SOARBOND LTD.,

Structural Calculations
Drawings and Details for a
Redevelopment
at

207, SUMATRA ROAD,

WEST HAMPSTEAD, LONDON NW 6 1 PF.

FOR MS. KERRY HAMILTON.

JOB No. 1381 JUNE 2015.

"A" REVISION 12. SEPT. 2016.

SOARBOND LTD.,
17 Clarendon Road, London W5 1AA, tel. 0208 997 8663

Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6			Job ref. 1381	
Part of structure Design parameters			Calc sheet No rev. A / 1	
Drawing ref.	Calc by KZ	Date 2 JUNE 2015.	Check by	Date

Ref.	Calculations	Output
	DESIGN PARAMETERS	

1. All new timber to be used in areas, as designed is to have the following grading in accordance with BS5268, part 2, 2002 as varied from CP112 to which design shall be carried out - 18% moisture content not exceeded.
   Table 10 $p_{tb} = 7.5$ N/mm^2
   $E_{mean} = 10800$ N/mm^2
   $p_{t\ shear} = 0.71$ N/mm^2
   $p_{tt} = 4.5$ N/mm^2
   $p_{tc} = 7.9$ N/mm^2

Old timber $p_{tb} = 7.5$ N/mm^2
$E_{min} = 7200$ N/mm^2

Output: Hemfir selected C24 timber or equivalent.

Where main members are required and loading for long term at max. design loads does not occur. Use factors:-

K3	= 1.25 snow + dead	
K3	= 1.00 floors	
K4	= 1.1 bearing	
K7 $(300 / 200)^{0.11}$	= 1.046	- 200 depth
K7 $(300 / 225)^{0.11}$	= 1.032	- 225 depth
K8	= 1.10 load share	

2. Structural steelwork, as simply supported beams and portals, is to be designed plastically to BS5950:1985 for all steel to BS4360 as varied by euro codes grade 43A unless so specified on drawings.

3. Brickwork and blockwork to be in mortar designation (iii) with facing bricks / flettons having a minimum strength of 27.5 N/mm^2, whereas light-weight blocks to be 7N/mm^2 compressive stress strength, brickwork overall stress capacity f_k, to be 7.9N/mm^2. Blockwork overall stress capacity, f_k, to be 5.4N/mm^2 and partial safety factor taken as 3.1 in both cases. For check onto old brickwork, general stress capacity in compression to be assumed as 0.42N/mm^2 net increase from new build, unless noted otherwise.

Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6			Job ref. 1381		
Part of structure Design parameters			Calc sheet No rev. A / 2		
Drawing ref.	Calc by KZ	Date JUNE 2015.	Check by	Date	

Ref.	Calculations	Output
	4. Concrete to be designed to BS8110:1985, part 1. Concrete to be C35 / 20mm agg at 28 days with:- $f_{cu} = 35$ N/mm^2 $f_q = 0.35$ N/mm^2 -say no reinforcement. Cover to earth faces 40mm min whereas to other faces use 25mm minimum. All reinforcement to be high yield noted as H.Y., (designation "T") with $f_y = 460$ N/mm^2 5. Allowable ground bearing capacity of soil assumed to be 110 kN/m^2 at 900mm depth. Value to be checked in strip footing excavation before concreting. Depth of mass concrete below 900mm invert to requirements of L.A. For further requirements see relevant drawing.	

Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6		Job ref 1381	
Part of structure Loadings		Calc sheet No B / 1	rev.
Drawing ref.	Calc by KZ	Date JUNE 2015.	Check by	Date

Ref.	Calculations	Service load 1.0 GK	Service load 1.0 QK	Ultimate load 1.4 GK	Ultimate load 1.6 QK
	TYPICAL ROOF LOADING				
	Dead load:				
	Concrete tiles	0.50			
	Sheathing felt	0.10			
	Battens 20 x 10 @150				
	0.02 x 0.01 x 5.9 / 0.15	0.01			
	Insulation	0.01			
	Joists 200 x 50 @ 400				
	0.20 x 0.05 x 5.9 / 0.40	0.15			
	Total	0.77		1.07	
	Live load:				
	Snow @ 30° - no access		0.75		1.20
		1.52 kN / m^2		2.27 kN / m^2	
	TYPICAL CEILING LOADING				
	Dead load:				
	Joists 100 x 50 @ 400				
	0.10 x 0.05 x 5.9 / 0.40	0.07			
	Lathe and plaster 18mm - say				
	0.018 x 18 x 0.9	0.29			
	Services - say	0.10			
	Total	0.46		0.64	
	Nominal live load:		0.25		0.40
		0.71 kN / m^2		1.04 kN / m^2	
	225mm SOLID WALL				
	Dead load:				
	BWK 0.225 x 17	3.83			
	Plaster 2 sides – say 2 x 0.012 x 18	0.43			
	Services - nominal	0.05			
		4.31 kN / m^2		6.03 kN / m^2	

Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6			Job ref. 1301	
Part of structure Loadings			Calc sheet No B / 2	rev.
Drawing ref.	Calc by KZ	Date June 05.	Check by	Date

Ref.	Calculations		Service load 1.0 GK	Service load 1.0 QK	Ultimate load 1.4 GK	Ultimate load 1.6 QK
	CAVITY WALL LOADING					
	Dead load:					
	Outer skin face brick	0.103 x 17	1.75			
	Inner blockwork	0.1 x 9	0.90			
	Insulation - say		0.02			
	Plaster inner skin	0.012 x 18	0.22			
	Services - say		0.05			
	Total		2.94 kN / m^2		4.12 kN / m^2	
	FLOOR - SUSPENDED FIRST					
	Dead load:					
	Carpet - say		0.05			
	Hardboard	0.005 x 5.9	0.03			
	T+G floor boards	0.02 x 5.9	0.12			
	Joists 200 x 50 @ 400	0.20 x 0.05 x 5.9 / 0.40	0.15			
	Plasterboard + skim under	0.013 x 18	0.23			
	Services - say		0.05			
	Total		0.64		0.89	
	Live load:			1.50		2.40
			2.14 kN / m^2		3.29 kN / m^2	
	GROUND BEARING CONCRETE GROUND FLOOR					
	Dead load:					
	Carpet		0.05			
	Hardboard	0.005 x 5.9	0.03			
	T+G floor boards + battens	(0.020 + 0.02) x 5.9	0.13			
	Insulation		0.01			
	200 R.C. slab	0.20 x 24	4.80			
	50 blinding	0.05 x 24	1.20			
	Total		6.22		8.71	
	Live load:			1.50		2.40
			7.72 kN / m^2		11.11 kN / m^2	

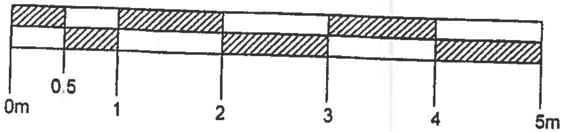
Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6			Job ref. 1381	
Part of structure Loadings			Calc sheet No rev. B / 3	
Drawing ref.	Calc by KZ	Date June 2015.	Check by	Date

Ref.	Calculations	Service load		Ultimate load	
	CHEEKS	1.0 GK	1.0 QK	1.4 GK	1.6 QK
	Dead load:				
	Clay tiles	0.50			
	Battens 20 x 12 @ 150				
	0.02 x 0.012 x 5.9 / 0.15	0.01			
	Felt	0.17			
	Verticals @ 600				
	0.15 x 0.05 x 5.9 / 0.6	0.07			
	Insulation 150mm	0.02			
	Plasterboard 0.012 x 18	0.22			
	Services - say	0.05			
	Total	1.04 kN / m^2		1.45 kN / m^2	
	150 LIGHTWEIGHT BLOCKWORK WALL				
	Dead load:				
	120 blockwork -				
	Breeze or similar 0.12 x 13	1.56			
	Render one side 15mm 0.015 x 24	0.36			
	Plaster second side 0.012 x 18	0.22			
	Services - nominal say	0.05			
	Total	2.19 kN / m^2		3.07 kN / m^2	
	PARTITIONS				
	Dead load:				
	Head and Sole plates				
	2 x 0.1 x 0.05 x 5.9 / 3.0m	0.02			
	Verticals @ 600				
	0.1 x 0.05 x 5.9 / 0.6	0.05			
	Insulation	0.01			
	Services - say	0.10			
	Plasterboard 2 x 0.012 x 18	0.43			
	Total	0.61 kN / m^2		0.86 kN / m^2	

Project 207 SUMATRA ROAD, WEST HAMPSTEAD, NW6		Job ref. 1381	
Part of structure Loadings		Calc sheet No B / 4	rev.
Drawing ref.	Calc by KZ / Date JUNE 2015.	Check by	Date

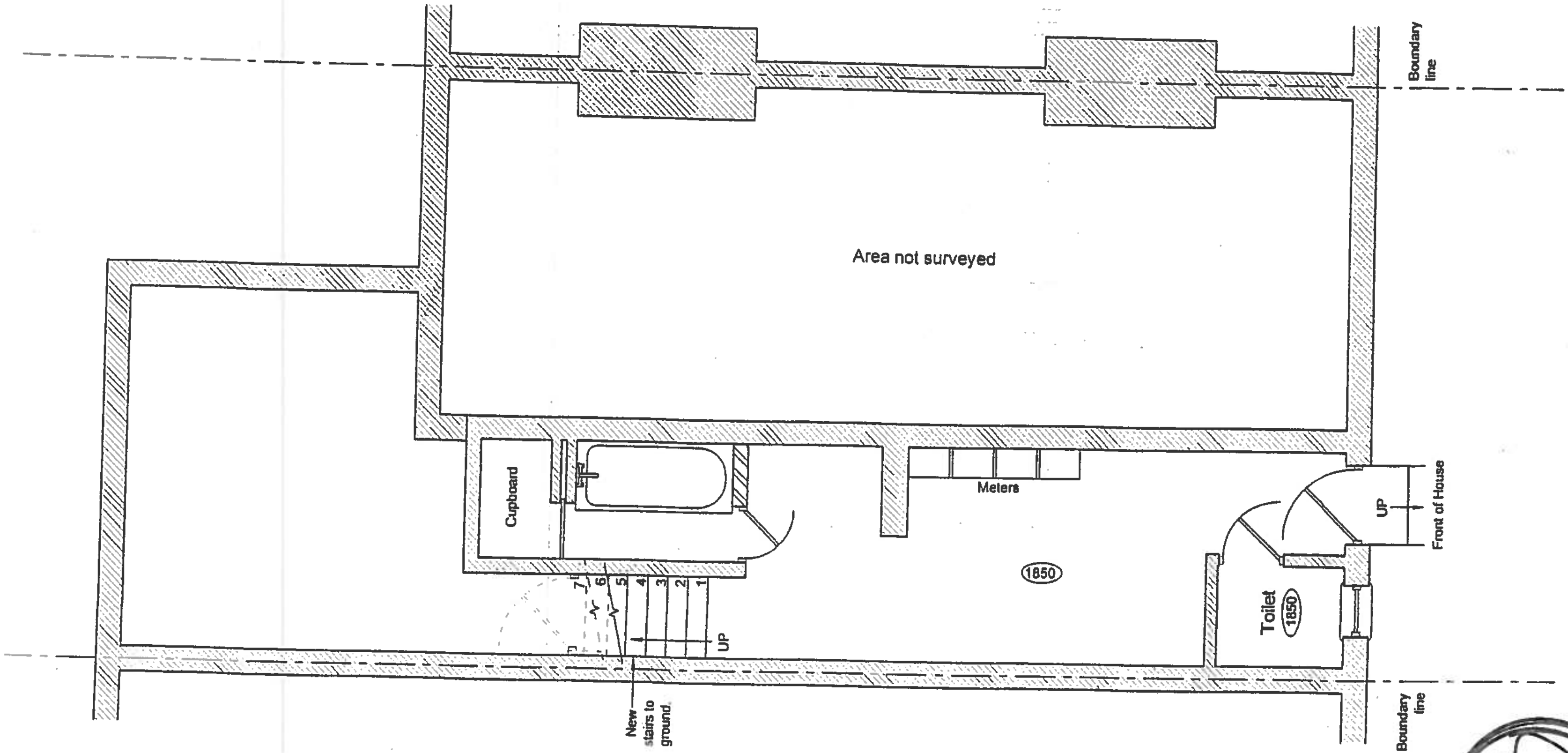
Ref.	Calculations	Service load 1.0 GK	Service load 1.0 QK	Ultimate load 1.4 GK	Ultimate load 1.6 QK
	FLAT ROOF				
	Dead load:				
	3 layers hot bound bituminous felt	0.45			
	Sheathing felt	0.10			
	18mm Marine ply 0.018 x 7.9	0.14			
	Joists - 150 x 50 SW @ 400 CRS	0.04			
	Insulation 100mm	0.01			
	Services - say	0.05			
	Plasterboard 0.013 x 18	0.23			
	Total	1.02		1.43	
	Live load: (no access)		0.75		1.2
	Total	1.77 kN / m^2		2.63 kN / m^2	
	TYPICAL SUSPENDED CONCRETE FLOORS				
	Dead load:				
	175mm concrete slab 0.175 x 24	4.20			
	Services - say	0.10			
	False ceiling under - say	0.10			
	75mm screed	1.80			
	Insulation - say	0.01			
	Timber battens 0.02 x 0.01 x 5.9 / 0.15	0.01			
	Flooring 0.02 x 5.9	0.12			
	Total	6.34		8.88	
	Live load: (domestic)		1.5		2.4
	Total	7.84 kN / m^2		11.28 kN / m^2	
	100 BLOCKWORK WALLS				
	Dead load:				
	100 LWT block 0.1 x 9	0.90			
	10mm plaster – both sides 2 x 0.01 x 18	0.36			
	Services - say	0.05			
	Total	1.31 kN / m^2		1.84 kN / m^2	

Scale 1: 50 @A3



Notes:

1. This drawing to be read in conjunction with all other Architect's drawings, specification given on 09, and all other structural, electrical and mechanical drawings.
2. See elevations for details of windows and doors.
3. All new works to outside of existing house to match existing materials exactly.



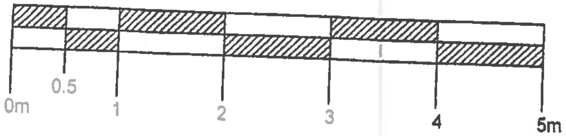
Existing Basement Layout

Scale 1:50 @A3

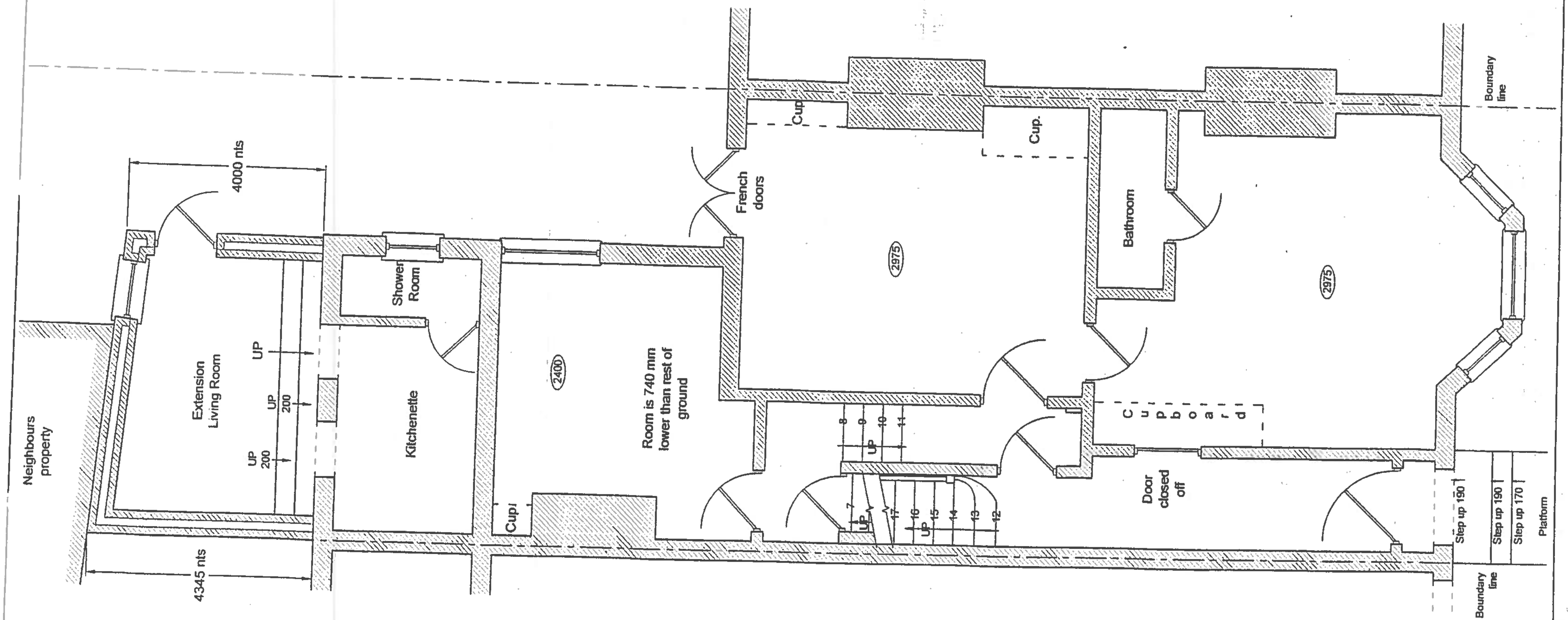
C13

SOARBOND Ltd.	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Existing Basement Layout	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zoblocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 01	

Scale 1: 50 @A3



Notes:
1. For general notes see drawing 1381 - 01.



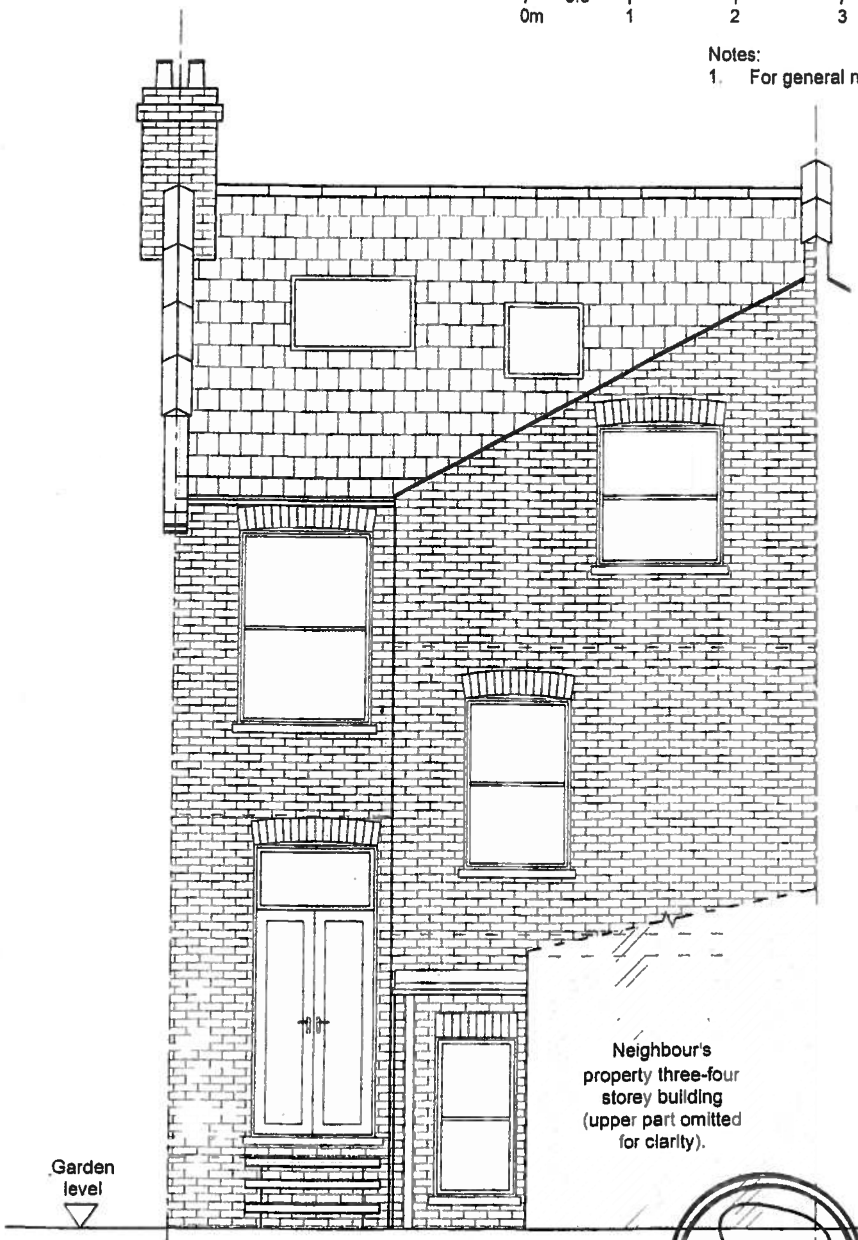
Existing Ground Floor Layout
Scale 1:50 @A3

SOARBOND Ltd. Chartered Engineers 17 Clarendon Road, Ealing, London W5 1AA; tel 020 8997 5663; mobile 07879 863 988; email soarbond@yahoo.co.uk	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Existing Ground Floor Layout	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zoblocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 02	

Scale 1:50 @A3

Notes:
1. For general notes see drawing 1381 - 01.

