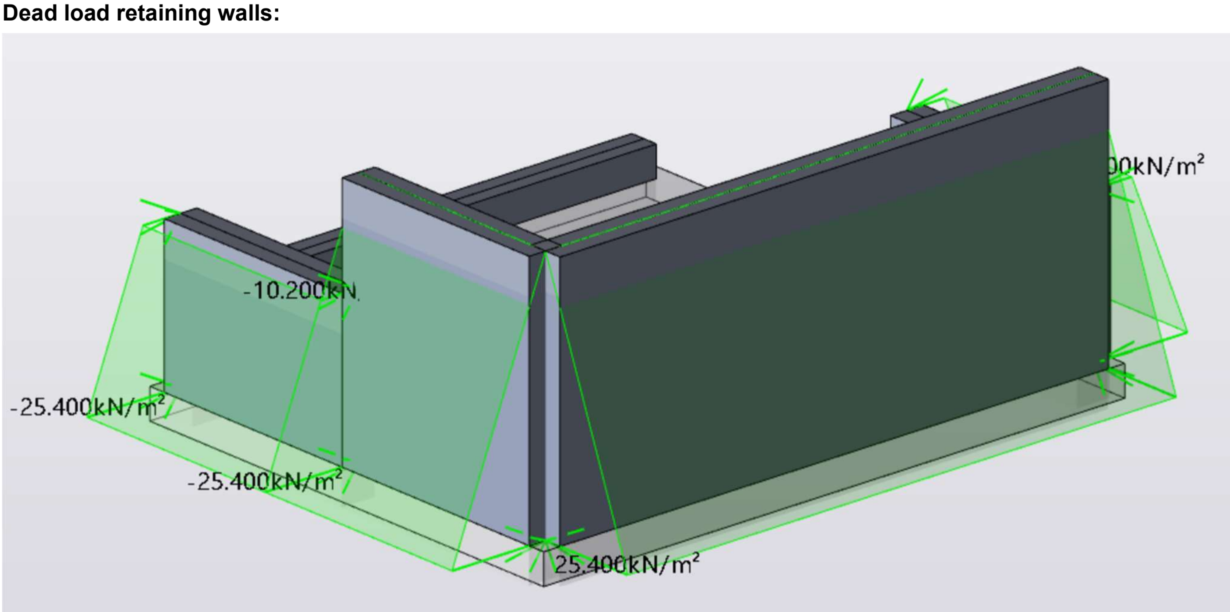
PROPOSED BEARING PRESSURE AND DEFLECTION

The proposed RC retaining walls and raft slab has been modelled in Tekla Structural Designer to check proposed bearing pressure and deflection. Ground stiffnes of 12.000 kN/m3 was assumed in the analysis (clayey soil qa<200kPa - reference: "Foundation Analysis and Design", Joseph E. Bowles, 1995).

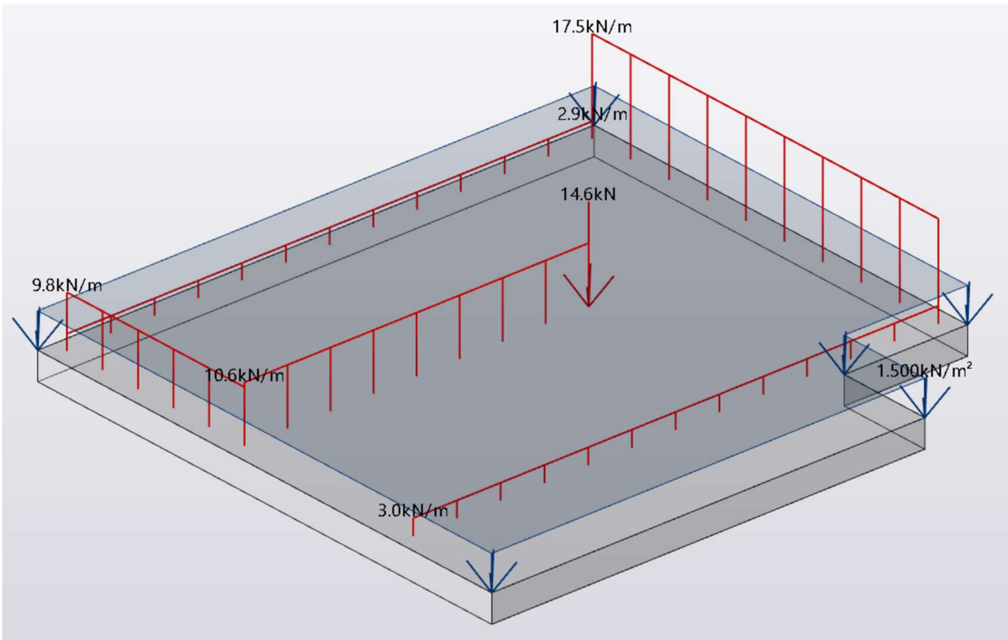
Dead load slab:



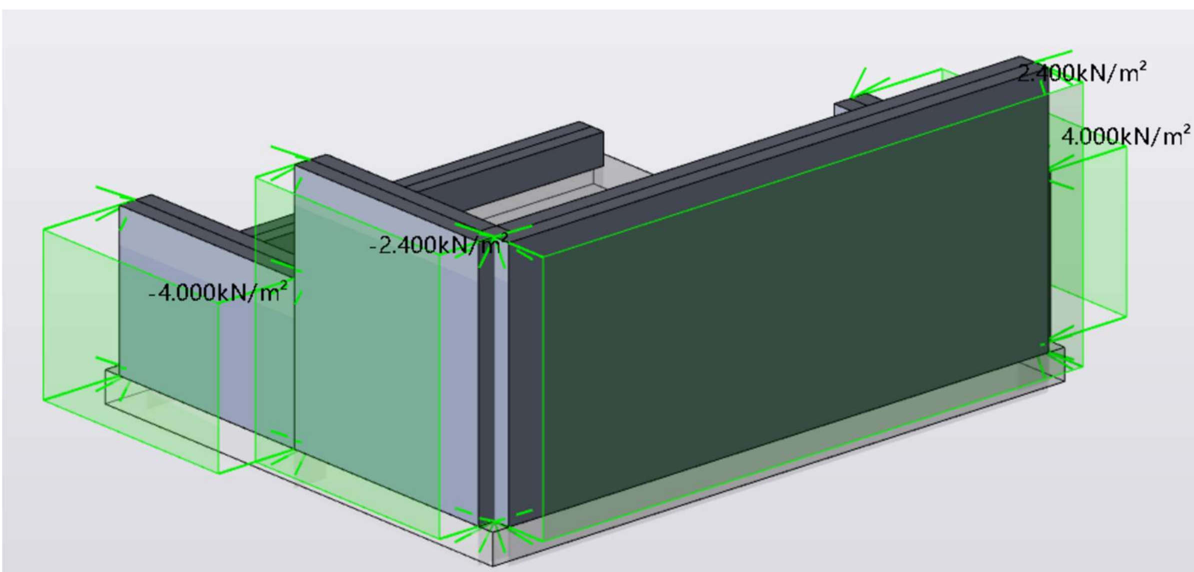
Dead load retaining walls:



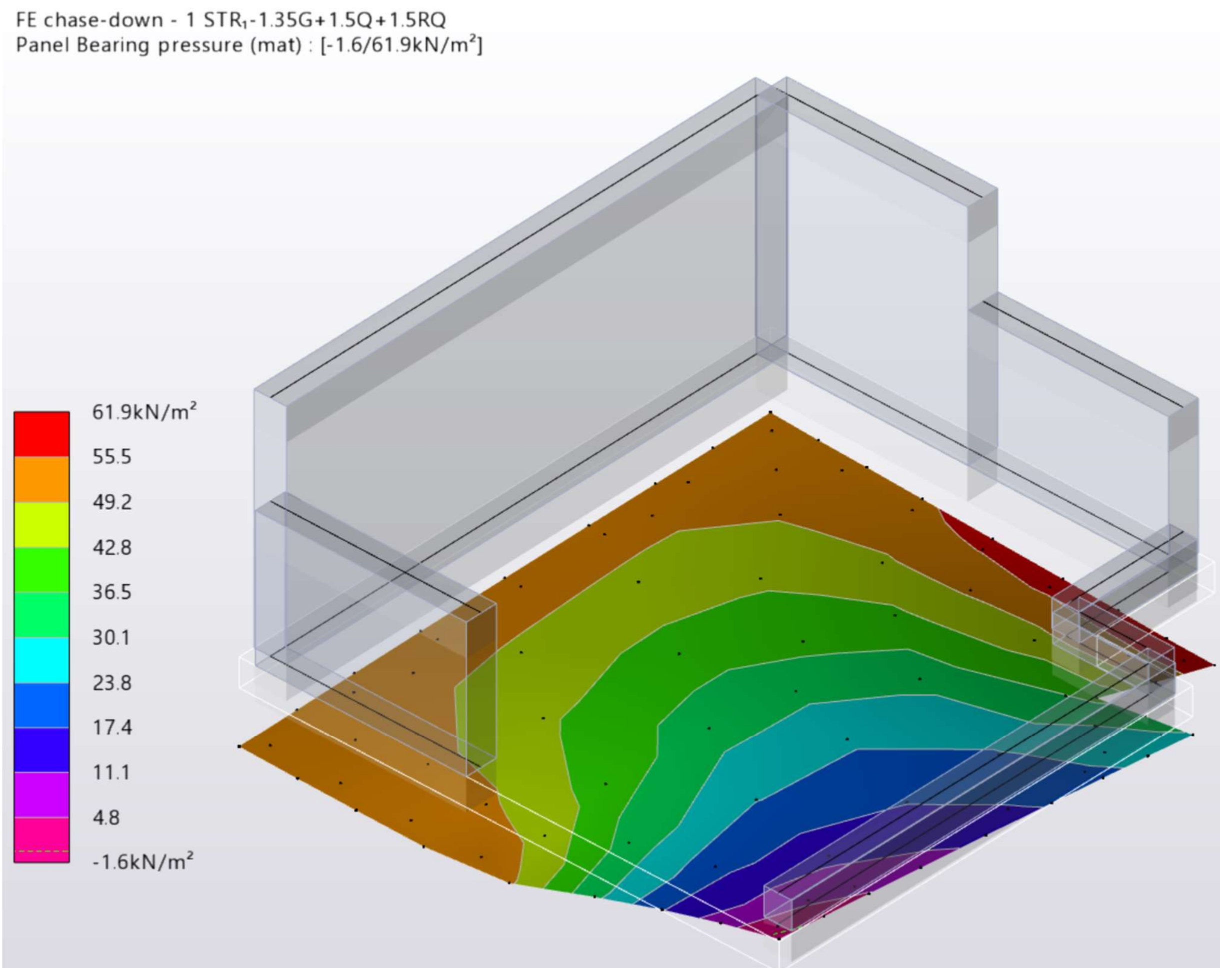
Live load slab:



Live load retaining walls:



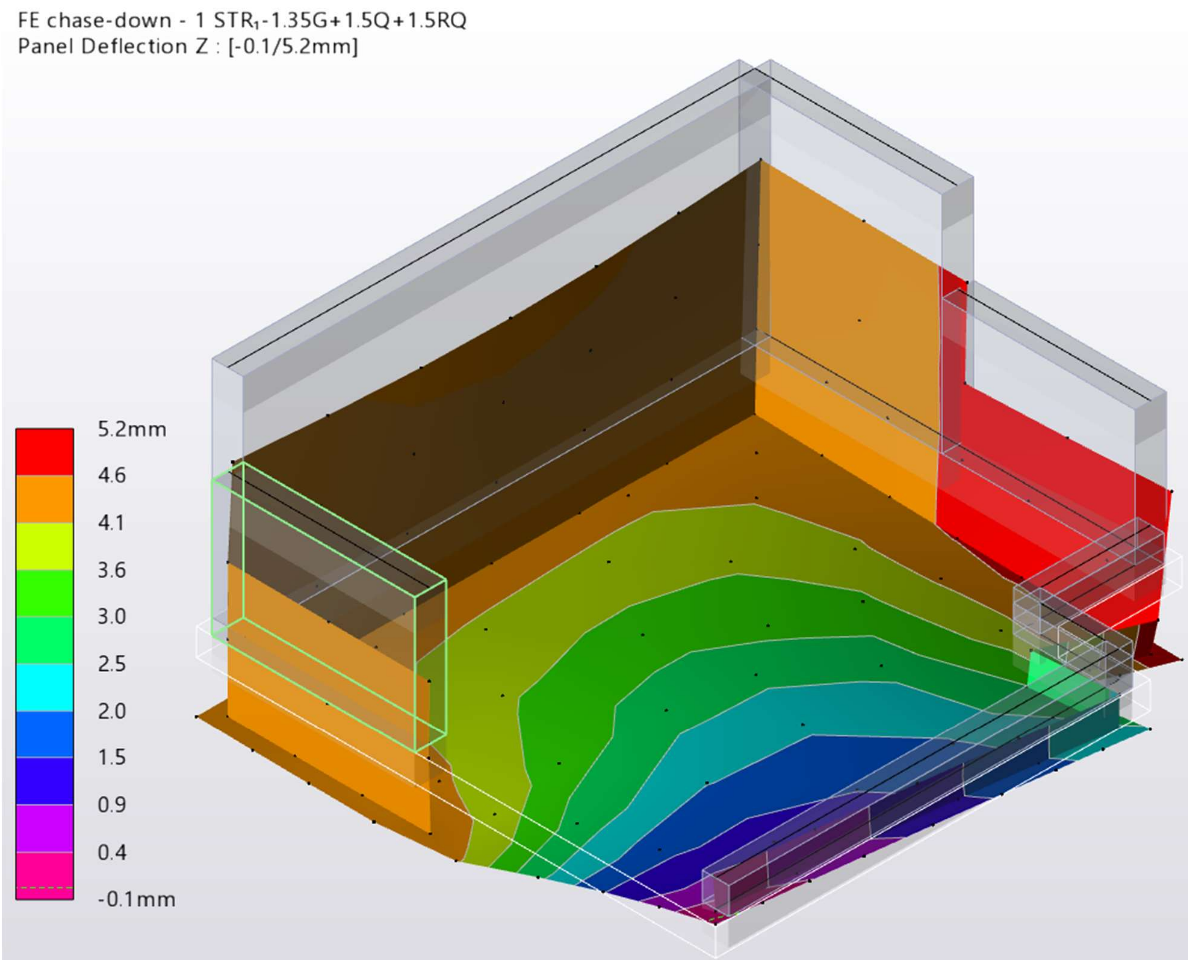
Bearing pressure:



Bearing pressure is less than 100kPa – Ok.

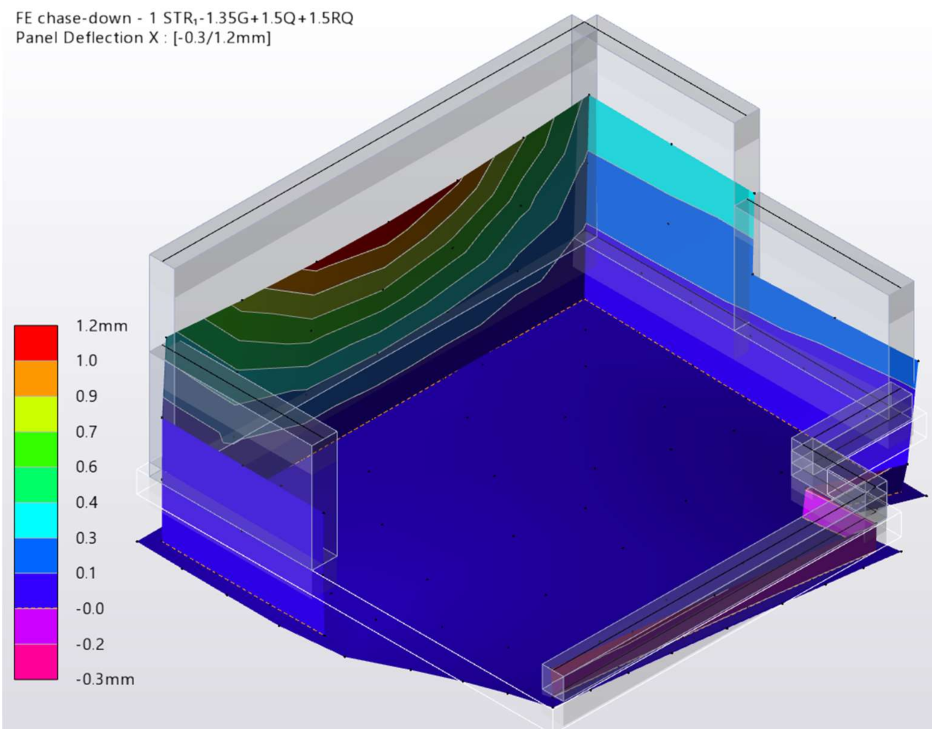
Tekla Tedds Cooper Associates 6 Bartholomew Place London EC1A 7HH	Project 17A Chesterford Gardens, NW3 7DD				Job no. CA6669	
	Calcs for Basement				Start page no./Revision 18/B	
	Calcs by DD	Calcs date 03/11/2023	Checked by	Checked date	Approved by	Approved date