Existing Front Elevation
Scale 1:50 @A3



Existing Rear Elevation
Scale 1:50 @A3

C151

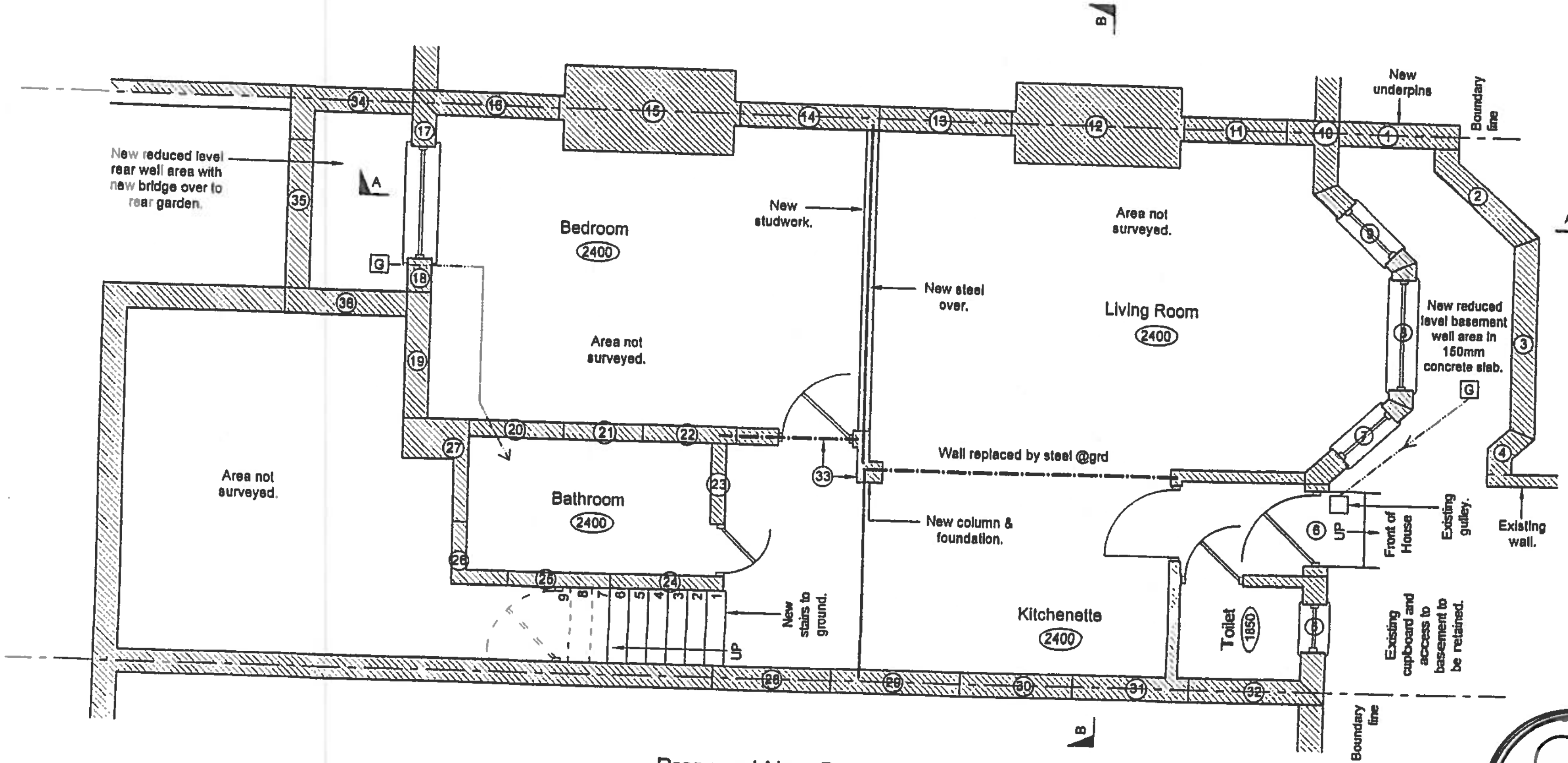
SOARBOND Ltd.

Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Existing Front Elevation & Existing Rear Elevation	
Professor Kerry Hamilton	
P. Obrzut	August 2017
W.K.J. Zablocki	August 2017
1381	1 : 50
1381 - 07	

Scale 1: 50 @A3

0m 0.5 1 2 3 4 5m

Notes:
1. This drawing to be read in conjunction with all other Architect's drawings, specification given on 09, and all other structural, electrical and mechanical drawings.
2. See elevations for details of windows and doors.
3. All new works to outside of existing house to match existing materials exactly.
4. Underpins shown thus 35



Proposed New Basement Layout
Scale 1:50 @A3

SOARBOND Ltd.

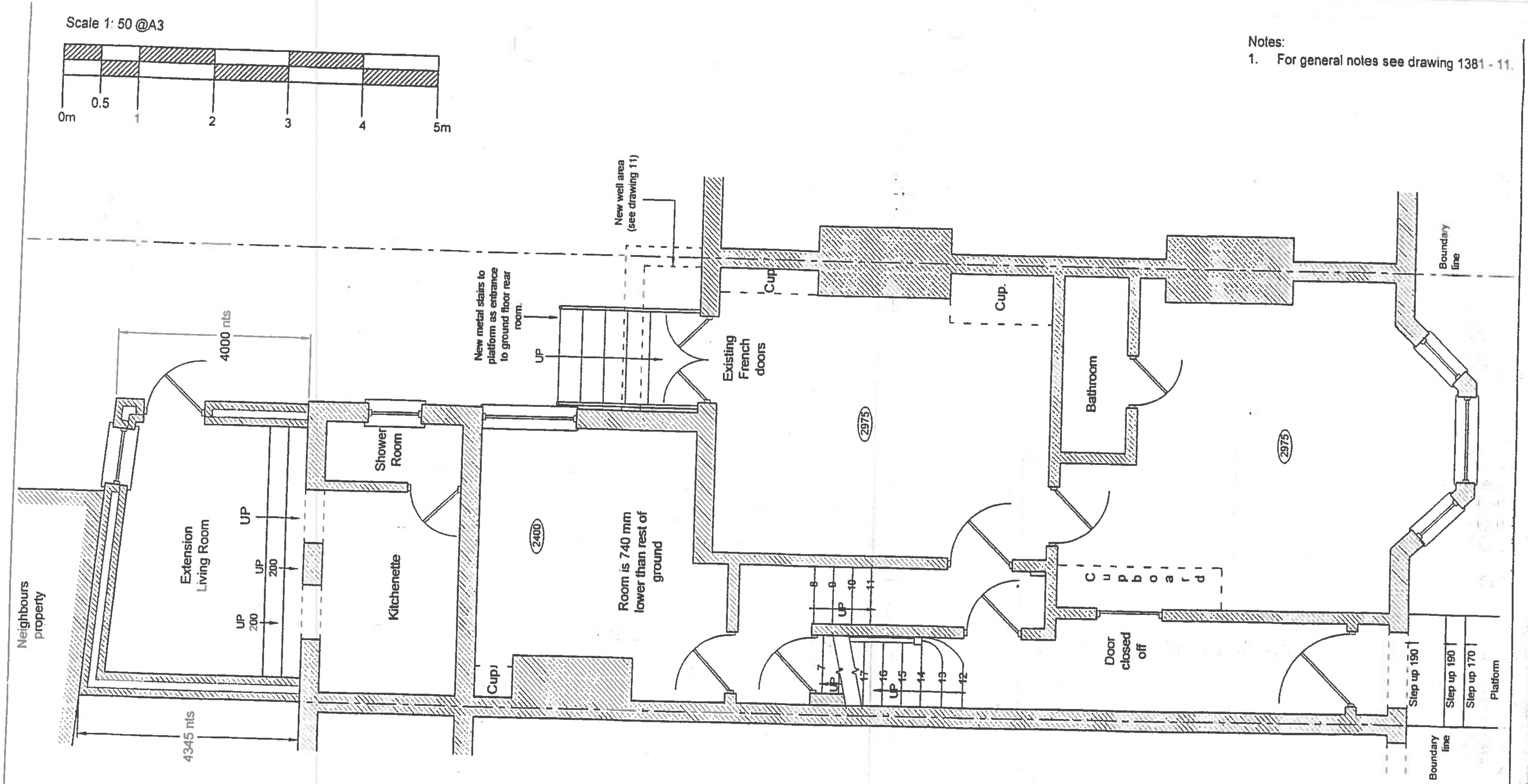
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF

Drawing title: Proposed Basement Layout

Client: Professor Kerry Hamilton

Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 – 11	

Notes:
1. For general notes see drawing 1381 - 11.



Proposed Ground Floor Layout
Scale 1:50 @A3

SOARBOND Ltd.
Chartered Engineers
17 Gerandon Road, Ealing, London W5 1AA
tel 020 8997 6863
mobile 07879 883 883

Job title
Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF

Drawing title
Proposed Ground Floor Layout

Client
Professor Kerry Hamilton

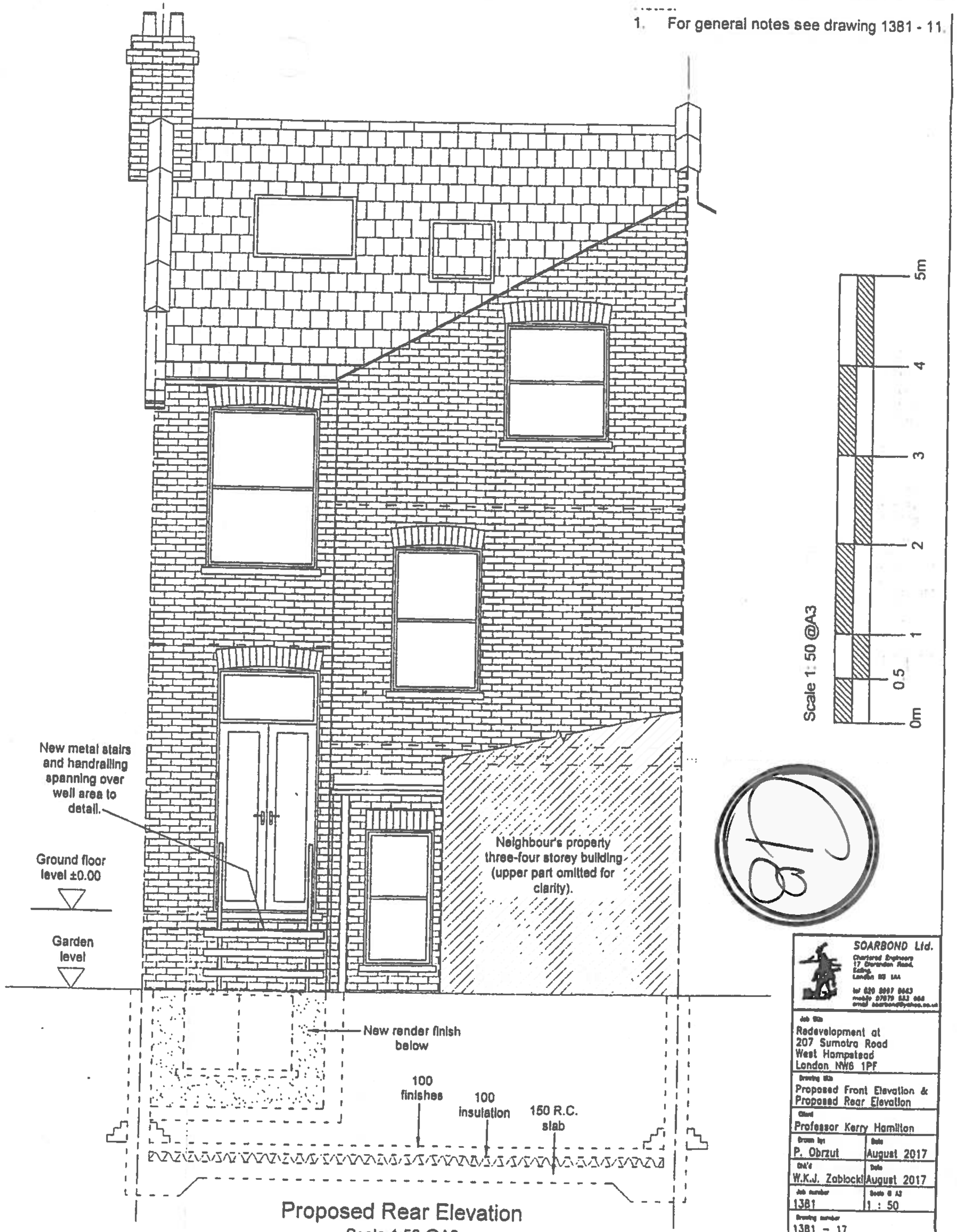
Drawn by	Date
P. Obrzut	August 2017
Chk'd	Date
W.K.J. Zablocki	August 2017
Job number	Scale @ A3
1381	1 : 50

Drawing number
1381 - 12

C7

1. For general notes see drawing 1381 - 11.

Proposed Front Elevation
Scale 1:50 @A3



Proposed Rear Elevation
Scale 1:50 @A3

SOARBOND Ltd.

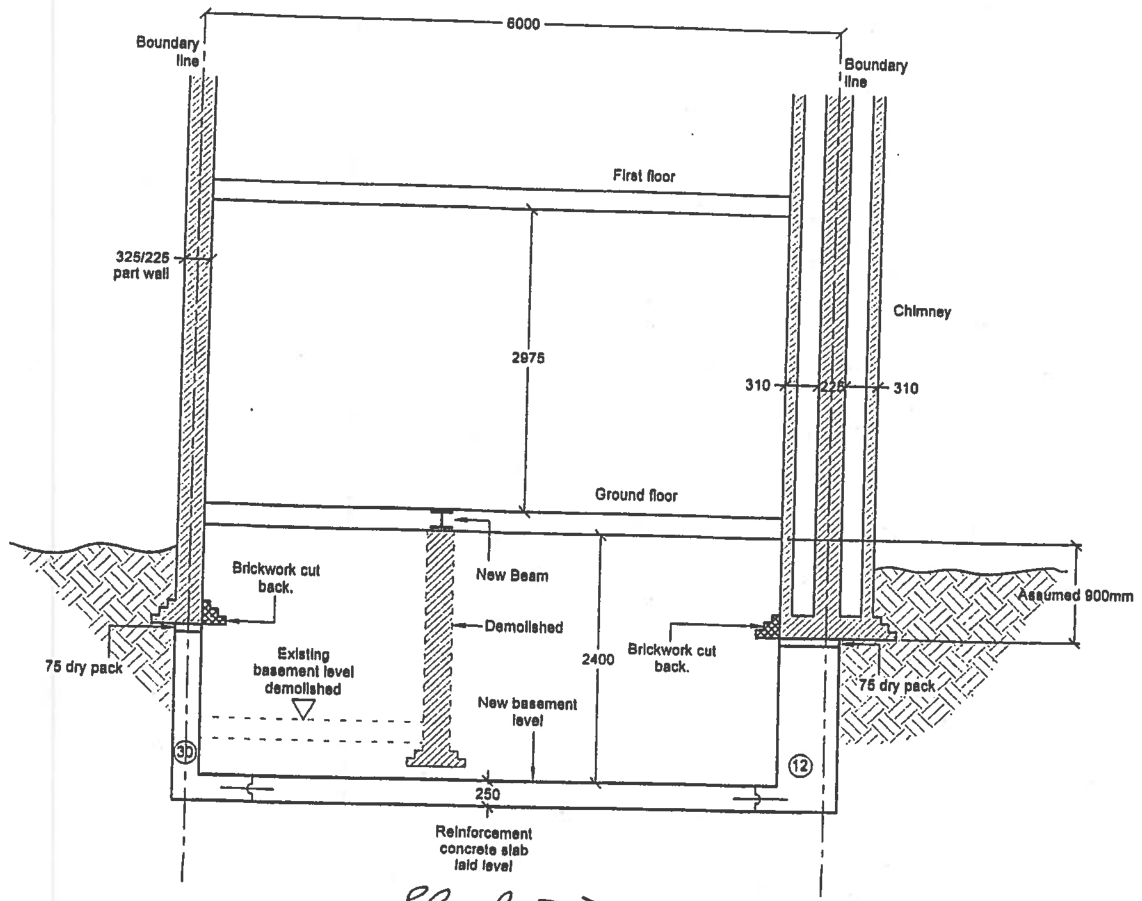
Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF

Proposed Front Elevation & Proposed Rear Elevation

Professor Kerry Hamilton

P. Obrzut	August 2017
W.K.J. Zablocki	August 2017
1381	1 : 50
1381 - 17	

C18



PROPOSED.

Section B-B

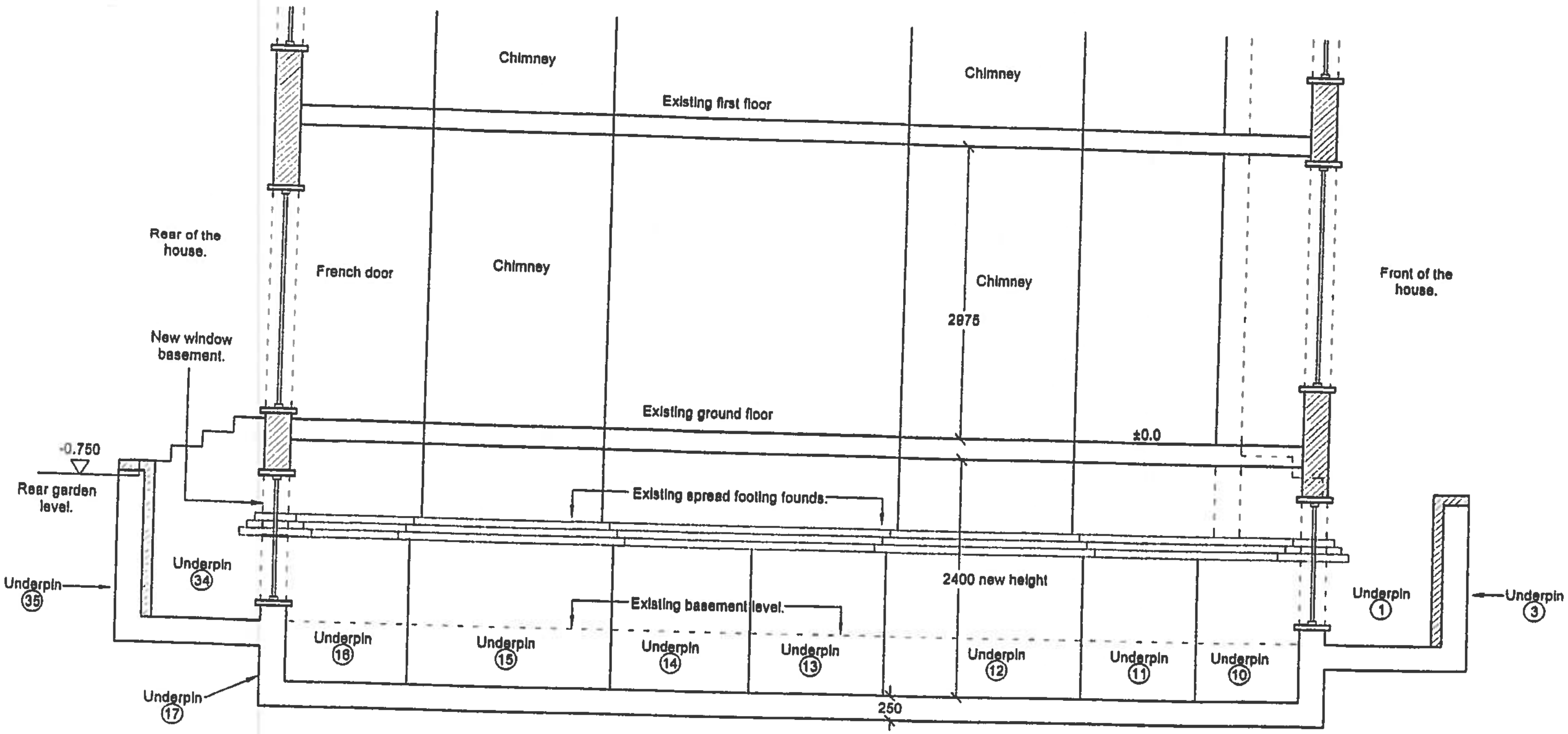
Scale 1:50 @A3

A REV K2
V5/2018
MARKED AS
PROPOSED BB

SOARBOND Ltd.	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Proposed Basement SECTION B-B	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 31	

A.

G9



Proposed Section A-A

Scale 1:50 @A3

SOARBOND Ltd.

Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF

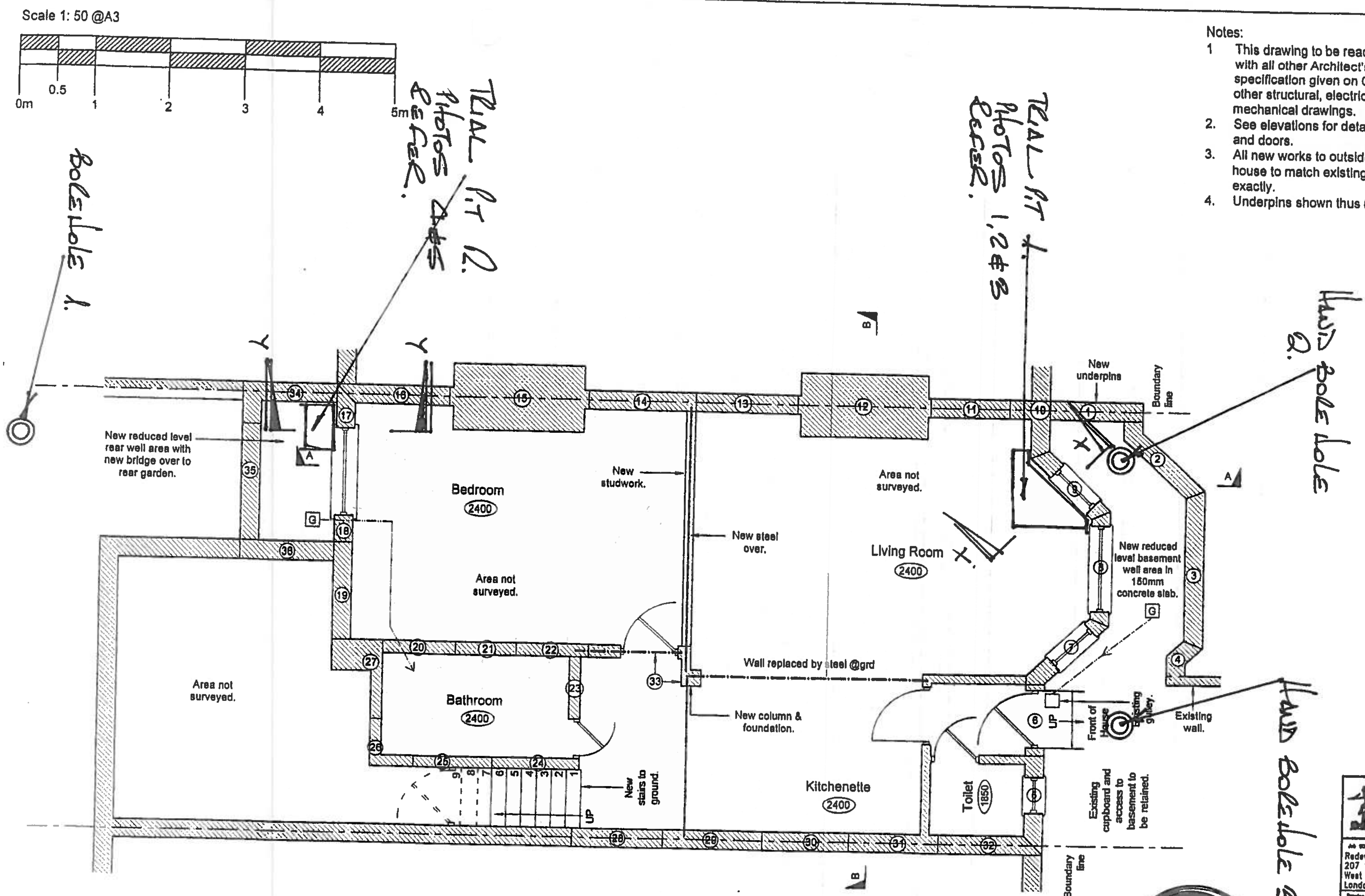
Drawing title: Proposed Section A-A

Client: Professor Kerry Hamilton

Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zoblocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 32	

Notes:

1. This drawing to be read in conjunction with all other Architect's drawings, specification given on 09, and all other structural, electrical and mechanical drawings.
2. See elevations for details of windows and doors.
3. All new works to outside of existing house to match existing materials exactly.
4. Underpins shown thus (35)



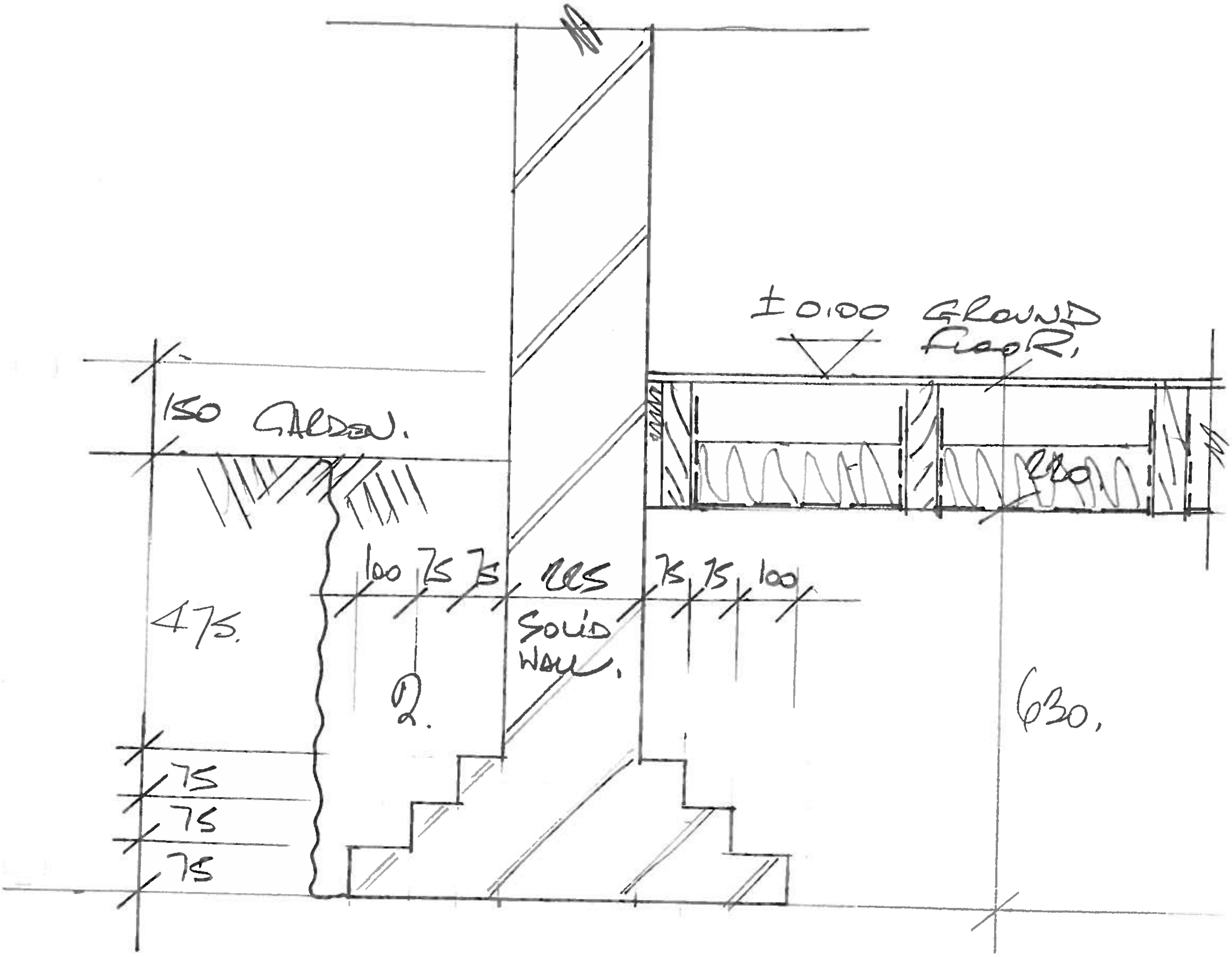
Proposed New Basement Layout

Scale 1:50 @A3

C21

SOARBOND Ltd.	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Proposed Basement Layout	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zoblocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 – 11	

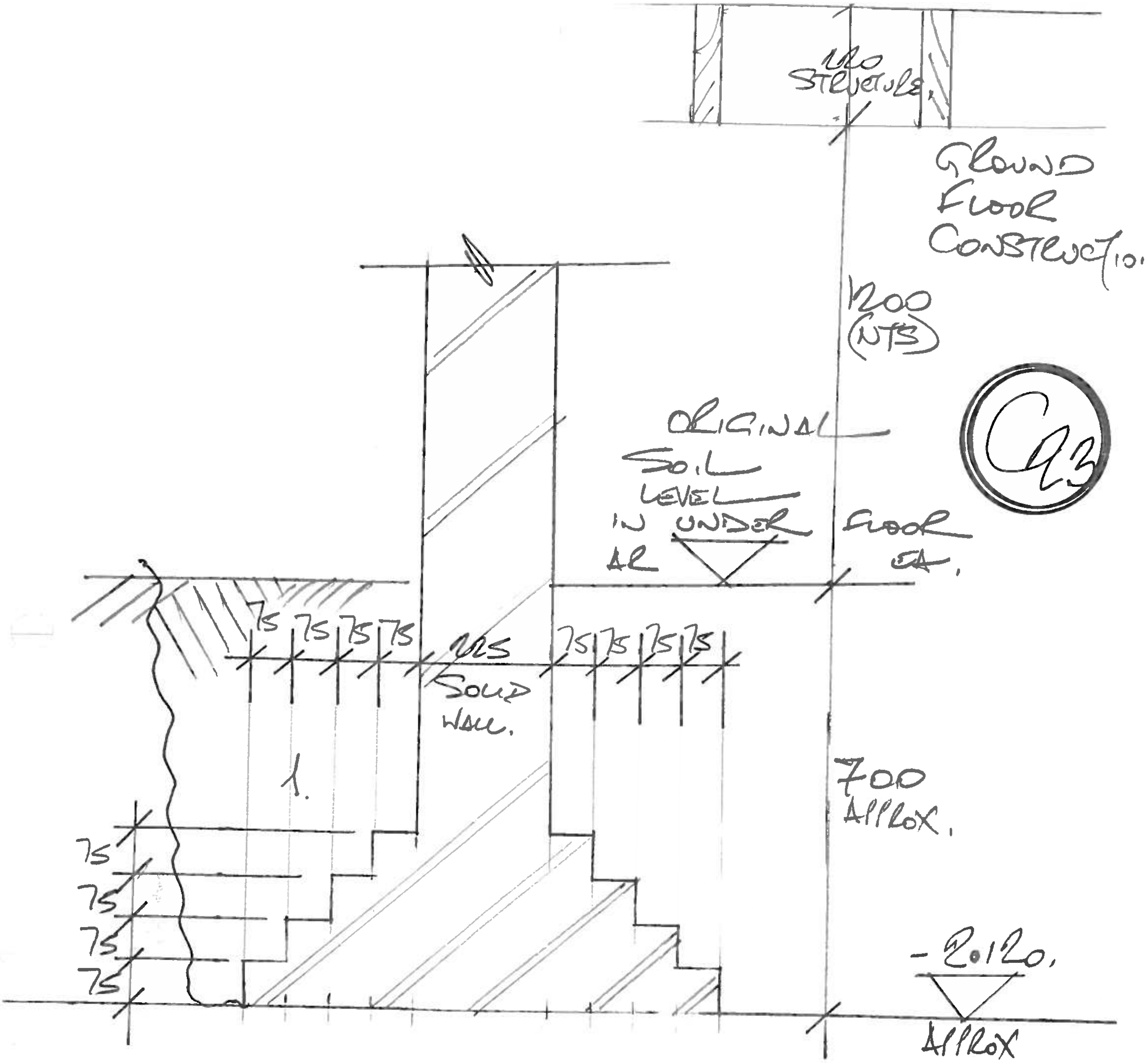
C22



SECTION Y-Y @ 1:10.

SHOWING EXISTING HOUSE FOUNDATIONS (PHOTOS 4+5 LATER)

207 SUMATRA RD.,
WEST HAMPSTEAD,
LONDON NW6 1PF
B001 / KZ 24.09.18/2



CA3

SECTION X-X @ 1:10

SHOWING HOUSE FOUNDATIONS (PHOTOS 1, 2, 3 REFER).

207 SUMATRA ROAD,
WEST HAMPSTEAD,
LONDON NW6 1PF

1381 / KZ 24.09.18/1

Project 207 SUMATRA RD, NW6			Job ref 1381		
Part of structure BASEMENT DESIGN			Calc sheet no E1 / 1	rev	
Drawing ref	Calc by WL	Date SEPT 2013.	Check by	Date	

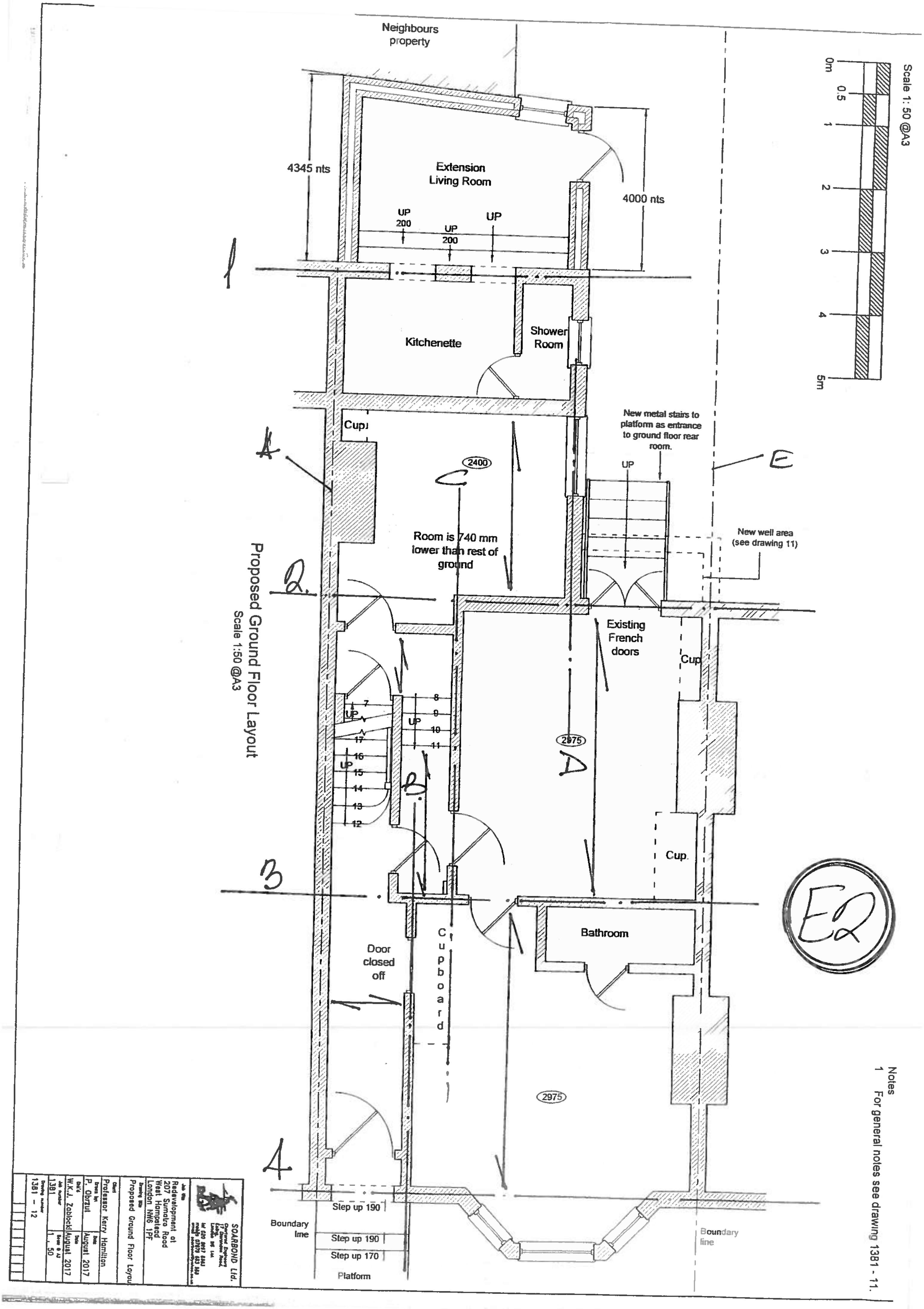
Ref	Calculations	Output

THE FOLLOWING CALCULATIONS HAVE BEEN PREPARED TO SHOW THAT THE REINFORCED CONCRETE, "U" FRAMED BASEMENT WILL CARRY THE VERTICAL LOADING AND NOT SUFFER MAJOR (>2mm) SIDE MOVEMENT BASED ON HORIZONTAL SOIL ACTIVE PRESSURE ETC.

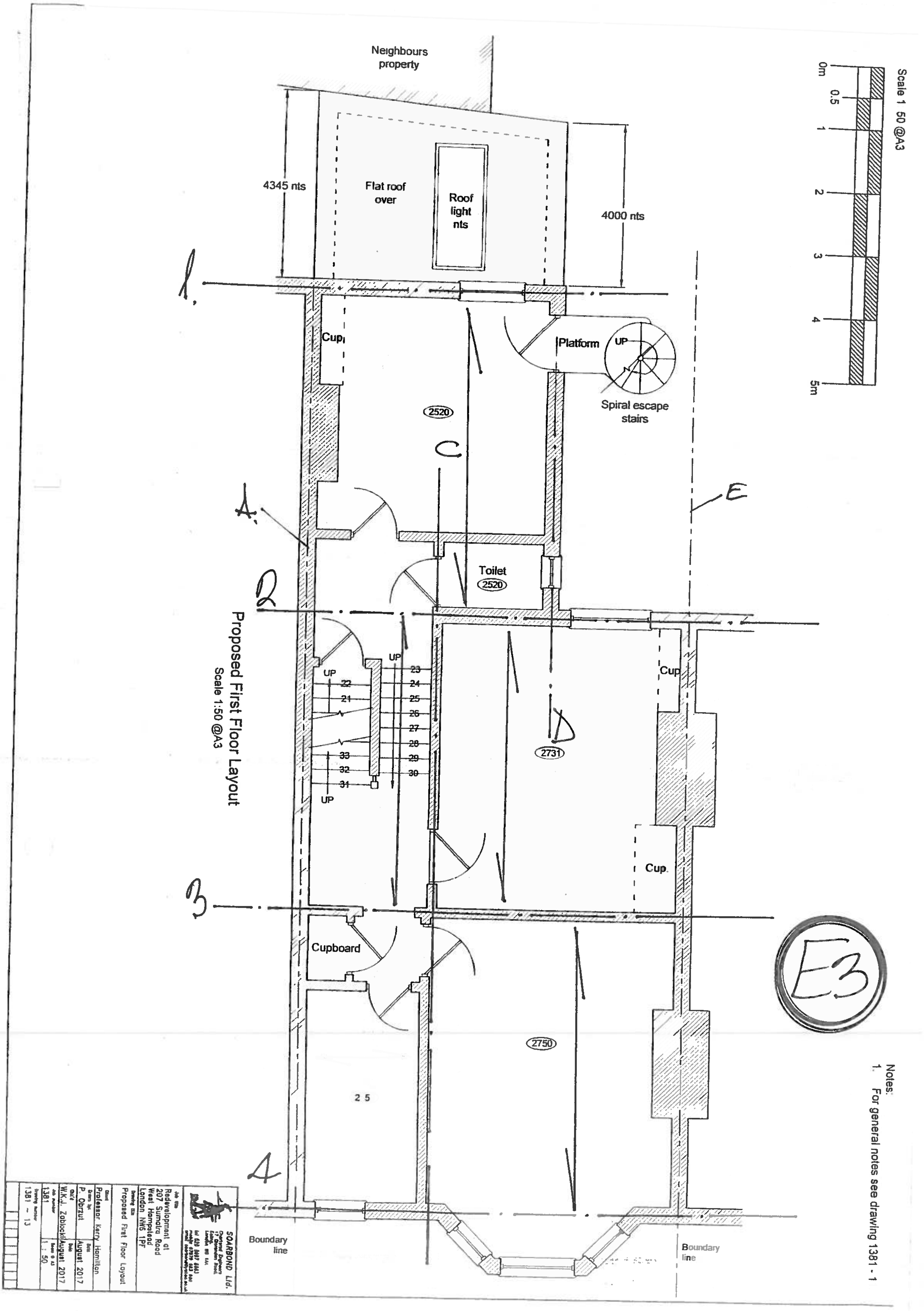
TOTAL HOUSE LOAD — LOAD RUN DOWN.

DRAWING SHOWN ON CIB GIVES EXTERNAL ELEVATIONS

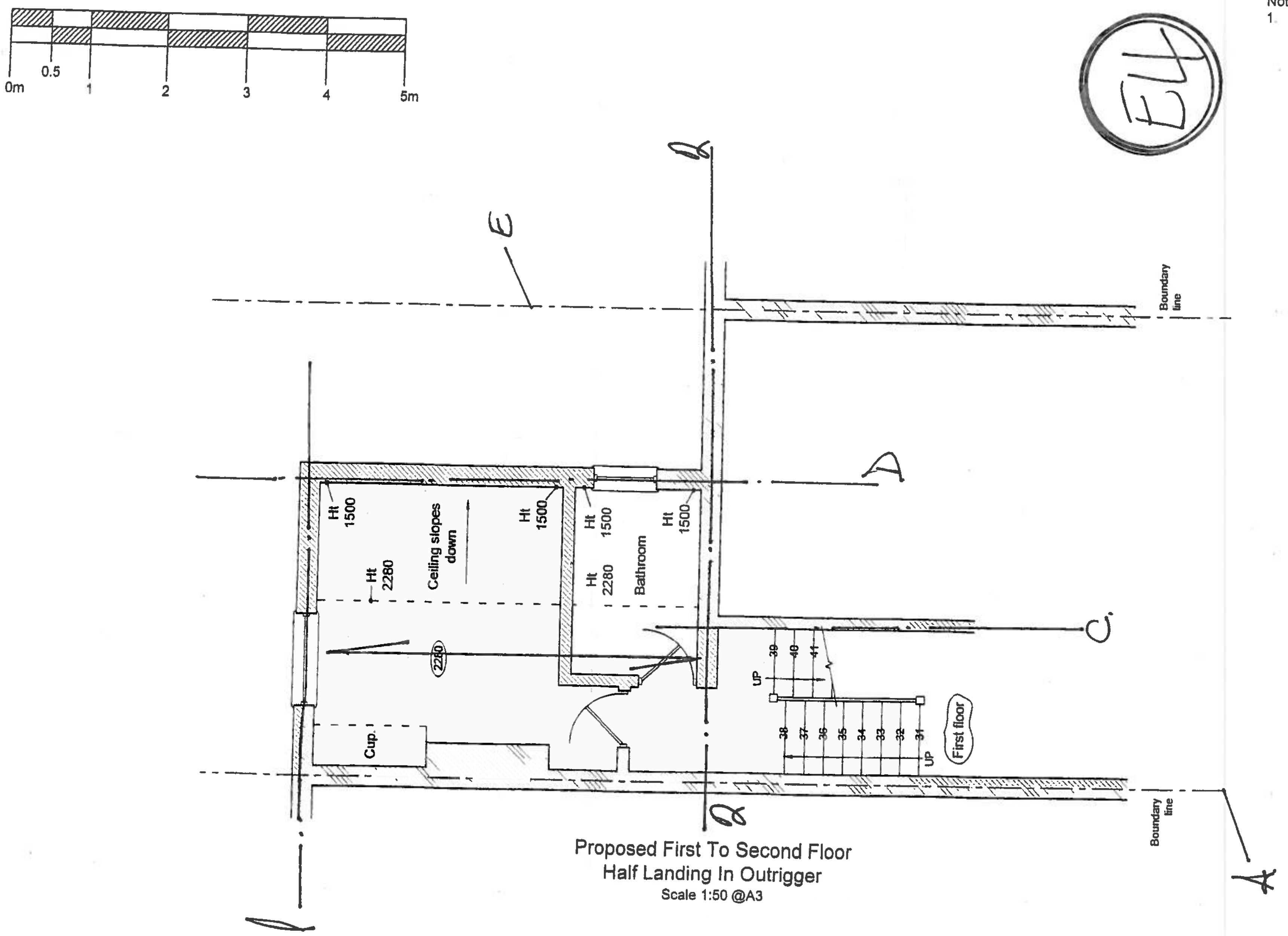
DRAWINGS E2 TO E6 GIVES PLAN LAYOUTS FROM GROUND TO ROOF. THEY HAVE BEEN MARKED UP WITH GRIDS, ASSUMED FLOOR SPANS TO GIVE LOADING RUN DOWN ETC.