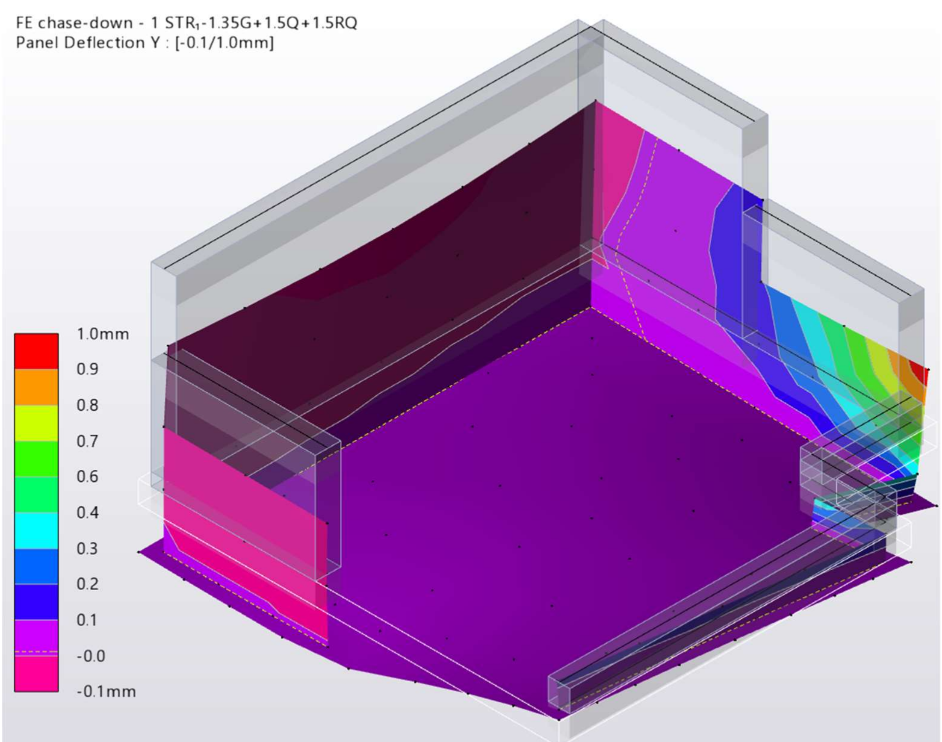
Vertical deflection:



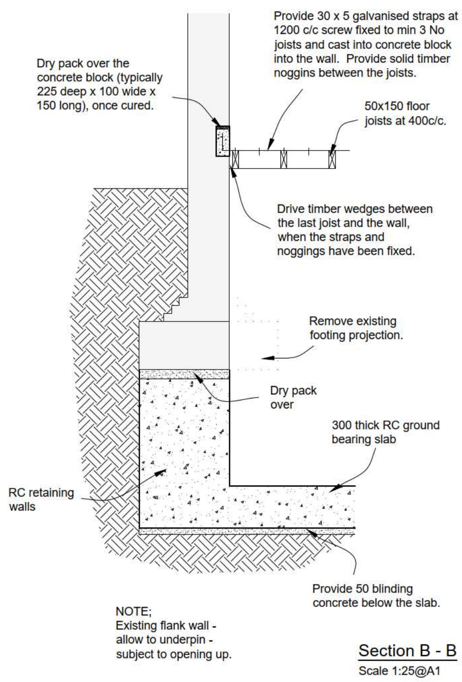
.

Horizontal deflection:





CHECK EXISTING MASONRY PANEL



MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.15

Masonry panel details

Retaining wall - Unreinforced masonry wall without openings

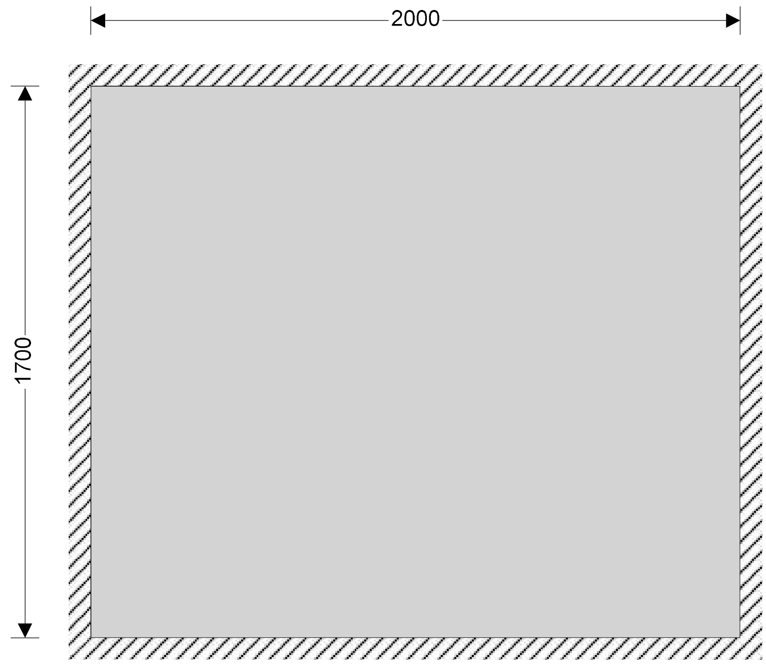
Panel length; L = **2000** mm; Panel height; h = **1700** mm

Panel support conditions

; **All edges supported, right and left continuous**

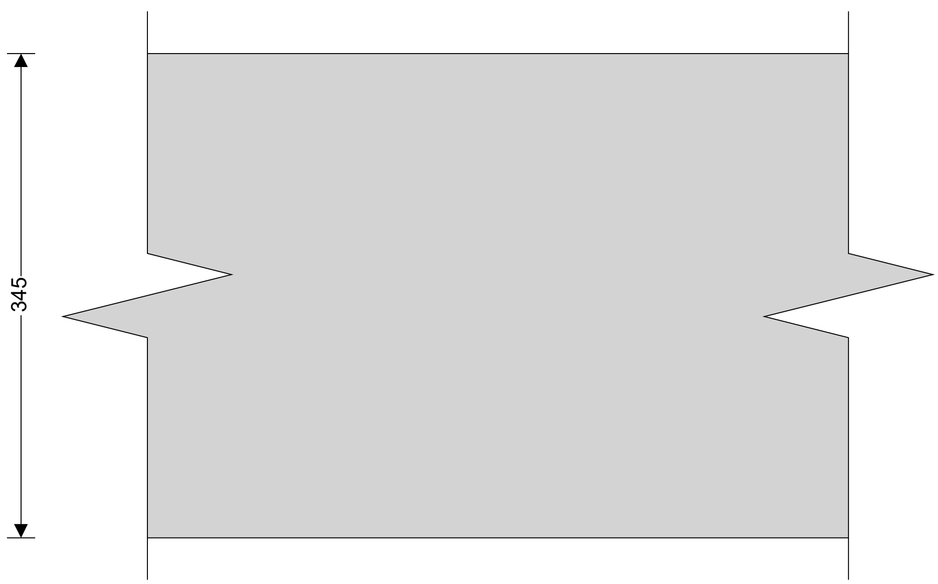
- Vertical supports provide enhanced resistance to lateral movement

Effective panel length; L_{ef} = **1500** mm; Effective panel height; h_{ef} = **1700** mm



Single-leaf wall construction details

Wall thickness; t = **345** mm; Effective wall thickness; t_{ef} = **345** mm



Masonry details

Masonry type; **Clay bricks having a water absorption between 7% and 12%**

Compressive strength of unit; p_{unit} = **7.0** N/mm^2; Mortar strength Class; **M2 / (iv)**

Height of masonry units; h_b = **65** mm; Density of masonry; γ = **18.0** kN/m^3

From BS5628-1 Table 2a - Characteristic compressive strength of masonry

Compressive strength; f_k = **2.44** N/mm^2

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Failure parallel to bed joints; f_{kx_para} = **0.35** N/mm^2; Failure perp. to bed joints; f_{kx_perp} = **1.00** N/mm^2

Lateral loading details

Characteristic wind load on panel; W_k = **10.000** kN/m^2

Vertical loading details

Dead load on top of wall; G_k = **67.4** kN/m;

Imposed load on top of wall; Q_k = **9.8** kN/m;