Neighbours property
Extension Living Room
4345 nts
4000 nts
UP 200
UP 200
UP
Kitchenette
Shower Room
Cup
New metal stairs to platform as entrance to ground floor rear room.
UP
2400
Room is 740 mm lower than rest of ground
New well area (see drawing 11)
Existing French doors
Cup
2975
Cup.
Bathroom
Door closed off
Cupboard
2975
Step up 190
Step up 190
Step up 170
Platform
Boundary line
Boundary line
A
B
C
D
E
1
2
3
4
E2
Proposed Ground Floor Layout
Scale 1:50 @A3
Scale 1: 50 @A3
0m 0.5 1 2 3 4 5m
Notes
1 For general notes see drawing 1381 - 11.
SOARBOND Ltd.
Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF
Proposed Ground Floor Layout
Professor Kerry Hamilton
P. Obrzut
August 2017
W.K.J. Zoblocki
August 2017
1381
1 : 50
1381 - 12


Neighbours property
4345 nts
Flat roof over
Roof light nts
4000 nts
Cup
Platform
UP
Spiral escape stairs
2520
C
A.
E
Toilet
2520
2
UP
UP
22
21
23
24
25
26
27
28
29
30
33
32
31
UP
Cup
D
2731
Cup.
3
Cupboard
2750
2 5
4
Boundary line
Boundary line
1.
Scale 1 50 @A3
0m
0.5
1
2
3
4
5m
Proposed First Floor Layout
Scale 1:50 @A3
Notes:
1. For general notes see drawing 1381 - 1
SOARBOND Ltd.
Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF
Proposed First Floor Layout
Professor Kerry Hamilton
P. Obrzut
August 2017
W.K.J. Zoblocki/August 2017
1381
1 : 50
1381 - 13
E3

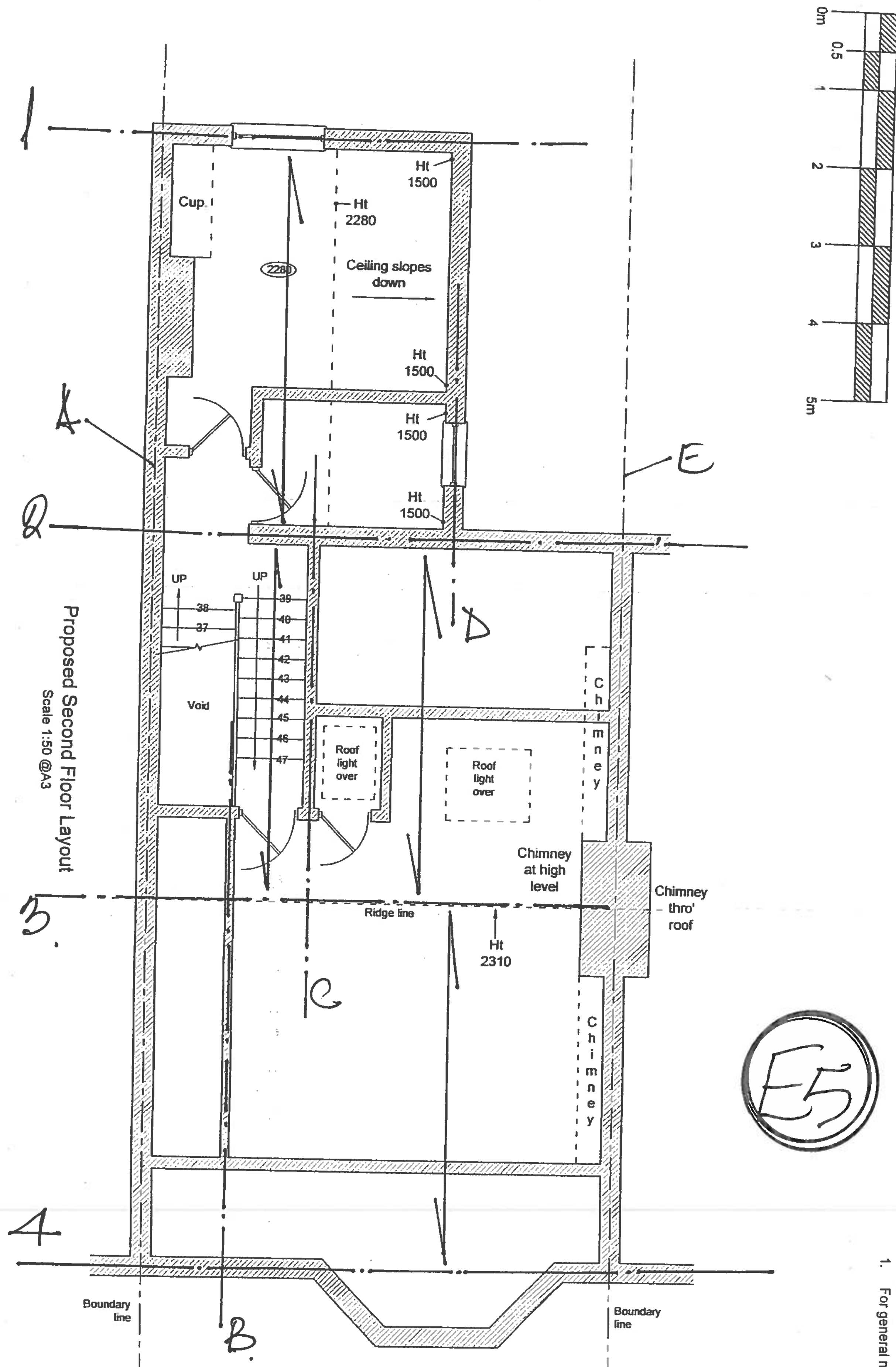
Scale 1: 50 @A3



Proposed First To Second Floor
Half Landing In Outrigger
Scale 1:50 @A3

Notes:
1. For general notes see drawing 1381 - 11.

SOARBOND Ltd. Chartered Engineers 17 Clarendon Road, Ealing, London W5 1AA tel 020 8997 8963 mobile 07879 883 885	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Proposed First to Second Floor Half Landing In Outrigger	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 14	



Proposed Second Floor Layout
Scale 1:50 @A3

E5

Notes:
1. For general notes see drawing 1381 - 11.

SOARBOND Ltd.

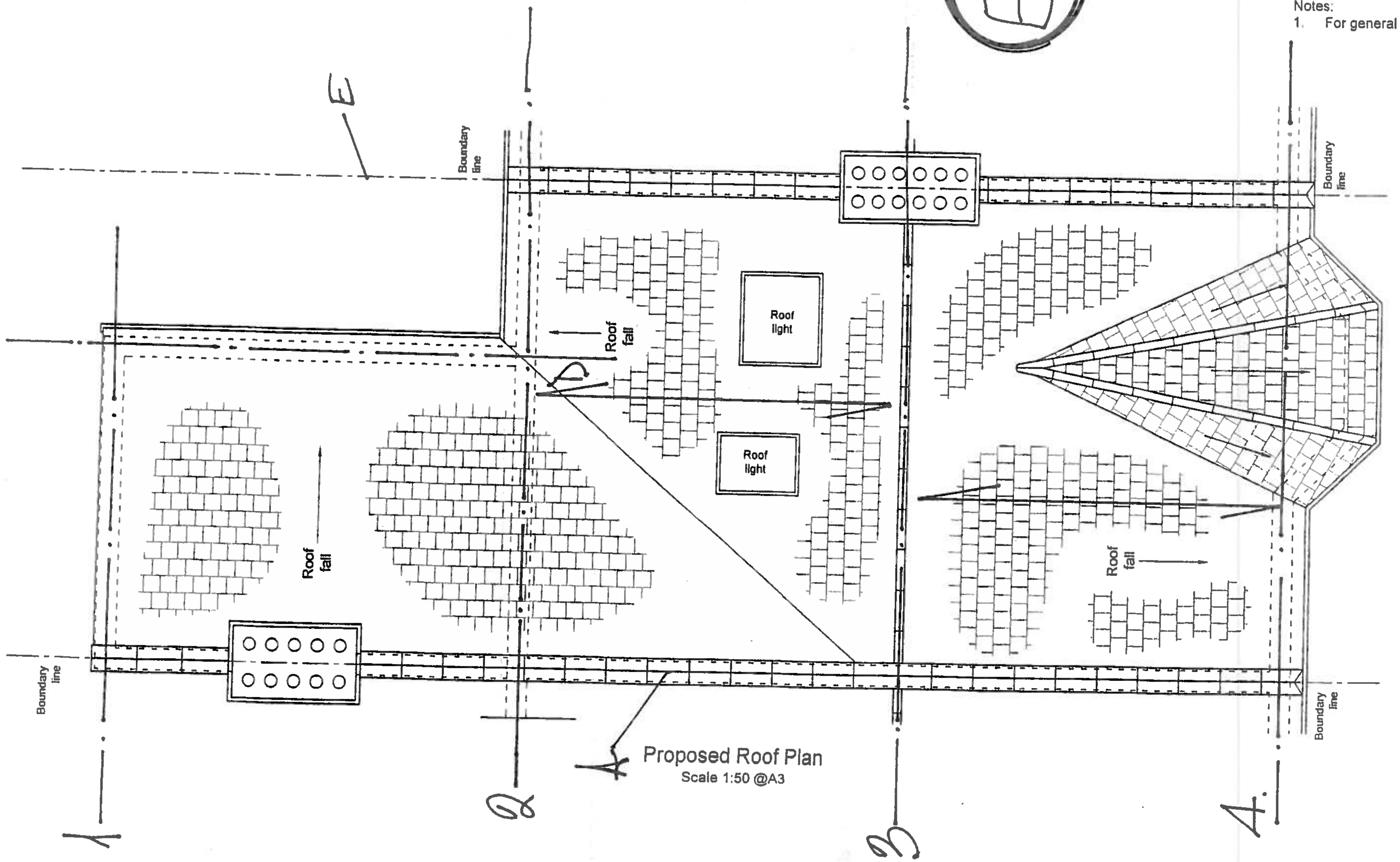
Redevelopment at
207 Sumatra Road
West Hampstead
London NW6 1PF

Proposed Second Floor Layout

Professor Kerry Hamilton

P. Obrzut	August 2017
W.K.J. Zabłocki	August 2017
1381	1 : 50

1381 - 15



Proposed Roof Plan
Scale 1:50 @A3

Scale 1: 50 @A3

0m 0.5 1 2 3 4 5m

Notes:
1. For general notes see drawing 1381 - 11

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Job title
Redevelopment at
207 Sumatra Road
West Hampstead
London NW6 1PF

Drawing title
Proposed Roof Plan

Client
Professor Kerry Hamilton

Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 - 16	

Project 207 Sumatra Road, NW6			Job ref 1381	
Part of structure Basement Design			Calc sheet no E1 7	rev
Drawing ref	Calc by WL	Date Oct 2020	Check by	Date

Ref	Calculations	Output
	Length = 6.1m	
E	Load Run Down on Line A2-E2	D L
E6	Roof 1+2/A-D	
	1.2m × 1.52 × 1.3 say = 2.37 kN/m	1.20 1.17
	2-3/A-E	
	(4.75/2) × 1.52 × 1.3 = 4.69 kN/m	2.38 2.31
E5	Second Floor	
	2-3 (4.75/2) × (2.14 + 0.25 stud) = 5.68 kN/m	2.11 3.57
	1-2 (5.15/2) × (2.14 + 0.25) 6.16 kN/m	2.29 3.87
	First Floor Ceiling	
	(4.75 + 5.15)/8 × 0.71 3.52 kN/m	2.28 1.24
E4	Part First - Second Mezzanine	
	(5.15/2) × (2.14 + 0.25) = 6.16 kN/m	2.29 3.87
E3	First Floor	
	2-3 (4.75/2) × 2.39 = 5.68 kN/m	2.11 3.57
	1-2 (5.15/2) × 2.39 = 6.16 kN/m	2.29 3.87
E2	Ground Floor	
	2-3 (3.23/2) × 2.39 = 3.86 kN/m	1.44 2.42
	1-2 (5.15/2) × 2.39 = 6.16 kN/m	2.29 3.87
	Stairs 2.07 × 0.5 × 2.14 × 1.3 × 1.5 / 6.15 = 0.70 kN/m	0.21 0.49
	Brickwork Founds to Second 80%	
	0.80 × 4.31 × 8.45m = 29.14 kN/m	29.14
	80.24 kN/m	49.99 30.25

Project 207 SUMATRA ROAD, NW6		Job ref 1881	
Part of structure BASEMENT DESIGN		Calc sheet no E4 B	rev 1
Drawing ref	Calc by WZ	Date OCT 2018	Check by	Date

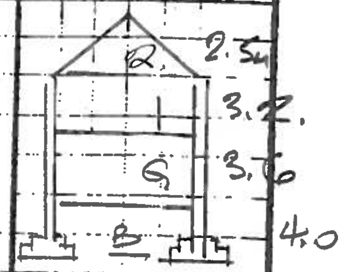
Ref	Calculations	Output
	LENGTH = 6.1 M	
	LOAD RUNDOWN ON LINE A3 - E3.	D L
E5	SECOND FLOOR $\left(\frac{4.72+4.72}{2}\right) \times 2.39$ = 11.27 kN/m	4.20 7.07
	FIRST FLOOR CEILING $\left(\frac{4.72+4.72}{2}\right) \times 0.71$ = 3.35 kN/m	2.17 1.18
E3	FIRST FLOOR $\left(\frac{4.72 \times 2}{2}\right) \times 2.39$ = 11.28 kN/m	4.20 7.08
E2	GROUND FLOOR $\left(\frac{4.72 \times 2}{2}\right) \times 2.39$ = 11.28 kN/m	4.20 7.08
	WALLING SAY 85%. $0.85 \times 4.31 \times 8.50$ kN* = 31.14 SAY kN/m	31.14
	* TO U/S OF BRICK WALL AT FOUNDATION LEVEL — 68.32 kN/m	43.91 22.41

Project	207 SUMATRA ROAD, NW6	Job ref	1381
Part of structure	BASEMENT DESIGN	Calc sheet no / rev	E1 9 / 1
Drawing ref		Calc by: W	Date: OCT 2010
Check by		Date	

Ref	Calculations	Output D	Output L
	LENGTH = 6.1 M		
	LOAD RUN DOWN ON LINE A4 – E4	D	L
E6	ROOF 3-4, $(4.75/2) \times 1.52 \times 1.3$ STAY $= 4.69$ kN/M	2.38	2.31
E5	SECOND FLOOR 3-4 $(4.75/2) \times (2.14 + 0.25_{\text{STUD}}) = 5.68$ kN/m	2.11	3.57
	FIRST FLOOR CEILING 3-4 $(4.75/2) \times 0.71 = 1.69$ kN/m	1.10	0.59
E3	FIRST FLOOR 3-4, $(4.75/2) \times (2.14 + 0.25) = 5.68$ kN/m	2.11	3.56
E2	GROUND FLOOR 3-4, $(4.75/2) \times (2.14 + 0.25) = 5.68$ kN/m	2.11	3.56
	WALLING 85% SAY FULL		
	$0.85 \times 4.31 \times (8.5 + 3.0_{\text{M}}) = 42.13$ kN/m	42.13	
	65.53	51.94	13.59
	kN/M.		

Project	207 SUMATRA RD., NW6	Job ref	1381
Part of structure	BASEMENT DESIGN	Calc sheet no / rev	E/10/1
Drawing ref	Calc by: WL	Date: OCT. 2018	Check by / Date

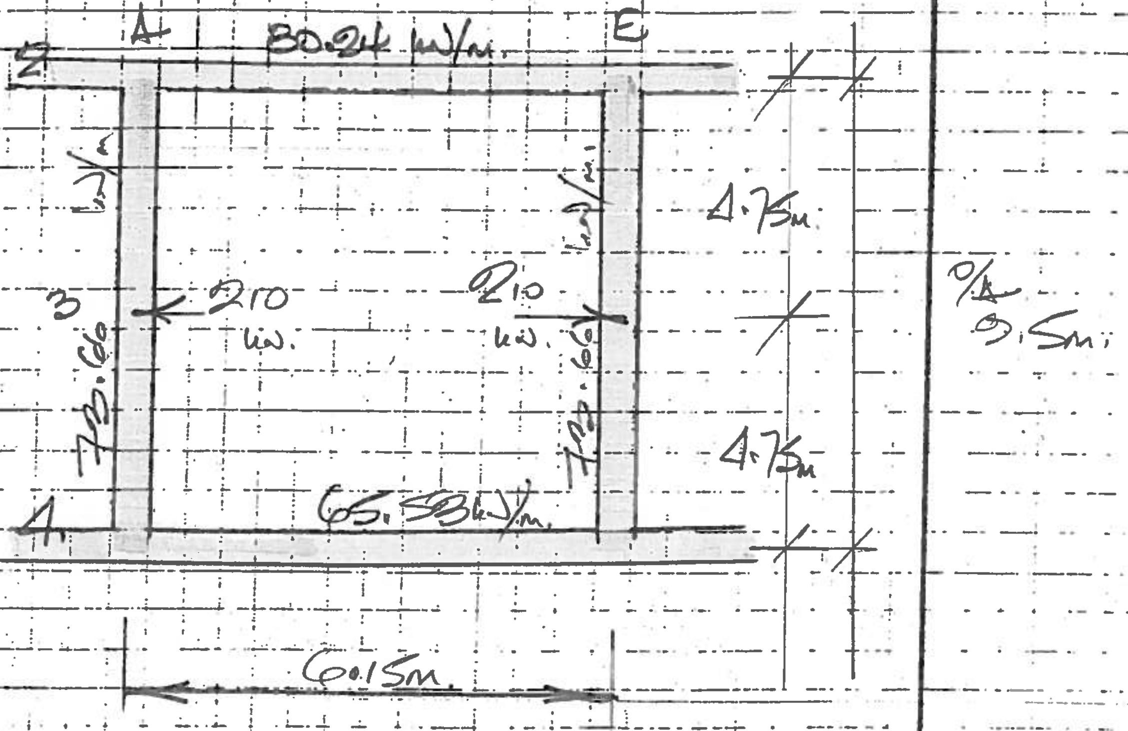
Ref	Calculations	Output	
	PARTY WALL LOAD RUNDOWN		
	SAY LOADINGS ARE SIMILAR		
	MAKE A2-A4 SIMILAR TO E2-E4.		
	ROOF LOADING	D	L
E6	2 × 1.2M SAY × 1.50 = 3.65 kN/m	1.85	1.80
E5	SECOND FLOOR 2 × 1.2M WIDTH × (2.14 + 0.25) = 5.74 kN/m	2.14	3.60
E3	FIRST FLOOR CEILING 2 × 0.6M WIDE × 0.71 = 0.85 kN/m	0.55	0.30
E3	FIRST FLOOR 2 × 1.2M SAY × 2.39 = 5.74 kN/m	2.14	3.60
E2	GROUND FLOOR 2 × 1.2M SAY × 2.39 = 5.74 kN/m	2.14	3.60
	WALLING (AVERAGE) (4.0 + 3.6 + 3.2 + 1.25) 4.31 = 51.94 kN/m	51.94	
	73.66 kN/m	60.76	12.90



Project	207 SUMATRA ROAD, NW6	Job ref	1381
Part of structure	BASEMENT DESIGN	Calc sheet no / rev	E, 11, 1
Drawing ref		Calc by	VP
Date	OCT 2018	Check by	
Date			

Ref	Calculations	Output

FROM LOAD RUN DOWNS ASSESS LOADING ONTO BASEMENT BOX AS:-



LOADINGS AT A3 (E3 SIMILAR) AT GROUND

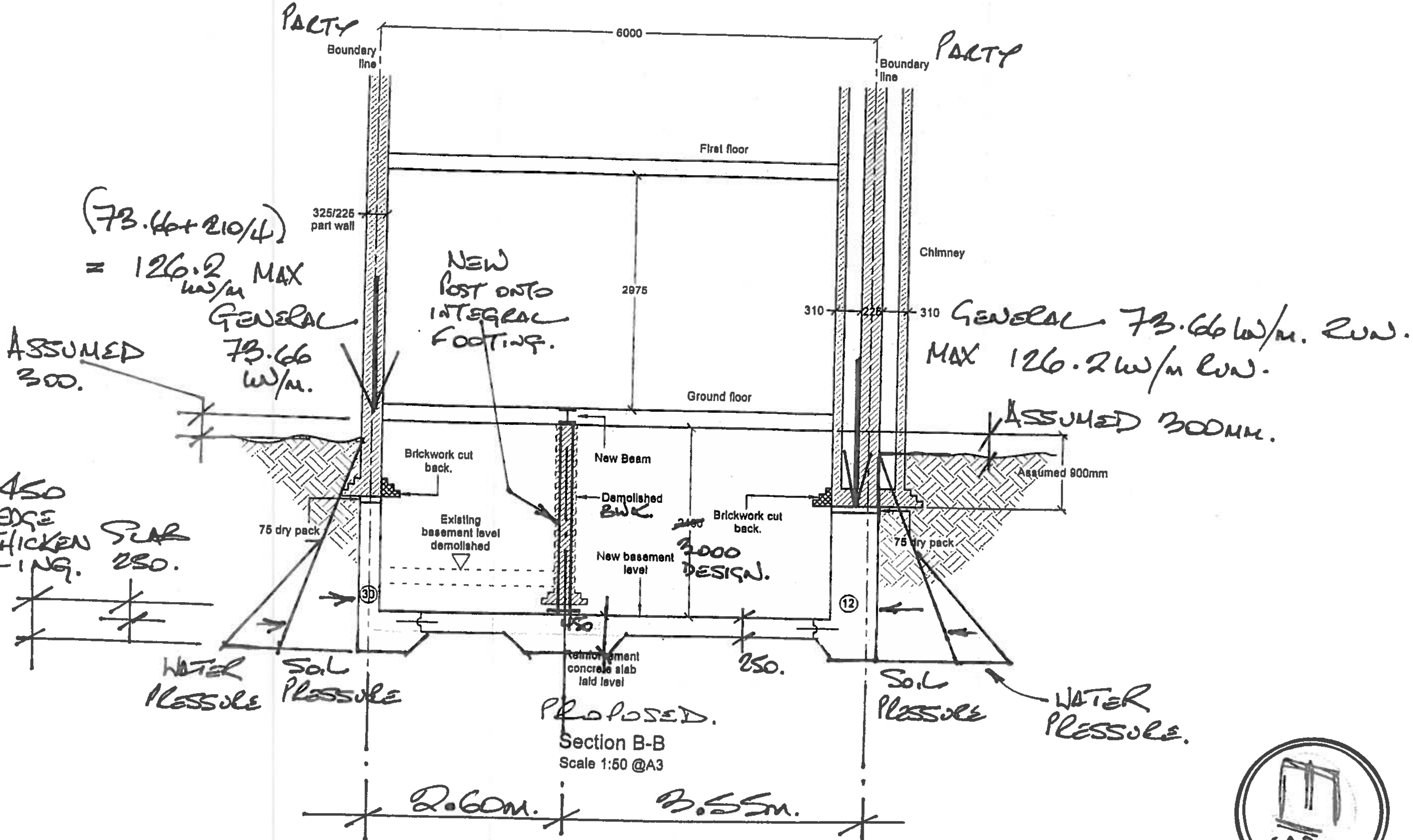
		D	L
$68.32 \times 6.15/2$	$= 210$ kN	141.2	68.5

ESTIMATE TOTAL LOADS.

		D	L
$2 \times 73.66 \times 9.5$	$= 1400.0$	1154.4	245.1
$+\ 80.24 \times 6.15$	$= 493.5$	307.5	186.0
$+\ 65.53 \times 6.15$	$= 403.0$	319.4	83.6
$+\ 2 \times 210$	$= 420.0$	282.4	157.8
GROSS SERVICE LOAD	$= 2716.5$ kN	2063.7	652.5

Project 207 SUMATRA ROAD, NW6			Job ref 1391	
Part of structure BASEMENT DESIGN			Calc.sheet no E / 12 /	rev
Drawing ref	Calc by WR	Date OCT 2018	Check by	Date

Ref	Calculations	Output
E11	BY INSPECTION, THE LOADINGS ON 4 SIDES OF BOX APPEAR TO BE UNIFORM SO THERE SHOULD BE ONLY SLIGHT "OUT OF BALANCE" OF PRESSURES. SO, IGNORE NOMINAL "OVERTURNING" MOMENT FROM LOADS ABOVE.	
	NETT SERVICE PRESSURE AT 4.0M BELOW GROUND – ACTUAL	
	$\frac{2716.5}{6.15 \times 9.5} = 46.5\ kN/m^2$	
	ALLOWED NETT BEARING AT 3.0M DEPTH	VERY SAFE.
	$110 + 3.0 \times 18 = 164.0\ kN/m^2$	
	FACTORED TOTAL LOAD ONTO BASE	
	$2068.7 \times 1.4 + 652.5 \times 1.6 = 3939.2\ kN.$	
	PRESSURE (ULT) TO U/S BASEMENT BOX	
	$\frac{3939.2}{9.5 \times 6.15} = 67.3\ kN/m^2$	35.32 11.17



Section B-B
Scale 1:50 @A3

X REV K2 15/2018 MARKED AS PROPOSED BB

SOARBOND Ltd.

Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF

Drawing title: Proposed Basement SECTION B-B

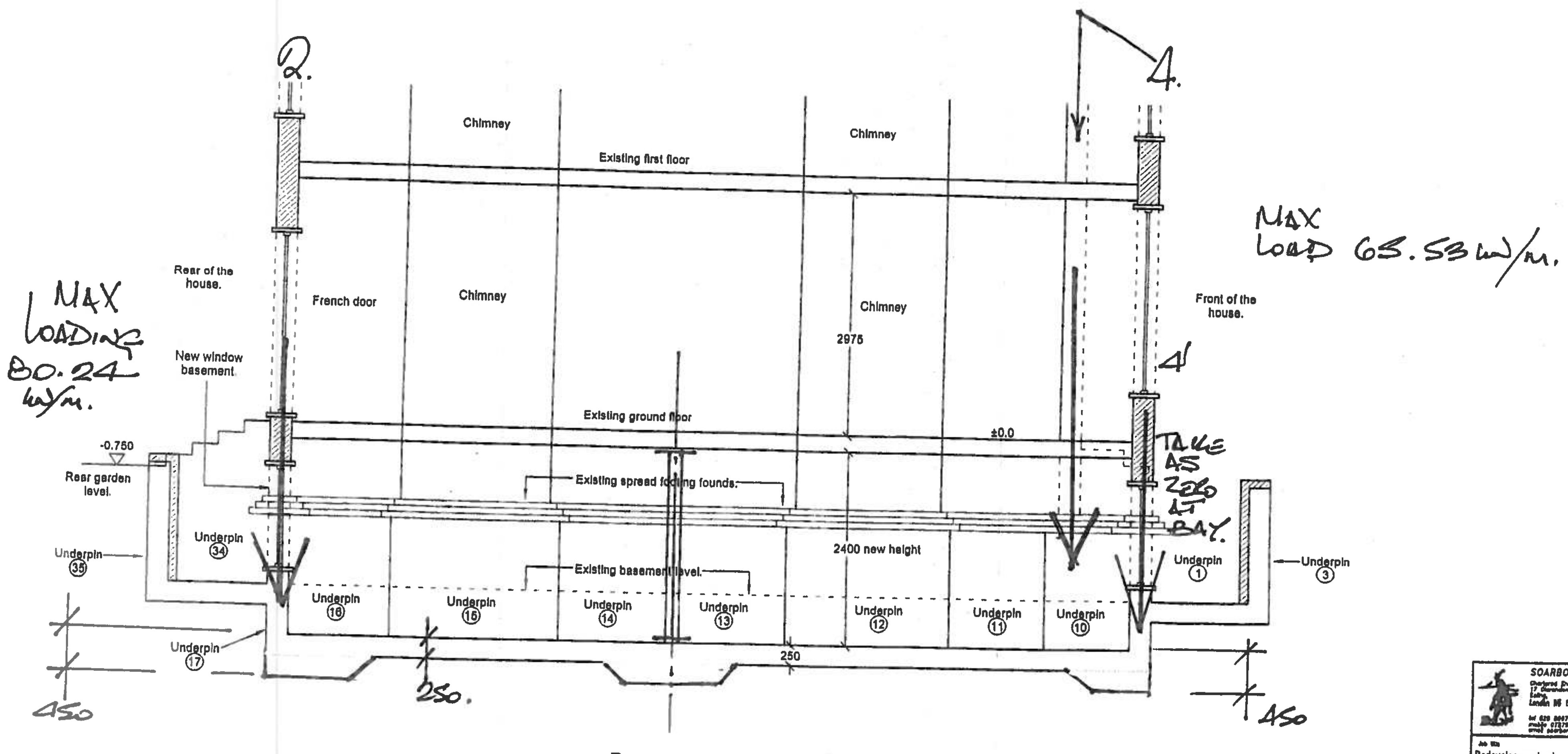
Client: Professor Kerry Hamilton

Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50

Drawing number: 1381 – 31

Rev: A

E3

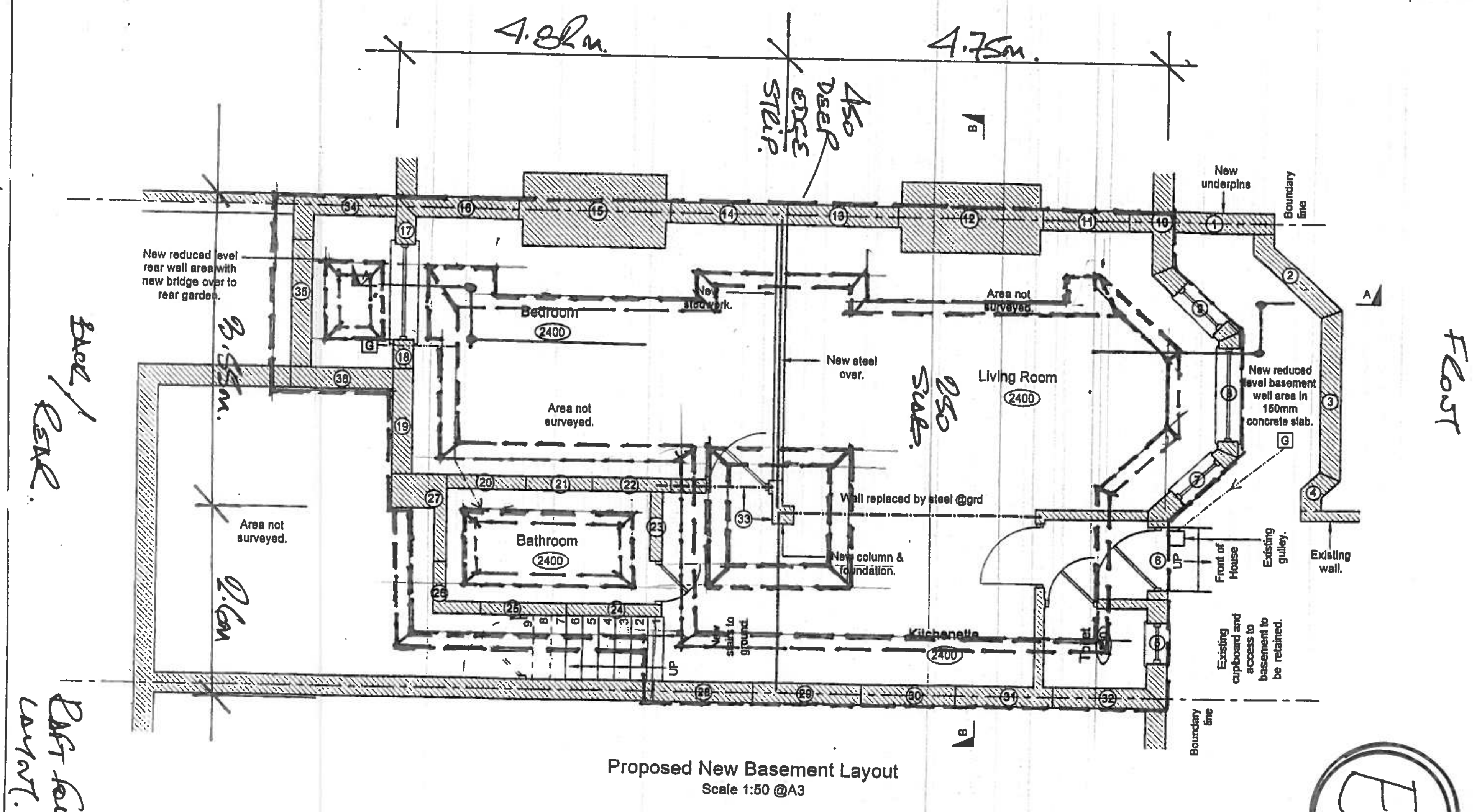


Proposed Section A-A

Scale 1:50 @A3

SOARBOND Ltd. Chartered Engineers 17 Clarendon Road, Luton, London N8 0AA	
Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Proposed Section A–A	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 – 32	

Notes:
1. This drawing to be read in conjunction with all other Architect's drawings, specification given on 09, and all other structural, electrical and mechanical drawings.
2. See elevations for details of windows and doors.
3. All new works to outside of existing house to match existing materials exactly.
4. Underpins shown thus (35)



Proposed New Basement Layout
Scale 1:50 @A3

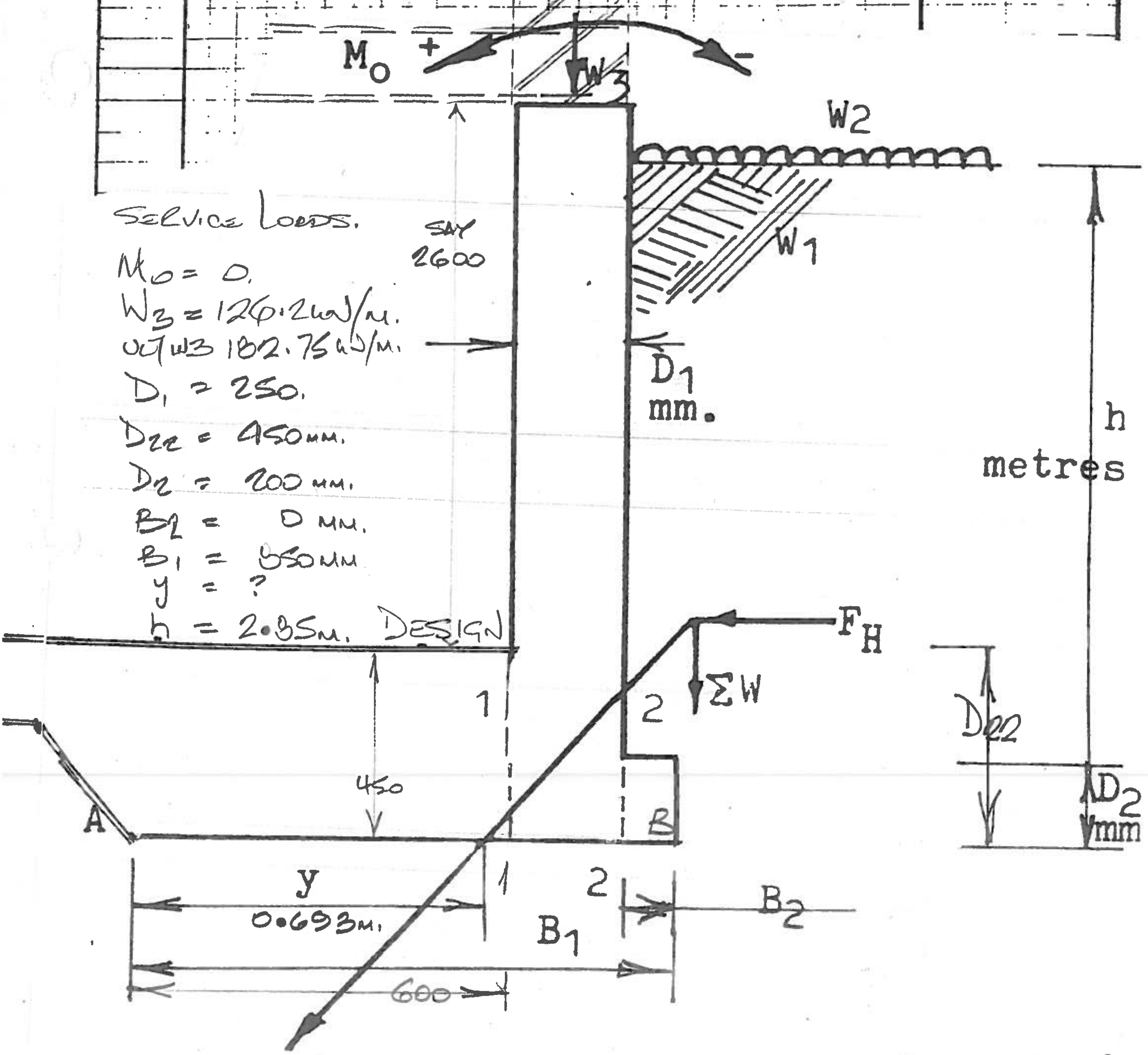
E15

SOARBOND Ltd.
Chartered Engineers

Job title: Redevelopment at 207 Sumatra Road West Hampstead London NW6 1PF	
Drawing title: Proposed Basement Layout	
Client: Professor Kerry Hamilton	
Drawn by: P. Obrzut	Date: August 2017
Chk'd: W.K.J. Zablocki	Date: August 2017
Job number: 1381	Scale @ A3: 1 : 50
Drawing number: 1381 – 11	

Project 207 SUMATRA RD, NW6			Job ref 1381	
Part of structure BASEMENT DESIGN			Calc.sheet no E, 16,	rev
Drawing ref	Calc by WZ	Date OCT. 2013	Check by	Date

Ref	Calculations	Output
	FOR SIMPLICITY DESIGN RETAINING WALL AND BASE STRIP FOOTING IN TWO STAGES AS :-	STAGE 1.



SERVICE LOADS.

$M_O = 0.$

$W_3 = 126.2 kN/m.$

ULT W3 182.75 kN/M.

$D_1 = 250.$

$D_{22} = 450 MM.$

$D_2 = 200 MM.$

$B_2 = 0 MM.$

$B_1 = 850 MM$

$y = ?$

$h = 2.35 M.$ DESIGN

Project 207 SUMATRA ROAD, NW6			Job ref 1381	
Part of structure BASEMENT DESIGN			Calc sheet no E7	rev
Drawing ref	Calc by WZ	Date OCT 2018	Check by	Date

Ref	Calculations	Output
		STAGE 2

SERVICE LOADS

M_O = 0

W_3 = 73.66 kN/m.

ULT W3 = 105.7 kN/m.

D_1 = 250 mm.

B_2 = 0 mm

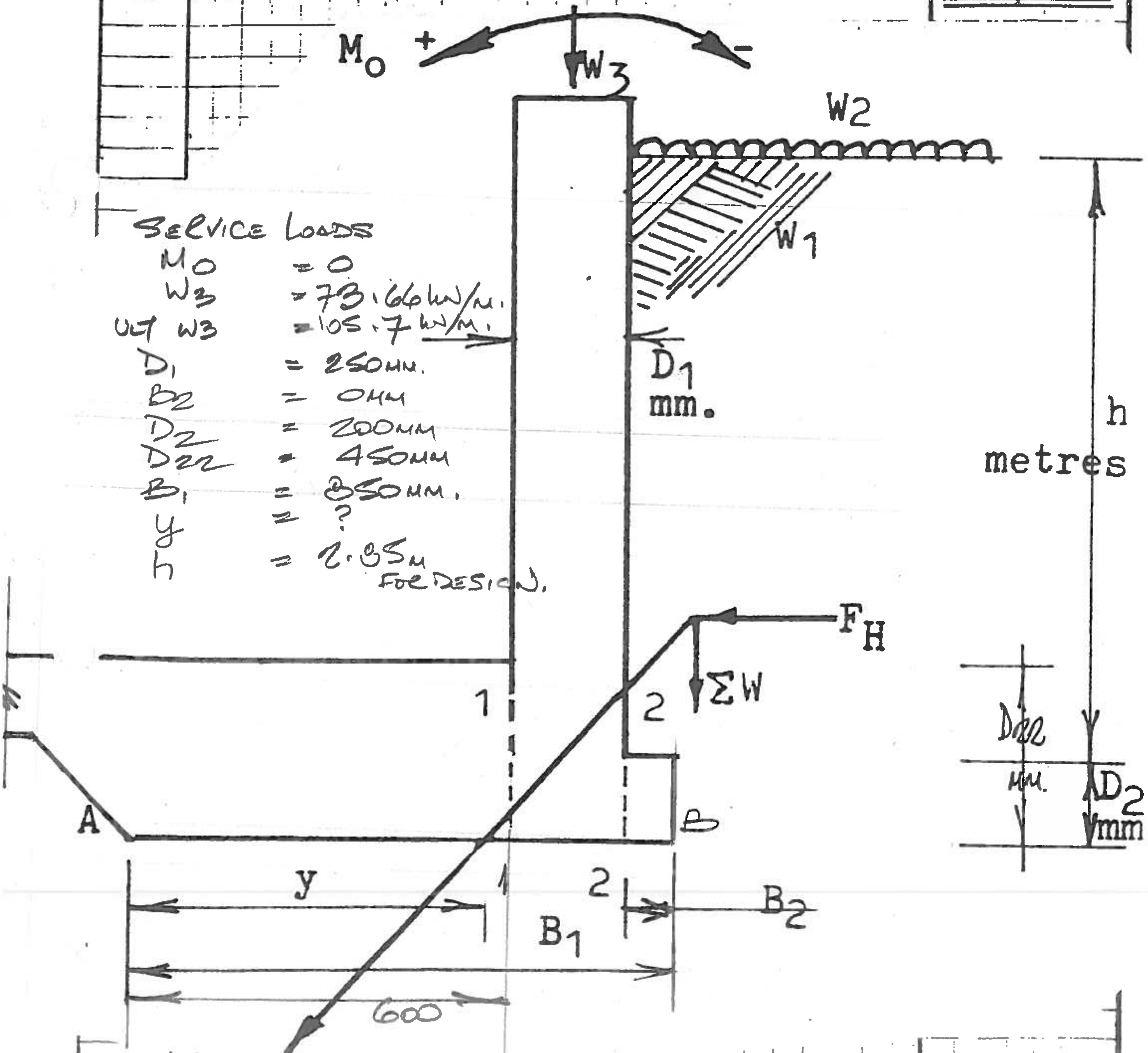
D_2 = 200 mm

D_{22} = 450 mm

B_1 = 850 mm.

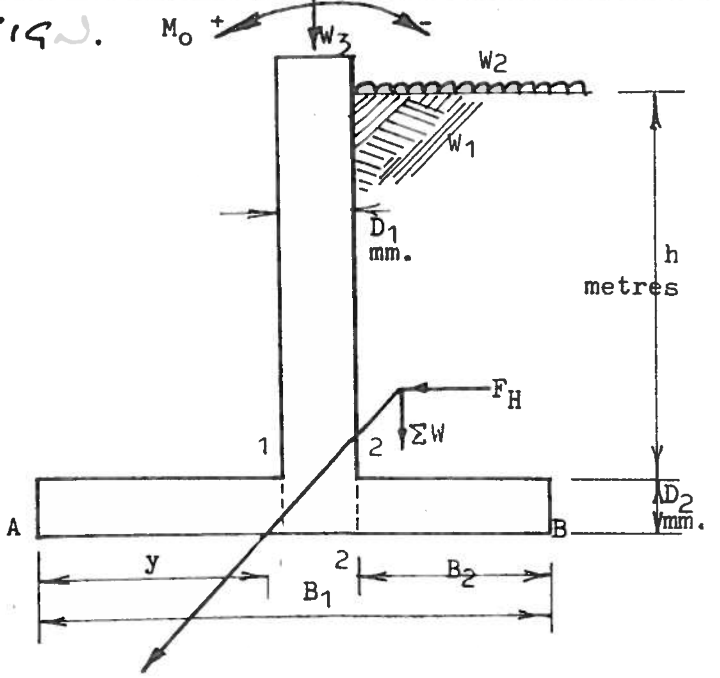
y = ?

h = 2.35 m FOR DESIGN.