Partial safety factors for material strength

Category of manufacture; **Category II**; Category of construction; **Normal**

Partial factor for compression; γ_{mc} = **3.50**; Partial factor for flexure; γ_{mf} = **3.00**

Partial factor for shear; γ_{mv} = **2.50**

Slenderness ratio (cl 24.1)

Allowable slenderness ratio; SR_{all} = 27; Slenderness ratio; SR = **4.3**

PASS - Slenderness ratio is less than maximum allowable

<u>Vertical loading (cl 28)</u>

Partial safety factors for design loads

Partial factor for dead load; γ_{fG} = **1.40**; Partial factor for imposed load; γ_{fQ} = **1.60**

Check vertical loads at top of wall

Design vertical load stress; f_v = **0.319** N/mm^2; Allowable stress capacity; f_{cap} = **0.697** N/mm^2

PASS - Allowable stress capacity exceeds design vertical load stress on wall

<u>Horizontal loading (cl 32)</u>

Limiting dimensions (cl 32.3)

Area of panel; A_p = **3.4** m^2; Limiting area of panel; A_{max} = **241.0** m^2

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension; L_{max} = **17250** mm

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial factor for wind load; γ_{fW} = **1.40**; Partial factor for dead load; γ_{fG} = **0.90**

<u>Design moments of resistance in panels (cl 32.4.2)</u>

Elastic moment of resistance; M_d = **6.613** kNm/m

<u>Design moment in panels (cl 32.4.2)</u>

Design moment in wall; $M = \alpha \times W_k \times \gamma_{fW} \times L^2$ = **1.464** kNm/m

PASS - Resistance moment exceeds design moment

;

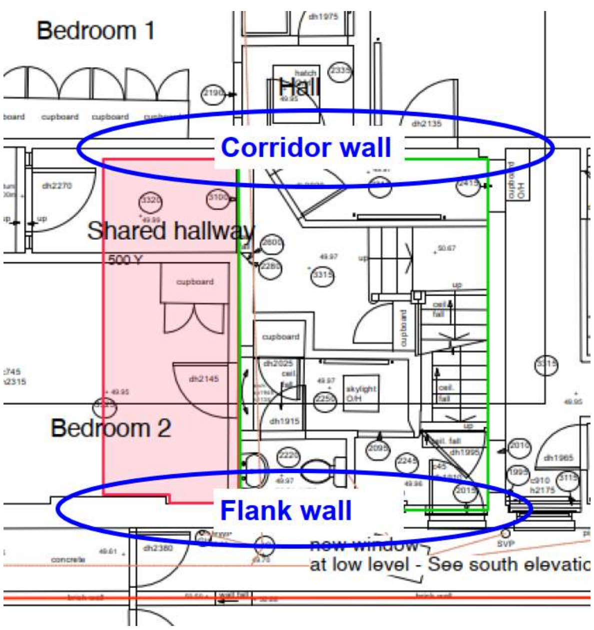
DAMAGE CATEGORY ASSESSMENT

This analysis is based on the guidance of CIRIA C760.

The two primary sources of cracking are vertical distortions from differential settlement and tapering cracks arising from the horizontal tension strains from the settlement/rebound wave. Burland suggested that the façade of the building can be considered as a large deep beam with the bending and diagonal strains within it depending on its proportions, i.e. ratio of the Length/Height, L/H. On tall narrow buildings, with L/H below unity, diagonal cracking from differential settlement predominates whereas on long squat buildings or terraces, tension cracks due to bending predominates.

The two types of cracking relate to vertical and horizontal strains. Utilising the concept of the limiting strains, envelopes of increasing damage can be developed combining the two types of movement for various building proportions. This is of limited value and it is more useful in practice to develop envelopes of different damage categories for a given façade proportion. The two axes on the Burland Scale charts are vertical differential settlement, Δ/L, and horizontal strains ε_h on to which different crack severity envelopes can be plotted.

Flank and corridor wall



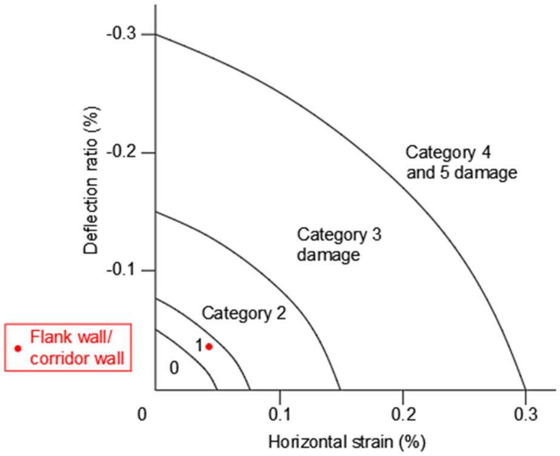
The flank wall and corridor wall are going to be partially underpinned. This is a three storey building with a height of 11m and the basement is 7m long giving a L/H ration just over 0.5.