CANTILEVER RETAINING WALLS -
ANALYSIS OF SPECIFIED CROSS SECTION FOR GIVEN
SET OF LOAD CONDITIONS - METRIC UNITS.

TYPICAL RETAINING WALL DESIGN.



PROGRAM DESCRIPTION

This program calculates the bending moments and associated steel areas at the quarter points up the loaded length of the cantilever stem. It also calculates the bending moments, associated steel areas and shear stresses at sections 11 and 22 in the toe and heel. The horizontal force required for a factor of 2 against sliding, the maximum ground pressure, the position of the resultant of the forces acting on the base and the factor against overturning are also calculated.

PROGRAM DETAILS

The bending moment in the stem at a distance h_n below ground level is found from the equation

$$M_n = \frac{k_A h_n^2}{6} \cdot \left(\frac{W_1 h_n}{102} + 3W_2\right) + M_o \qquad \text{kNm/m}$$

where k_A, W_1, W_2, and M_o are defined overleaf.

E18

M_n is calculated for h_n values of 0, $h/4$, $h/2$, $3h/4$ and h, and the result printed together with the required steel areas. If the value of M_n is positive the steel is required in the face adjacent to be retained material.

To ensure a factor of safety of 2 against sliding the value of $F_s = 2F_H - \mu\Sigma W$ is determined. If F_s is positive then an additional horizontal force is necessary.

The horizontal force $2F_H$ is found from the equation

$$2F_H = k_A \left(h + \frac{1}{10^3} D_2\right)\left[\frac{W_1}{102}\left(h + \frac{1}{10^3} D_2\right) + 2W_2\right] \qquad \text{kN/m}$$

where h and D_2 are defined overleaf.

The total vertical load ΣW has the value

$$\Sigma W = \bar{W}_1 + \bar{W}_2 + \bar{W}_3 \qquad \text{kN/m}$$

where $$\bar{W}_1 = B_2\left[\frac{W_1 h}{102} + W_2 + \frac{24D_2}{10^3}\right] \qquad \text{kN/m}$$

is the summation of the vertical forces on the base between point B and section 22

$$\bar{W}_2 = W_3 + \frac{24D_1}{10^3}\left(h + \frac{D_2}{10^3}\right) \qquad \text{kN/m}$$

is the summation of the vertical forces on the base between section 22 and 11

and $$\bar{W}_3 = \frac{24D_2}{10^3}\left(B_1 - \frac{D_1}{10^3} - B_2\right) \qquad \text{kN/m}$$

is the summation of the vertical forces on the base between section 11 and point A.

The safety factor against overturning O_F has the value M_2/M_1

where $$M_1 = \frac{k_A}{6}\left(h + \frac{D_2}{10^3}\right)^2 \left[\frac{W_1}{102}\left(h + \frac{D_2}{10^3}\right) + 3W_2 \right] \quad \text{kNm/m}$$

and $$M_2 = \bar{W}_1\left(B_1 - \frac{B2}{2}\right) + \bar{W}_2\left(B_1 - B_2 - \frac{D_1}{2.10^3}\right) + \frac{\bar{W}_3}{2}\left(B_1 - B_2 - \frac{D_1}{10^3}\right) - M_o \quad \text{kNm/m}$$

The resultant of the forces on the base intersects the foundation level at a distance y from A where

$$y = \frac{M_2 - M_1}{\Sigma W} \quad \text{m}$$

The maximum ground pressure developed at A depends on the value of y as follows

when $1/3.B_1 \leq y \leq 1/2.B_1$, $$P_{max} = \frac{\Sigma W}{B_1} + \frac{6\Sigma W\,(1/2B_1 - y)}{B_1^{\,2}} \quad \text{kN/m}^2$$

but when $y \leq 1/3.B_1$, $$P_{max} = \frac{2\Sigma W}{3y} \quad \text{kN/m}^2$$

where B_1 is defined overleaf. When $y > 1/2.B_1$ the computer will stop.

E19

Similarly the values of the bending moments and shear forces on section 11 and 22 of the base also depend on the value of y such that for section 11 when $1/3.B_1 \leq y \leq 1/2.B_1$

$$S_{11} = \frac{\Sigma W}{B_1} x_1.\left[1 + \frac{6(\frac{B_1}{2} - y)}{B_1}\left(1 - \frac{x_1}{B_1}\right)\right] - \bar{W}_3 \quad \text{kN/m}$$

$$M_{11} = \frac{\Sigma W}{2B_1} x_1^2 \left[1 + \frac{(\frac{B_1}{2} - y)}{B_1}\left(6 - \frac{4x_2}{B_1}\right)\right] - 1/2\, x_1\bar{W}_3 \quad \text{kNm/m}$$

but when $1/3.B_1 \geq y \geq 1/3x_1$:

$$S_{11} = \frac{\Sigma W}{3y} x_1 .\left(2 - \frac{x_1}{3y}\right) - \bar{W}_3 \quad \text{kN/m}$$

$$M_{11} = \frac{\Sigma W x_1^2}{9y} .\left(3 - \frac{x_1}{3y}\right) - 1/2 x_1 \bar{W}_3 \quad \text{kNm/m}$$

and when $y \leq 1/3x_1$:

$$S_{11} = \Sigma W - \bar{W}_3 \quad \text{kN/m}$$

$$M_{11} = \Sigma W(x_1 - y) - 1/2 x_1 \bar{W}_3 \quad \text{kNm/m}$$

For the shear force S_{22} and bending moment M_{22} on section 22 when $1/3B_1 \leq y \leq 1/2\,B_1$:

$$S_{22} = \frac{\Sigma W}{B_1} x_2 \left[1 - \frac{6(\frac{B_1}{2} - y)}{B_1}\left(1 - \frac{x_2}{B_1}\right)\right] - \bar{W}_1 \quad \text{kN/m}$$

$$M_{22} = \frac{\Sigma W x_2^2}{2B_1}\left[1 - \frac{(\frac{B_1}{2} - y)}{B_1}\left(6 - \frac{4x_2}{B_1}\right)\right] - 1/2 x_2 \bar{W}_1 \quad \text{kNm/m}$$

but when $1/3.B_1 \geqslant y \geqslant 1/3.(B_1 - x_2)$:

$$S_{22} = \Sigma W\left[1 - \frac{1}{3y}(B_1 - x_2)\right]^2 - \bar{W}_1 \qquad \text{kN/m}$$

$$M_{22} = \Sigma Wy\left[1 - \frac{1}{3y}(B_1 - x_2)\right]^3 - 1/2 x_2 \bar{W}_1 \qquad \text{kNm/m}$$

Elo

and when $y \leqslant 1/3.(B_1 - x_2)$:

$$S_{22} = -\bar{W}_1 \qquad \text{kN/m}$$

$$M_{22} = -\frac{1x_2}{2}\bar{W}_1 \qquad \text{kNm/m}$$

where $x_1 = B_1 - B_2 - \frac{D_1}{10^3}$ and $x_2 = B_2$.

NOTE: When the bending moments M_{11} and M_{22} are positive the steel is required in the bottom face of the base, and if negative in the top face. The shear stresses at sections 11 and 22 are calculated from the shear forces in the normal way.

The steel required at any section is calculated assuming $P_{st} = 230 N/mm^2$ and $Pcb = 6.9 N/mm^2$.

INPUT DETAILS

The input is listed in the following sequence; the last item is N on the input form

Item	Units	Max. digits	Description	Example
71	-	-	Program number	71
W_1	kg/m^3	5	Density of soil	1025
W_2	kN/m^2	5	Surcharge	26
M_0	kNm/m	5	Applied moment on wall from direct force W_3	50
h	m	5	Loaded height of stem	5.5
k_A	-	4	Pressure coefficient	0.3
d_1	mm	4	Effective depth to reinforcement in stem	420
D_2	mm	4	Overall base thickness	470
B_2	m	4	Length of heel	3
D_1	mm	4	Overall stem thickness	470
W_3	kN/m	5	Vertical load on wall	40
B_1	m	4	Total length of base	4.57
μ	-	4	Coefficient of friction soil/concrete	0.3
d_2	mm	4	Effective depth to reinforcement in base	420

OUTPUT DETAILS

Besides all the input details, the bending moments, with their relevant areas of steel, at the quarter points in the stem of the retaining wall are printed.

The factor of safety against overturning together with the additional horizontal force needed to be developed to achieve a factor of safety of 2 and the distance from A of the resultant force on the base, are printed.

Finally, the maximum ground pressure, the bending moments and the shear stresses at section 11 and 22 are printed, together with the relevant areas of steel reinforcement.

If the value of y is printed repetitively with D◇, after program check Y, then $y > 1/2.B_1$. If the value of d_1 or d_2 is printed repetitively with E◇ or D◇ respectively, then compression steel is required at that section.
If the value of q_{11} or q_{22} is printed repetitively with A◇ then the shear stress has exceeded the permissible value of 0.67 N/mm^2.

E21

Project 207 SUMATRA RD, NW6	Job ref 1381	
Part of structure BASEMENT DESIGN	Calc sheet no C1 022	rev
Drawing ref	Calc by / Date OCT 2003	Check by / Date

Ref	Calculations	Output
E18 – E21	STAGE 1. (RETAINING WALL).	(BY INSPECTION, MAKE STAGE 2 SIMILAR)
	$K_A = 0.35$	
	$W_1 = 1800$ kgs/m³	
	$W_2 = 2.5$ kN/m²	
	$M_0 = 0$	
	$h = 2.6$m	
	$M_u = \frac{0.35 \times 2.6^2}{6}\left(\frac{1800 \times 2.6}{102} + 3 \times 2.5\right) + 0$	
	$= 21.05$ kNm/m	(SERVICE)
	ULT M_u SAY 21.05×1.52 (SAY)	
	(ULT LOADS) $= 32$ kNm/m	
STEEL	REINFORCEMENT $d = 250 - 50 - 16/2$	⇒ 192MM
	$M_u / bd^2 = \frac{32 \times 10^6}{1000 \times 192^2}$	
	$= 0.87$	
	$\frac{As\,100}{bd^2} = 0.25$	
	$As = \frac{0.25 \times 1000 \times 192}{100}$	
	$= 480$ mm²/m	USE T16@ 200CRS (1010)
STEEL	DISTRIBUTION STEEL	
	$= \frac{0.12 \times 1000 \times 184}{100}$	
	(SERVICE FIGURES) $= 221$ mm²/m	USE T12@ 250 CRS. (452).
	F_H $0.35\left(2.6 + \frac{450}{1000}\right)\left[\frac{1800}{102}\left(2.6 + \frac{450}{1000}\right)\frac{1}{2} \times 2.5\right]$	
	$= 62.3$ kN/m	

Project ~~207 SUMATRA RD, NW6~~		Job ref 1381
Part of structure ~~BASEMENT DESIGN~~		Calc sheet no ~~C, 23,~~ rev
Drawing ref	Calc by ~~WZ~~ Date ~~OCT 2018~~	Check by Date

Ref	Calculations	Output
	(SERVICE FIGURES)	
	$\bar{W}_1 = 0 \left[\frac{1800}{10^2} \times 2.6 + 2.5 + 24 \times 0.45 \right]$	
	$= 0$ kN/m	
	$\bar{W}_2 = 126.3 + 24 \times 0.25 (3.05)$	
	$= 144.6$ kN/m.	
	$\bar{W}_3 = 24 \times 0.45 (0.85 - 0.25 - 0)$	IGNORE
	$= 6.5$ kN/m.	BASEMENT L.L.
	$\Sigma W \; 0 + 144.6 + 6.5 = 151.1$ kN/m	
	FACTOR OF SAFETY $= F_S$	$\mu = 0.3$ SAY
	$F_S \Rightarrow 2F_H - \mu \Sigma W$	
	$F_S \; 62.8 - 0.3 \times 151.1 = +17.5$ kN/m	SO SLIDING IS NOT A PROBLEM FOR STORE 1.
	SAFETY FACTOR AGAINST OVERTURNING $O_F = M_2/M_1$	
	$M_1 \; \frac{0.35}{6}(3.05)^2 \left[0 + 3 \times 2.5 \right]$	
	$= 4.07$	kNm/m
	$M_2 \Rightarrow 0 + 144.6(0.725) + \frac{6.5}{2}(0.6)$	
	$M_2/M_1 \Rightarrow \qquad = 108.74$	kNm/m.
	$108.74 / 4.07 = 26.7 \gg 2.0$	O.K AS EXPECTED.
	$y \; \frac{M_2 - M_1}{\Sigma W} \; \frac{108.74 - 4.07}{151.1} = 0.693$ m.	$y > \frac{1}{2}B$.
	CONSEQUENTLY BASE DESIGN IS NOT CRITICAL	$\Sigma W \gg F_H$.

Project 207 SUMATRA RD, NW6	Job ref 1381
Part of structure BASEMENT DESIGN	Calc sheet no C, 24 rev

Drawing ref	Calc by	Date OCT 2018	Check by	Date

Ref	Calculations	Output

$X_1 = 850 - 0 - 250 = 0.6m$

$X_2 = 0m$

11. CHECK S_{11} & M_{11}

$S_{11} = \frac{151.1 \times 0.6}{0.85}\left[1 + 6\left(\frac{\frac{0.85}{2} - 0.693}{0.85}\right)\left(1 - \frac{0.6}{0.85}\right)\right] - 6.5$ kN/m

$= -34.5$ kN/m.

$M_{11} = \frac{151.1 \times 0.6^2}{2 \times 0.85}\left[1 + \left(\frac{\frac{0.85}{2} - 0.693}{0.85}\right)\left(6 - \frac{4 \times 0}{0.85}\right)\right] - \frac{0.6 \times 6.5}{2}$

$= 131.45 - 1.95$

$= 129.5$ kNm/m.

CHECK SHEAR

d 450 − 50 − 10/2 ⇒ 390

$q_s = \frac{34.5 \times 10^3}{1000 \times 390 \times 0.6} = 0.147$ N/mm² $\ll$ 0.7 N/mm² ALLOWED. O.K.

CHECK REINFORCEMENT

M_u $\frac{129.5 \times 1.525 \times 10^6}{1000 \times 390^2} = 1.25$ M_u/bd^2

$100A_s/bd = 0.35$

A_s $\frac{0.35 \times 1000 \times 390}{100} = 1365$ mm²/m. USE H20 @ 200 CRS. IN BOTTOM.

DIST. $\frac{0.12 \times 1000 \times 380}{100} = 456$ mm²/m USE T12 @ 200 CRS DIST.

Project: 207 SUMATRA RD, NW6		Job ref: 1381	
Part of structure: BASEMENT DESIGN		Calc sheet no: E / 251	rev
Drawing ref	Calc by: WL / Date: OCT 2010	Check by	Date

Ref	Calculations	Output

TYPICAL RAFT SLAB REINFORCEMENT

TAKE OUT LOAD TO U/S RAFT

E12 PRESSURE (ULT) = 67.3 kN/m² — 35.32 11.17

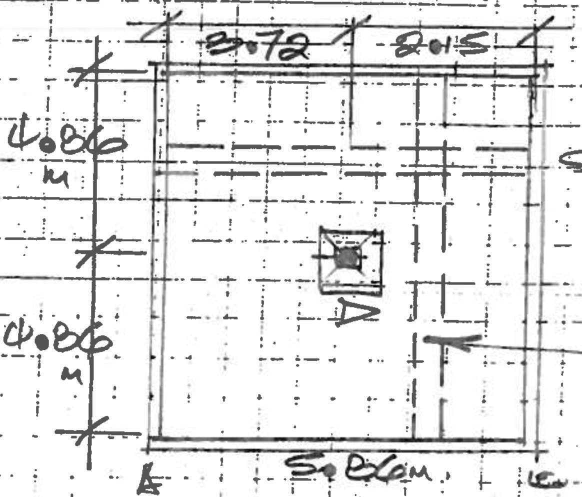
TAKE REDUCTIONS AS:—

FLOOR SLAB + FINISHES = −11.11 kN/m² — 60.22 10.50

IMPOSED LOAD ONTO RAFT 56.2 kN/m²

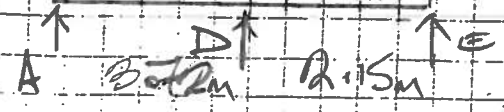
29.10 9.67

DESIGN RAFT AS:—



STRIP A DESIGN — 1 METRE STRIP

56.2 kN/m.



ALLOW FULL LOADING HERE IN LIEU OF 75%/25% ULT. D L.

SPAN AD

$R_A = R_D$ 56.2 × 3.72/2 = 104.6 kN/m — 54.13 18.0

M FIXED A (=D)

56.2 × 3.72²/12 = 64.8 kNm/m

M FREE 56.2 × 3.72²/8 = 97.2 kNm/m

Project 207 SUMATRA RD., NW6	Job ref 1381
Part of structure BASEMENT DESIGN	Calc sheet no E / 26 / 1 rev

Drawing ref	Calc by	Date	Check by	Date
		OCT 2018		

Ref	Calculations	Output

SPAN DE

R_D $(= R_E)$ $56.2 \times 2.15/2$ = 60.4 kN

D L
31.3 10.4

M FIXED D $(= E)$
$56.2 \times 2.15^2/12$ = 21.65 kNm

M FREE $56.2 \times 2.15^2/8$ = 32.5 kNm

DISTRIBUTION FACTORS

AT D

DA $$\frac{3/4 \times 1/3.72}{3/4 \times 1/3.72 + 3/4 \times 1/2.15} = 0.37$$

DE = 0.63

$\Sigma = 1.0$

MOMENT DISTRIBUTION

0	0.37	0.63	0
−64.8	+64.8	−21.65	+21.65
+64.8	+32.4	−10.83	−21.65
	−23.95	−40.77	
	+73.25	−73.25	
0			0

62.3

(37.2) (32.5)

66.05 HOGGING

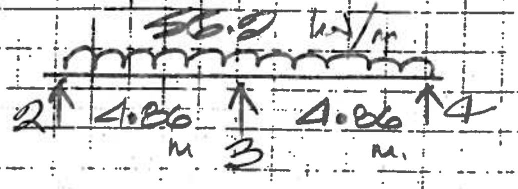
MAX M 66.05 kNm/m

Project	207 SUMATRA RD, NW6	Job ref	1381
Part of structure	BASEMENT DESIGN	Calc sheet no / rev	E, 27,
Drawing ref	Calc by: WZ / Date: OCT 2018	Check by	Date

Ref	Calculations	Output

STRIP FD DESIGN

DESIGN THE STRIP AS:-



USE 56.2 TO ACHIEVE MAX MOMENTS & REINFT DETAILS.

23 SPAN (= 3-4)

R 56.2 x 4.86 / 2 = 136.6 kN/m

D 70.7 L 28.5

M FIXED 23 (= 32)

56.2 x 4.86² / 12 = 110.6 kNm/m

M FREE 56.2 x 4.86² / 8 = 165.9 kNm/m

DISTRIBUTION FACTORS

BY INSPECTION 32 = 0.5

34 = 0.5

Σ = 1.0

MOMENT DISTRIBUTION

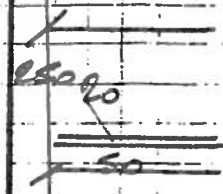
0	0.5	0.5	0
-110.6	+110.6	-110.6	+110.6
+110.6	+55.3	-55.3	-110.6
	+165.9	-165.9	
0	141.0		0
(165.9)		(165.9)	
95.4		95.4	

(FREE M)

SPAN M

M MAX = 141.0 kNm

Project 207 SUMATRA ROAD, NW6				Job ref 1331	
Part of structure BASEMENT DESIGN				Calc.sheet no E, 28	rev
Drawing ref	Calc by WZ	Date OCT 2018		Check by	Date

Ref	Calculations	Output
	CHECK 250 R.C SLAB	
	STRIP A SPAN	
	$M_u/bd^2 \quad \dfrac{66.05\times10^6}{1000\times190^2} = 1.83$	
	$100As/bd = 0.49$	
	$As \quad \dfrac{0.49\times1000\times190}{100} = 931\ mm^2/m$	USE H16 @ 200 CENTRES.
	$DIST. \quad \dfrac{0.12\times1000\times180}{100} = 216\ mm^2/m$	USE H10 @ 200 c/cs.
	STRIP B SPAN	
	$M_u/bd^2 \quad \dfrac{141.0\times10^6}{1000\times190^2}$ SAY $= 3.90$	
	$100As/bd = 1.2$	
	$As \quad \dfrac{1.20\times1000\times190}{100} = 2280\ mm^2/m$	USE H20 @ 125 c/cs OVER 1/3 MIDDLE SPAN.
	DIST USE H16 @ 200 AS ABOVE.	
	SEE DRAWING FOR REINF LAYOUT.	

DESIGN OF
CONCRETE
STRUCTURES

WINTER & NIELSEN.

1. Bearing Capacity and Settlement of Foundations

Introduction

A foundation is the means by which the loads from a structure are transmitted into the geological deposits beneath it. These loads must be transmitted in such a manner that two criteria are satisfied:

(i) There is no risk of shear failure in the underlying deposits.
(ii) The resulting settlement of the foundation is acceptable to both the appearance and the function of the structure.

Bearing capacity terms

Shallow foundation
A foundation whose depth below the surface, z, is equal to, or less than, its least dimension, B.

Deep foundation
A foundation whose depth below the surface is greater than its least dimension.

Overburden pressure
The vertical pressure at a point, caused solely by the weight of the geological deposits above it. The effects of a foundation load are not included.

Ultimate bearing capacity
The average contact pressure between the foundation and the soil which will produce shear failure in the soil.

Maximum safe bearing capacity
The maximum contact pressure to which the soil can be subjected without risk of shear failure. This is based solely on the strength of the soil and is simply the ultimate bearing capacity divided by a suitable factor of safety.

Allowable bearing pressure
The maximum allowable net loading intensity on the soil allowing for both shear and settlement effects.

E29

Types of foundations

Strip foundation

A foundation of continuous length. The term is also used to describe a foundation whose length is considerably greater than its width.

Pad foundation

A spread footing designed to carry either one or more column loads. When underlying soil conditions permit, it is usual to provide each column of a structure with its own pad foundation but there are often occasions when a pad foundation carries two or more columns.

Raft foundation

This is a rather loose term used to denote any foundation that covers a large area. A raft foundation can vary from a fascine mattress carrying a road over a marsh to an extensive basement of deep beams and slabs supporting a large structure.

Piled foundation

Piling can be either end bearing, frictional or a combination of the two (see chapter 2).

Determination of the ultimate bearing capacity of a soil

Terzaghi's equations

These equations for the ultimate bearing capacity, q, of shallow strip, square and circular foundations are based on Prandtl's analysis of the effects of a punch penetrating into metal and were first published in 1943. They are well known and their derivation is described in most soils text books (Smith 1978).

For a strip footing:

$$q = cN_c + \gamma z N_q + 0.5\gamma B N_\gamma$$

For a circular footing:

$$q = 1.3cN_c + \gamma z N_q + 0.3\gamma B N_\gamma \text{ (where B = diameter).}$$

For a square footing:

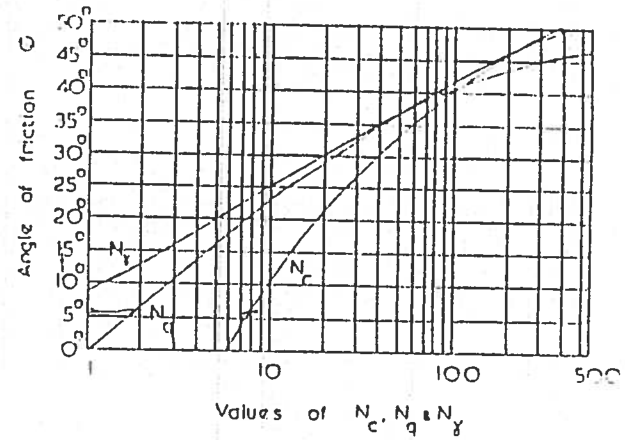
$$q = 1.3cN_c + \gamma z N_q + 0.4\gamma B N_\gamma$$

For a rectangular footing:

$$q = cN_c\left(1 + 0.3\frac{B}{L}\right) + \gamma z N_q + 0.5\gamma B N_\gamma\left(1 - 0.2\frac{B}{L}\right)$$

In the equations:

c = unit cohesion of the soil
γ = unit weight of the soil
z = depth from surface to underside of foundation
N_c, N_q and N_γ are bearing capacity coefficients which depend upon ϕ, the angle of friction of the soil, and can be obtained from fig. 1.1.



ϕ	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°	50°
N_c	5.7	7.3	9.6	12.9	17.7	25.1	37.2	57.8	95.7	172	[illegible]
N_q	1.0	1.6	2.7	4.4	7.4	12.7	22.5	41.4	81.3	173	415
N_γ	0.0	0.5	1.2	2.5	5.0	9.7	19.7	42.4	100	[illegible]	[illegible]

Fig. 1.1 Terzaghi's bearing capacity coefficients

Value of N_c for cohesive soils

From fig. 1.1 is seen that when $\phi = 0°$, $N_c = 5.7$, $N_q = 1.0$ and $N_\gamma = 0$. Hence, according to Terzaghi, when $\phi = 0°$; the ultimate bearing capacity of a strip footing is:

$$q = 5.7c + \gamma z$$
$$= 5.7c \text{ for a surface footing.}$$

E30

Skempton (1951) showed that for cohesive soils ($\phi = 0°$) N_c tends to increase with depth and suggested that values of N_c greater than 5.7 could be inserted into the Terzaghi equations when appropriate. His suggested values for N_c are shown in fig. 1.2.

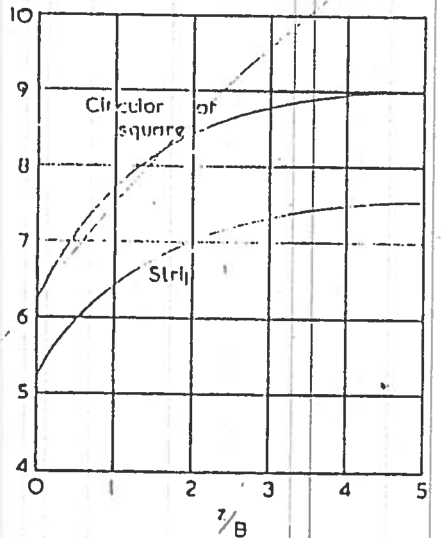


Fig. 1.2 Variation of coefficient N_c with depth (Skempton)

Net ultimate bearing capacity and safe bearing capacity

The foregoing equations give values for q, the ultimate bearing capacity of the soil on which the foundation is to sit. To obtain the safe bearing capacity, the relevant value for q must be divided by a factor of safety against bearing capacity failure, generally taken as 3.

However, it must be remembered that, due to the excavation of material in order that the foundation can be installed at depth z, there will be a relief in vertical pressure at the foundation level of γz. This means that the factor of safety should be applied to the *net* and not to the *gross* value of ultimate bearing capacity. If the excavation is subsequently backfilled the overburden pressure is restored and the net bearing capacity therefore applies.

For a strip footing:

$$q_{net} = cN_c + \gamma z N_q + 0.5\gamma B N_\gamma - \gamma z$$
$$= cN_c + \gamma z(N_q - 1) + 0.5\gamma B N_\gamma$$

and safe bearing capacity = $(q_{net}/3) + \gamma z$.

When the footing is at the surface of the soil, $z = 0$ and safe bearing capacity = $q/3 = cN_c/3$.

Example 1.1

A circular foundation is to carry a concentric column load of 850 kN.

The foundation will be installed at a depth of 1.0 m in a partially saturated soil which has an angle of friction of 10°, a unit cohesion of 40 kN/m² and a unit weight of 18 kN/m³.

Ignoring the weight of the foundation and given that the excavated soil will not be backfilled, determine a suitable dimension for the diameter of the foundation. Assume a factor of safety against bearing capacity failure of 3.

For a circular foundation:

$$q_{net} = 1.3\,cN_c + \gamma z(N_q - 1) + 0.3\,\gamma B N_\gamma$$

From fig. 1.1, for $\phi = 10°$, $N_c = 9.6$, $N_q = 2.7$ and $N_\gamma = 1.2$

$$\therefore q_{net} = 1.3 \times 9.6 \times 40 + 18 \times 1 \times 1.7 + 0.3 \times 18 \times B \times 1.2$$
$$= 529.8 + 6.48\,B.$$

Safe bearing capacity:

$$= \frac{529.8 + 6.48B}{3} + 18 \times 1$$
$$= 194.6 + 2.16B.$$

Hence, ignoring the weight of the foundation:

$$\frac{850}{\pi B^2/4} = 194.6 + 2.16\,B$$

or

$$B^3 + 90.1\,B^2 - 501.1 = 0$$

whence

$$B = 2.328$$

so that the foundation requires a diameter of about 2.35 m.

Effective stress form of the equations

The foregoing Terzaghi equations are expressed in terms of total stress but are equally valid when expressed in terms of effective stress.

E31

For a strip footing:

$$q = c'N_c + p_0'N_q + 0.5\,\gamma' BN_\gamma$$

where c' = unit cohesion of soil with respect to effective stresses,
p_0' = effective overburden pressure (total pressure–pore water pressure)
N_c, N_q and N_γ are bearing capacity coefficients which depend upon the value of ϕ', the angle of internal friction with respect to effective stress, and can be obtained from fig. 1.1.
γ' = submerged unit weight of soil.

Example 1.2

A square footing is required to carry a column load of 560 kN with a factor of safety against bearing capacity failure of 3. The foundation is to be placed at a depth of 1.5 m below the surface of a loose granular soil where the water table will be at foundation level.

The angle of friction of the soil can be assumed to be 28° and the bulk unit weights of the soil above and below the water table are 16.5 and 18.3 kN/m³ respectively.

If the excavation will be backfilled to its original level, determine a suitable size for the foundation.

For a square foundation:

$$q_{net} = 1.3\,cN_c + p_0'(N_q - 1) + 0.4\,\gamma' BN_\gamma.$$

The soil is granular, so $c' = 0$ and the equation becomes:

$$q_{net} = p_0'(N_q - 1) + 0.4\,\gamma' BN_\gamma.$$

From fig. 1.1, for $\phi' = 28°$, $N_q \approx 20$ and $N_\gamma \approx 17$.

Then $$q_{net} = 1.5 \times 16.5 \times 19 + 0.4B \times 17\,(18.3 - 9.81)$$
$$= 470.25 + 57.7B \text{ kN/m}^2.$$

Safe bearing capacity = $156.8 + 19.2B$ kN/m².

Hence, ignoring the weight of the foundation:

$$560/B^2 = 156.8 + 19.2B$$

or $$560 = 156.8B^2 + 19.2B^3$$

whence $$B = 1.72 \text{ m.}$$

Example 1.3

A block of flats is to be erected on a raft foundation at a site where the soil profile consists of 1 m of fill material overlying a deep bed of saturated clay. The dimensions of the raft will be 24 m × 12 m and it will be founded at a depth of 2 m below the surface of the ground.

A site investigation has shown that the undrained shear strength of the clay increases from 50 kN/m² at proposed foundation level to 80 kN/m² at a depth of 14 m. The unit weights of the fill and the clay are 17 and 20 kN/m³.

If the gross pressure at foundation level from the raft will be 1[illegible] kN/m², check the factor of safety against bearing capacity failure, which should not be less than 2.5.

No ground water conditions have been noted, ∴ use total stress equations.

For a rectangular foundation:

$$q_{net} = cN_c\left(1 + 0.3\,\frac{B}{L}\right) + \gamma z(N_q - 1) + 0.5\,\gamma BN_\gamma\left(1 - 0.2\,\frac{B}{L}\right)$$

With $\phi_u = 0°$ (soil is a saturated clay), $N_c = 5.7$, $N_q = 1.0$ and $N_\gamma = 0$

$$\therefore q_{net} = cN_c\left(1 + 0.3\,\frac{B}{L}\right).$$

The cohesion of the soil varies with depth. In this type of problem it is general practice to assume that the unit cohesion value to be used in the bearing capacity formula is the average value between foundation level and a depth of 2/3 B below it.

At 2/3 B below foundation level,

$$c_u = 50 + (80 - 50)\,\frac{8}{12} = 70 \text{ kN/m}^2$$

$$\therefore \text{average } c_u = \frac{1}{2}(50 + 70) = 60 \text{ kN/m}^2.$$

Hence, $$q_{net} = 60 \times 5.7\left(1 + 0.3 \times \frac{12}{24}\right) = 393.3 \text{ kN/m}^2$$

E32

Let factor of safety = F.

Then safe bearing capacity:

$$= \frac{393.3}{F} + (1 \times 17 + 1 \times 20)$$

$$= \frac{393.3}{F} + 37.$$

Gross contact pressure from the raft:

$$= 175\ kN/m^2$$

$$\therefore F = \frac{393.3}{175 - 37} = 2.85 > 2.5 \ldots\ldots O.K.$$

Modification of Terzaghi's equations for deep foundations

In the case of a deep foundation ($z > B$) the ultimate bearing capacity is increased due to the side friction generated around the surface of the excavation. This increase can be allowed for by the following technique.

Consider a rectangular foundation, of dimensions B × L, founded at a depth z.

Ultimate bearing capacity of foundation = $q + 2(L + B)zf_s$

where q = ultimate bearing capacity of a shallow foundation of dimensions B × L

f_s = unit skin friction (as calculated for the shaft of a pile—see chapter 2).

Choice of soil parameters

As the bearing capacity equations can be expressed in terms of either total or effective stress, the design engineer has to decide which conditions most closely fit his problem.

For free draining soils the effective stress parameters will apply in the soil at all stages of construction and life of the foundation.

When a foundation load is applied to a saturated clayey soil the excess pore water pressures generated can only dissipate with time.

Therefore, for most foundation problems of this type, the short term stability, i.e. the stability immediately after construction, should be checked in terms of total stress.

For the estimation of the long term stability of a foundation on clayey soil, i.e. after the dissipation of excess pore pressures, the effective stress parameters are relevant. However, it should be remembered that dissipation of pore water pressure is accompanied by consolidation of the soil which generally leads to an increase in strength.

It is therefore rarely necessary to check the long term stability of a foundation.

Further bearing capacity equations

Since Terzaghi's original equations were published several authors have proposed modifications and improvements.

Meyerhof (1951) evolved equations for deep foundations and Hansen (1957) proposed equations for cohesive soils which are considered to be more accurate than those of Terzaghi. Balla (1962) proposed a theory which is considered to be the most appropriate for granular soils, or soils possessing little cohesion.

The fact remains, however, that Terzaghi's original equations are still the most widely used and the reason is not far to seek. They can be used for all types of soils and give conservative values for the ultimate bearing capacity which often lead to safe bearing capacities close to the allowable bearing pressures when settlement effects are taken into account.

Determination of bearing capacity values for granular soils

Although Terzaghi's equations can be applied to all types of soil they are often impractical with granular soils because it is almost impossible to obtain undisturbed samples that are sufficiently satisfactory to be meaningfully tested in the laboratory.

However, with these soils, the most decisive factor in the determination of the allowable bearing pressure is the amount of settlement that the foundation will experience. Reasonably satisfactory predictions of settlement in granular soils can be made from the results of in situ tests and it is now general practice to estimate the required parameters on this basis.

E33

There are two main types of test, those that penetrate into the soil and those that are applied at its surface.

Penetration tests

The two most commonly used penetration tests are the Standard penetration test and the Dutch cone penetration test. Both are extremely simple to carry out, which accounts for their popularity.

(a) The Standard penetration test (S.P.T.)

The Standard penetration test, described in most soils text books, is a dynamic test carried out in a borehole and consists in determining the S.P.T value, N, i.e. the number of blows of a standard weight that are required to drive a 35 mm interior diameter sampling tube through a depth of 300 mm into the soil, having already driven it through a depth of 150 mm.

Terzaghi and Peck (1948) established an empirical relationship between the S.P.T. value, N, and the bearing pressure, p, that results in settlements not greater than 25 mm for foundations of varying widths. This relationship is illustrated in fig. 1.3 and is applicable to foundations unaffected by ground water. The authors suggested that, when the ground water level is at a depth below the foundation of less than B, the values of p quoted in the figure should be halved, as settlements are approximately doubled.

Terzaghi and Peck also suggested that in saturated (i.e. below the water table) fine and silty sands the N value could be altered due to the low permeability of the soil.

They suggested that N be corrected by using the formula:

$$N_{corrected} = 15 + \frac{N - 15}{2}$$

where N = actual blow count obtained from test.

Various criticisms and suggestions have been applied to Terzaghi and Peck's N-p relationship since its introduction.

Meyerhof (1956) considered that the values of p were conservative and could be increased by 50%. He also considered that, whilst

6.15,

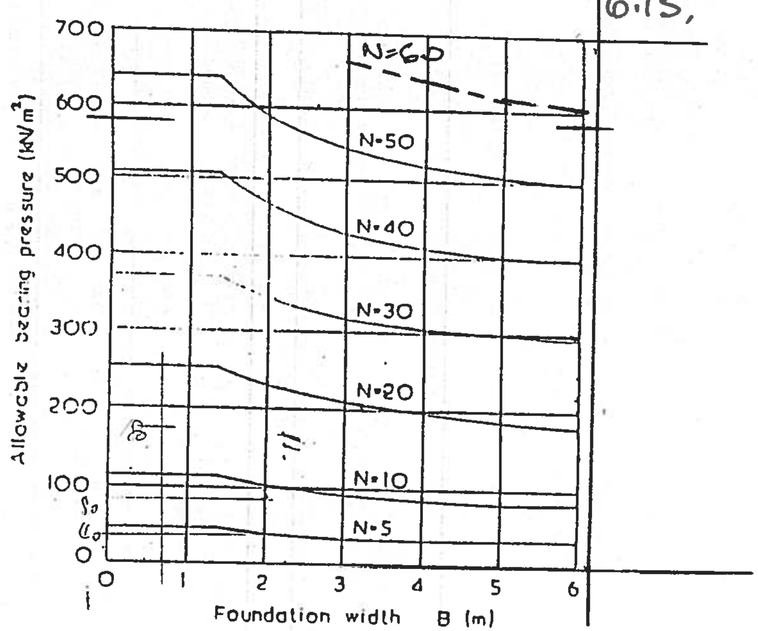


Fig. 1.3 Relationship between allowable bearing pressure, p, and S.P.T. value, N (Terzaghi and Peck, 1948)

agreeing with the correction for submerged silty and fine sands, the practice of halving allowable pressure values for foundations subjected to ground water conditions was possibly overconservative in that the influence of the water table is already included in the measured value of N and therefore need not be allowed for again.

Correction for overburden pressure

The effect of the removal of the overburden pressure prior to the test has been studied by a number of investigators: Gibbs and Holtz (1957), Thorburn (1963), Peck and Bazaraa (1969), Peck *et al.* (1974).

The correction factor to be applied to the actual blow count, N, as proposed by Peck and Bazaraa, is reproduced in fig. 1.4 and is considered to give realistic values. It is seen that the correction imposes a significant increase in N for shallow depths.

E34

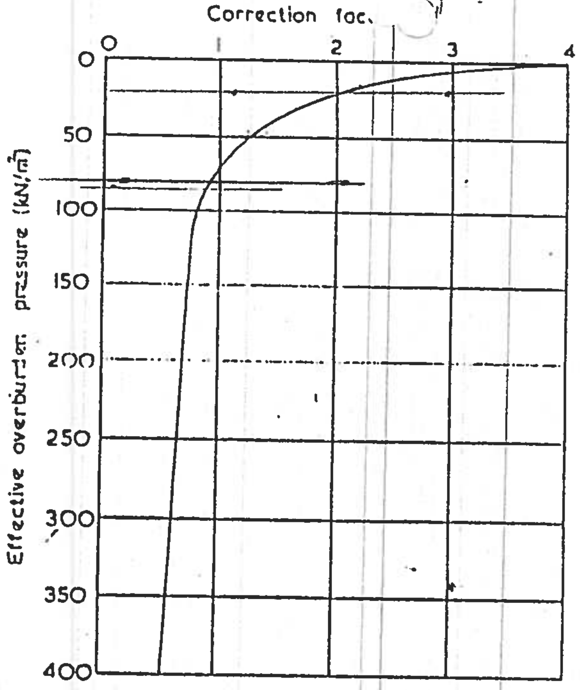


Fig. 1.4 Correction factor to be applied is the S.P.T. value, N, to allow for the effect of overburden pressure (after Peck and Bazaraa, 1969)

Peck and Bazaraa agreed with Meyerhof that Terzaghi and Peck's p values were low and they also recommended that they be increased by 50%. They further suggested that, when the ground water level is at a depth of less than B/2 below the foundation, the 25 mm settlement of the Terzaghi and Peck curve will be increased by a factor K, where K is the ratio of the effective overburden pressure at 0.5B below the foundation when the soil is unsubmerged to that at the same level when the soil is submerged.

Example 1.4
A total uniformly distributed load of 9.5 MN is to be supported by a raft founded at a depth of 1.5 m in a granular soil. The bulk unit weight of the soil is 18 kN/m³ and ground water level occurs at a depth of 10 m below the surface. S.P.T. values, taken at 1 m and 6 m depths, were N = 8 and N = 28 respectively.
The proposed dimensions of the raft are 8 m × 6 m. Check that

the average settlement of the raft will not exceed 25 mm.

Effective overburden pressure at 1 m depth = 1 × 18 = 18 kN/m².
From fig. 1.4, corrected N = N' ≈ 8 × 2.4 = 19.
Effective overburden pressure at 6 m depth = 6 × 18 = 108 kN/m².
From fig. 1.4, corrected N = N' ≈ 28 × 0.8 = 22.
Average N' value ≈ 20.
From fig. 1.3, for N' = 20 and B = 6 m, allowable bearing pressure = 180 kN/m².

According to Peck and Bazaraa this can be increased by 50%.
Thus, design allowable bearing pressure = 1.5 × 180 = 270 kN/m².
∴ total foundation load to cause an average settlement of 25 mm

$$= \frac{270}{1000} \times 8 \times 6 = 12.96 \text{ MN}.$$

As the actual foundation load is 9.5 MN, the settlement of the raft will average less than 25 mm.

$$\text{Actual average settlement} = \frac{9.5}{12.96} \times 25 = 18 \text{ mm}.$$

Example 1.5
What would the average settlement of the raft in example 1.4 have been if the ground water level had been at a depth of 1.5 m, all other conditions remaining the same?
As the ground water level is at 1.5 m depth, the effective stress at 1 m depth is unaltered.

$$\therefore N' = 19 \text{ (as before)}.$$

Effective stress at 6 m depth = 6 × 18 − 9.81 × 4.5 = 63.9 kN/m².

From fig. 1.4, N' ≈ 1.1 × 28 = 31.

$$\therefore \text{Average } N' = 25.$$

From fig. 1.3, and increasing by 50%,

Allowable bearing pressure = 1.5 × 240 = 360 kN/m² for a 6 m wide raft.

E35

This now has to be corrected for the high ground water conditions.

$$K = \frac{\text{Overburden pressure of unsubmerged soil at depth 0.5B below foundation}}{\text{Effective overburden pressure at same depth with G.W.L. at 1.5 m depth}}$$

$$= \frac{18(1.5 + 3)}{18(1.5 + 3) - 9.8 \times 3} = 1.57.$$

This indicates that, if the foundation were stressed to a bearing pressure of 360 kN/m², the total settlement would be in the region of $25 \times 1.57 = 39$ mm.

$$\text{Actual stress from foundation load} = \frac{9.5 \times 1000}{6 \times 8} = 198 \text{ kN/m}^2.$$

$$\therefore \text{Actual settlement of raft} \approx \frac{198}{360} \times 39 = 22 \text{ mm}$$

which is less than the normally permitted maximum settlement of 25 mm. The proposed dimensions of the raft are therefore satisfactory.