The maximum vertical deflection for the basement is -5.2mm. The sagging can be conservatively considered half of that (-5.2mm / 2 = -2.6mm).

The maximum horizontal movement for the basement is 1.2mm. This horizontal movement is small compering to a typical basement as retaining walls are just 2.6m high and they are restrained by RC walls, masonry walls and steel columns. The maximum horizontal movement is multiplied by 2.5 to allow for movement during construction (1.2mm * 2.5 = 3mm).

Deflection ratio: $\Delta/L=-2.6*100/7*10^3 = -0.037\%$

Horizontal strain: $\varepsilon_h=\delta_h/L=3.0*100/7*10^3 = 0.043\%$

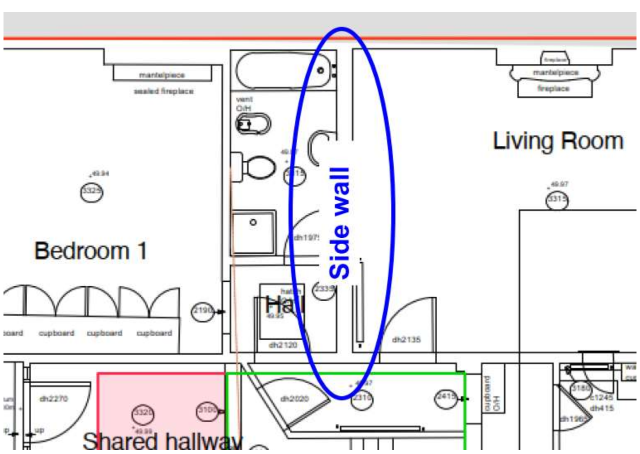


Relationship between damage category and deflection and horizontal tensile strain for (L/H) = 0.5

The predicted movement of the flank wall and corridor wall due to basement extension is in Burland Category 1, Very Slight.

A formal monitoring system should be employed during the construction phase in order to detect movement early, if any occurs.

Side wall



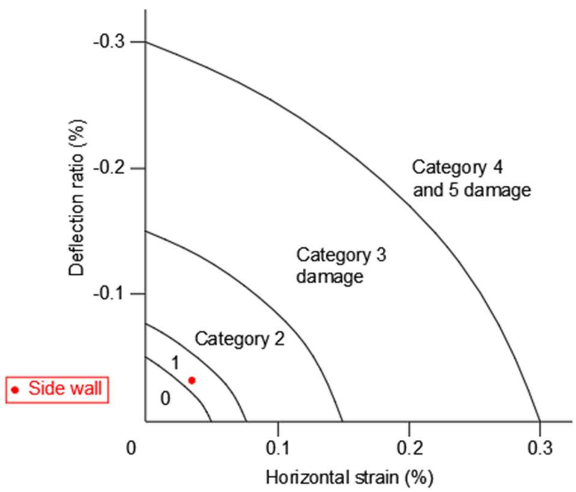
The side wall next to the basement is 5.2m long and with a height of 11m giving a L/H ration just under 0.5.

The maximum vertical deflection for the basement is -5.2mm.The basement in this location will be approx. 200mm lower than the existing basement. The sagging can be conservatively considered a third of the maximum vertical deflection (-1.7mm).

The maximum horizontal movement for the basement is 1.2mm. The basement is restrained by masonry wall at the junction with the side wall. The maximum horizontal movement is multiplied by 1.5 to allow for movement during construction (1.2mm*1.5 = 1.8mm).

Deflection ratio: $\Delta/L=-1.7*100/5.2*10^3 = 0.033\%$

Horizontal strain: $\varepsilon_h=\delta_h/L=1.8*100/5.2*10^3 = 0.035\%$



Relationship between damage category and deflection and horizontal tensile strain for (L/H) = 0.5

The predicted movement of the side wall due to basement extension is in Burland Category 1, Very Slight.