(b) The Dutch cone penetration test

The apparatus consists of a cylindrical probe, of 1000 mm² cross sectional area, headed by a cone with an apex angle of 60°. The probe is forced down through the soil at a steady rate and the resistance to penetration measured. If desired the point resistance and the resistance to side friction can be measured separately. The main disadvantage of the test is that it is not always possible to penetrate an underlying hard layer of soil.

The results of the test, as with the Standard penetration test, can be used to predict the possible settlement of a foundation placed on the soil tested.

The method of prediction generally used is that proposed by de Beer and Martens (1957):

$$\text{Constant of compressibility, } C_s = 1.5 \frac{C_r}{p_{0_1}}$$

where C_r = static cone resistance (kN/m²)

p_{0_1} = effective overburden pressure at point tested.

$$\text{Total immediate settlement, } \rho_i = \frac{H}{C_s} \log_e \frac{p_{0_2} + \Delta\sigma_z}{p_{0_2}},$$

where $\Delta\sigma_z$ = vertical stress increase at the centre of the consolidating layer of thickness H.

p_{0_2} = effective overburden pressure at the centre of the layer before any excavation or load application

After a series of observations on actual structures it is now generally felt that de Beer and Martens' method can give predicted settlements that are high, sometimes twice as much as the actual measured value. In view of this, the relationship between C_s and C_r proposed by Meyerhof (1965),

$$C_s = 1.9 \frac{C_r}{p_{0_1}}$$

is often preferred. This tends to give more conservative estimates of settlement.

A further method by which Dutch cone test results can be used to predict settlement has been prepared by Schmertmann (1970). It is based on the fact that the greatest strain beneath a foundation on sand is at a depth of approximately B/2 beneath it and on the assumption that significant vertical stresses do not extend beyond a depth of 2B.

The method is considered by many to give more realistic predictions than de Beer and Martens and has the advantage that it can predict settlement not only immediately after construction but also after several years as it can allow for creep effects.

Schmertmann's formula for the settlement of a foundation on sand can be expressed as:

$$\rho = C_1 C_2 \Delta\sigma_z \sum_{0}^{2B} \frac{I_z \Delta z}{2C_r}$$

where $\Delta\sigma_z$ = increase in vertical stress at depth z due to foundation load

C_r = static cone resistance

C_1 = correction factor for depth

C_2 = correction factor for creep

I_z = strain influence factor

Δz = thickness of layer under consideration.

With the knowledge that the greatest strain occurs at about B/2,

E36

the variation of I_z with depth is as shown in fig. 1.5.

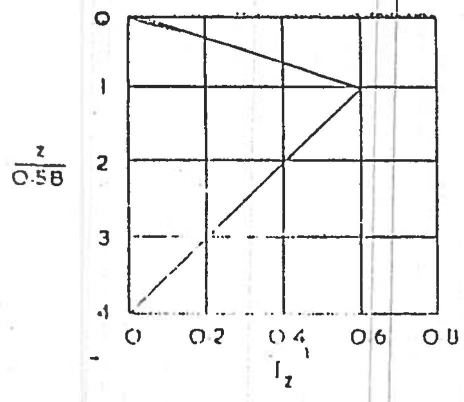


Fig. 1.5 Variation of I_z with z (Schmertmann, 1970)

C_1 and C_2 are obtained from the following formulae:

$$C_1 = 1 - 0.5\left(\frac{p'_0}{\Delta\sigma_z}\right)$$

where p'_0 = initial effective overburden pressure at the foundation

$$C_2 = 1 + 0.2\log_{10}\left(\frac{t}{0.1}\right)$$

where t = time in years.

The operation of both methods is best shown by means of an example.

Example 1.6

A 20 m × 4 m spread footing is to be founded in a deep sand layer of unit weight 18 kN/m³ at a depth of 1.8 m below its surface (fig. 1.6A).

When the full structure is completed, estimated construction time = 1 year, the gross loading from the foundation will be a uniform pressure of 204 kN/m².

Dutch cone tests carried down through the sand gave the results shown in fig. 1.6B.

Determine: (i) Settlement immediately after construction.
(ii) Settlement 4 years after construction.

For both methods the scatter of the test results must be approximated to an idealised form suitable for use in the calculations (fig. 1.6C).

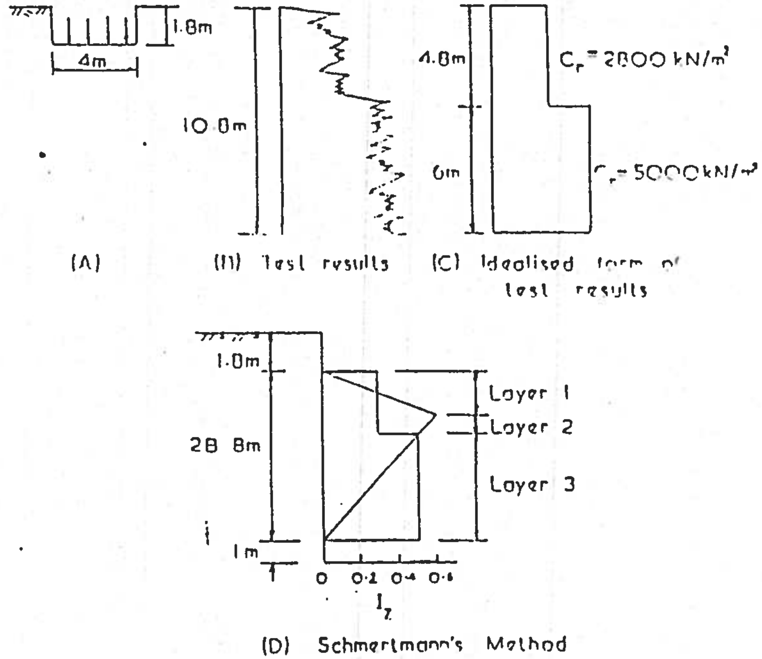


Fig. 1.6

(a) Schmertmann's method

The procedure is to superimpose fig. 1.5 on to fig. 1.6C and to divide the resulting diagram into slices that are convenient for analysis (fig. 1.6D). It is seen that, for this example, the minimum number of slices is three.

The calculations are best set out as in table 1.1:

Table 1.1

Layer no.	Thickness Δz (mm)	C_r (kN/m²)	I_z (average)	$\frac{I_z \Delta z}{C_r}$
1	2000	2800	0.3	0.1071
2	1000	2800	0.55	0.0982
3	5000	5000	0.25	0.125
				Σ 0.3303

E37

$$C_1 = 1 - 0.5\left(\frac{1.8 \times 18}{204 - 32.4}\right) = 0.9056.$$

Note: Relief in vertical pressure due to excavation for foundation = 1.8 × 18 = 32.4 kN/m².

(i) Settlement immediately after contruction.

The construction time will be one year and a simple assumption, so that C_2 can be evaluated, is that, instead of being applied gradually over one year, the foundation load is applied suddenly after six months, i.e. t = 0.5.

Then $$C_2 = 1 + 0.2 \log_{10}\left(\frac{0.5}{0.1}\right) = 1.14.$$

∴ Settlement at end of construction:

$$= 0.9056 \times 1.14 \times (204 - 32.4) \times 0.3303$$
$$= 59 \text{ mm}.$$

(ii) Settlement 4 years after construction:

$$t = 4.5 \qquad \therefore C_2 = 1.33$$

and $$\rho = 0.9056 \times 1.33 \times 171.6 \times 0.3303 = 68 \text{ mm}.$$

(b) de Beer and Martens' method

The idealised form of the soil conditions (fig. 1.6C) is used with this method. For simplicity, let us divide the soil beneath the foundation into three equal strips, each 3 m in thickness. Again, the calculations are best tabulated (tables 1.2 and 1.3).

Table 1.2

Slice no.	Depth below foundation to centre of layer, z (m)	p'_{o_2} (kN/m²)
1	1.5	3.3 × 18 = 59.4
2	4.5	6.3 × 18 = 113.4
3	7.5	9.3 × 18 = 167.4

Hence, according to this method, the settlement of the foundation immediately after construction, and then for all time = 69 mm.

Determination of $\Delta\sigma_z$ increments

The simplest, and quite often accurate enough, method for the

Table 1.3

Slice no.	p'_{o_2} (kN/m²)	C_r (kN/m²)	$C_s = \frac{1.9 C_r}{p'_{o_2}}$ (Meyerhof)	Gross $\Delta\sigma_z$ (kN/m²)	Net $\Delta\sigma_z$ (kN/m²)	$\log_e \frac{p'_{o_2} + \Delta\sigma_{z\,net}}{p'_{o_2}}$ (A)	$\frac{H}{C_s} \times (A)$
1	59.4	2800	90	188	156	1.288	42.9
2	113.4	5000	84	103	71	0.486	17.4
3	167.4	5000	57	63	31	0.170	8.9
							Σ 69.2

determination of the increase in vertical stress at a depth due to a foundation load is to assume that this load spreads itself downwards throughout the soil at a slope of 1 horizontal to 2 vertical. The reader might like to check that, with this assumption, the gross values of $\Delta\sigma_z$ are:

Centre of slice 1 = 138 kN/m²
Centre of slice 2 = 78 kN/m²
Centre of slice 3 = 52 kN/m².

A more sophisticated technique has been evolved by Steinbrenner (1934) and is described by Smith (1978). The values of $\Delta\sigma_z$ obtained by this method have been used in the preceding example.

Relationship between C_r and N

Various workers have attempted to arrive at a formula which would relate the S.P.T. value, N, to its corresponding static cone resistance, C_r. Results have been disappointing but a formula generally in use, although inaccurate, is:

$$C_r = 400N \text{ (kN/m}^2\text{)}$$

where N = the uncorrected S.P.T. value.

To emphasise the shortcomings of the formula the reader's attention is drawn to the work of Meigh and Nixon (1961) who showed that, over a number of sites, C_r values varied from 430N to 1930N (kN/m²).

The pressuremeter

Both the Standard penetration test and the Dutch cone penetration test have the advantages of cheapness and simplicity of operation.

The Dutch cone test is particularly useful in that it can provide a continuous profile of soil penetration resistance to depth.

However, the interpretation of the results from both tests involves the use of empirical rules which may, or may not, lead to accurate results.

It has been realised for a long time that, for relatively homogeneous soil conditions, the only in situ test that gives results that can be analysed to yield reasonably accurate estimations of soil parameters is the plate loading test. This test, briefly described in the next section, has the disadvantages of being both costly and time consuming.

A recent development in in situ testing techniques is the pressuremeter which can provide a quicker and cheaper form of test.

Originally developed by Menard (1957) the pressuremeter consists of an inflatable rubber bag which is lowered down a borehole in the soil to be tested. The bag is inflated with nitrogen against the sides of the borehole and, by measuring gas pressure and strain effects, it is possible to determine such soil properties as elastic modulus and undrained shear strength.

The main disadvantage of the Menard pressuremeter is that it can only be used down a borehole and the quality of the results obtained is heavily dependent upon the degree of disturbance suffered by the soil forming its sides.

Since its inception the pressuremeter has been subjected to modification by various workers but a major improvement has resulted from work carried out by Baguelin at the Laboratoire des Ponts et Chaussées, Paris (1974) and by Wroth at Cambridge University (1975). They have modified the apparatus to make it self boring. Two models of the self boring pressuremeter exist, one on each side of the Channel, but the basic operation of both is very similar.

It is claimed that the self boring pressuremeter has the advantages that the soil into which it is introduced suffers an almost negligible amount of disturbance. It is possible to obtain several soil parameters, such as undrained shear strength, Young's Modulus and the lateral earth pressure coefficient, K_0, all from the use of proven theories rather than from empirical rules.

The disadvantages appear to be that the apparatus is fairly complex, requires specialist operators and, until its durability has been proved, cannot be considered as being very robust. It is also not possible for the instrument to penetrate hard layers such as dense gravel, mudstone, etc.

The plate loading test

This is a semi-direct means for the determination of the bearing capacity of a relatively homogeneous soil and, whilst generally used for granular soils, can be used on clays.

The soil is excavated down to the proposed foundation level and a model footing, made up from thick plates of steel, is subjected to an increasing vertical stress. The settlement at various steps in the loading is noted.

The test plate is generally 305 mm square but round plates, and larger sizes, are sometimes used. Typical test arrangements and the results obtained are illustrated in fig. 1.7.

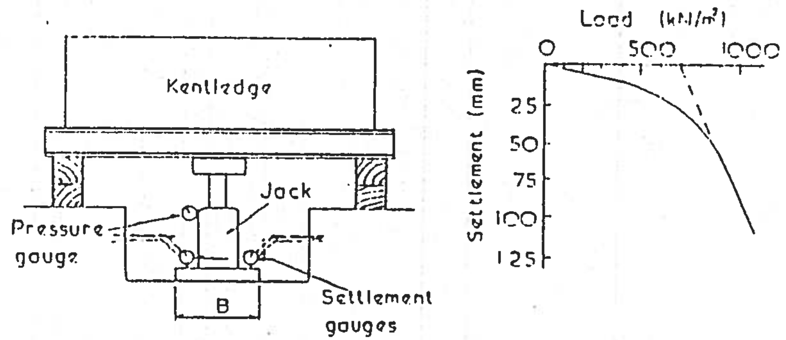


Fig. 1.7 Plate loading test

The ultimate bearing capacity of the soil is taken either as the loading pressure at which the load/settlement relationship approaches the vertical or that which produces a settlement of 25 mm.

With granular soils the usual problem is to determine the allowable bearing pressure for a particular foundation and it is generally taken to be the pressure that will cause an average settlement of the foundation of 25 mm.

The settlement of a square footing, kept at a constant pressure, increases as the size of the footing increases.

Terzaghi and Peck (1948) produced the following relationship:

$$S = S_1\left(\frac{2B}{B + 0.3}\right)^2$$

where S_1 = settlement of a loaded area of 0.305 m × 0.305 m under a given loading intensity of p

S = settlement of a square, or rectangular, footing of width B m under the same pressure p.

39

Knowing the size of the footing, B and the fact that S = 25 mm, it is possible to work backwards and obtain S_i. The test pressure corresponding to S_i is the allowable bearing pressure. The process is one of iteration, starting with an assumed value for B.

It should be noted that the zone of soil affected by a plate load test is much smaller than will be affected beneath the full foundation. Horizontal and vertical variations in the soil may not be reflected in the plate load test results and it is useful to have borings taken at the load test site to check that the soil is sufficiently homogeneous for plate load tests to be realistic.

Procedure in foundation design

The bearing capacity of a proposed foundation can be obtained either by inserting measured soil parameters into the relevant bearing capacity formula or from the results of in situ tests.

When no information is available at the start of a design project reference can always be made to tables of safe bearing capacities prepared by various authorities.

Table 1.4 lists typical safe bearing capacities (based on CP 2004 'Foundations') that can be associated with particular soil types.

With the use of such values a preliminary estimate of the foundation size can be made. At this stage the foundation depth should be decided upon from a consideration of such factors as the depth of frost penetration, position of the water table and the location of a suitable soil bearing stratum. Afterwards the size of the foundation can be checked in terms of soil parameters or in situ test results, as described previously.

Table 1.4 Values of safe bearing capacity (kN/m²)

Soil type	Dry	Submerged
Compact well graded sands and gravel-sand mixtures	500	250
Loose well graded sands and gravel-sand mixtures	300	150
Compact uniform sands	300	150
Loose uniform sands	160	80
Very stiff boulder clays and hard clays with a shaly structure		500
Stiff clays and sandy clays		300
Soft clays and silts		160
Very soft clays and silts		25

Settlement of foundations

Note. This subject has been fairly well covered in chapters 9 and 10 of the companion volume to this text, *Elements of Soil Mechanics for Civil and Mining Engineers*, and only a brief summary will be presented here.

(a) *Cohesive soils*

(i) Immediate settlement

An expression for the *mean* settlement of a reinforced concrete surface foundation was suggested by Skempton in 1951.

$$\rho_i = \frac{pB(1-\mu^2)I_p}{E}$$

where p = uniform contact pressure
B = width of foundation
E = modulus of elasticity of soil
μ = Poisson's ratio of soil
I_p = influence factor depending upon the dimensions of the foundation, i.e. length, L and width, B.

The values for I_p suggested by Skempton are given in table 1.5.

Table 1.5

L/B	I_p
circle	0.73
1 (square)	0.82
2	1.00
5	1.22
10	1.26

Typical values for μ are:

Saturated clay	0.4–0.5
Partially saturated clay	0.1–0.3
Silt	0.3–0.35
Sand	0.2–0.4

The above formula, based on the theory of elasticity, applies to a

E45

footing supported on a semi-infinite soil, i.e. in practical terms, a soil layer of not less than 2B (ideally, not less than 4B) in thickness.

Janbu *et al.* (1956) proposed a formula for the mean settlement, ρ_i, when the consolidating layer is of a finite thickness H:

$$\rho_i = \mu_0 \mu_1 pB \text{ (for saturated clays).}$$

Values of the coefficients μ_0 and μ_1 are dependent upon the ratio H/B and L/B and are presented in fig. 1.8.

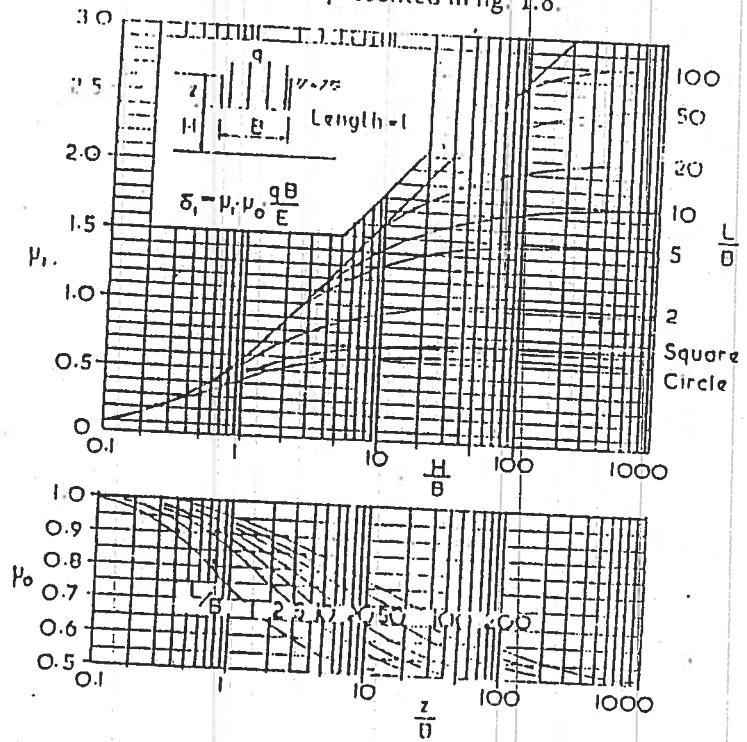


Fig. 1.8 Coefficients μ_0 and μ_1 for the determination of the mean immediate settlement of a foundation (after Janbu *et al.*, 1956)

When the soil deposits beneath a foundation consist of a series of layers the mean settlement of the foundation can be obtained from the principle of superposition, as shown in the following example.

Example 1.7

A 7 m thick layer of clay has a value of E = 25 MN/m² and overlies a further 4 m thick layer of clay which has E = 40 MN/m². The lower

clay overlies bedrock which can be regarded as incompressible.

A square foundation slab of dimensions 5 m × 5 m is to be constructed at a depth of 2 m below the surface of the upper clay and will impose a uniform bearing pressure of 85 kN/m².

Estimate the value of mean immediate settlement of the foundation.

The problem is illustrated in fig. 1.9.

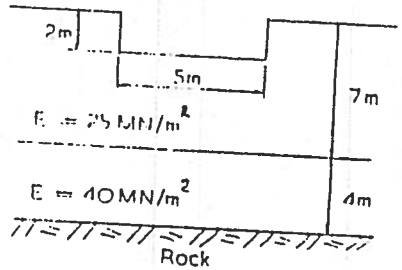


Fig. 1.9 Example 1.7

Consider upper layer of clay:

$$z/B = 2/5 = 0.4; L/B = 5/5 = 1.0 \text{ and } H/B = 5/5 = 1.0.$$

From fig. 1.8, $\mu_0 = 0.9$ and $\mu_1 = 0.45$

$$\therefore \rho_i = 0.9 \times 0.45 \times \frac{85 \times 5}{25} = 6.9 \text{ mm.}$$

This is not the total settlement of the foundation as there will be some compression in the lower clay. This can be found by the following procedure:

If this lower clay had extended from the ground surface for the full depth of 11 m, then z/B = 0.4; L/B = 1.0 and H/B = 9/5 = 1.8, which gives $\mu_0 = 0.9$ and $\mu_1 = 0.55$.

Hence ρ_i would have been $0.9 \times 0.55 \times 85 \times \frac{5}{40} = 5.3$ mm.

If the clay had extended from the ground surface to the 7 m depth, then z/B = 0.4; L/B = 1.0 and H/B = 5/5 = 1.0 which gives $\mu_0 = 0.9$ and $\mu_1 = 0.45$ and ρ_i would have been

$$0.9 \times 0.45 \times 85 \times \frac{5}{40} = 4.3 \text{ mm.}$$

EA1

Hence ρ_i due to compression of lower clay = 5.3 − 4.3 = 1.0 mm.

∴ Total mean immediate settlement of the foundation = 6.9 + 1.0 = 7.9 mm.

Estimation of the value of the modulus of elasticity, E

The determination of a realistic value for E is critical in the calculation of ρ_i by the above methods. Such a value is best obtained from in situ tests, such as the pressuremeter or the plate loading test.

It may also be estimated from undrained triaxial tests with the test sample firstly consolidated with a cell pressure of between 0.5 and 0.67 times the in situ effective overburden pressure. The value of E is taken to be the slope of the stress/strain plot, or secant modulus, over the stress range applicable to the problem.

E is also roughly related to the undrained shear strength of clay but the relationship can vary considerably. For normally consolidated clays, E lies somewhere between 500 c_u and 1500 c_u, the lower end of the range applying to clays of high plasticity.

(ii) Consolidation settlement

The consolidation of a clay layer of thickness, H, is given by the expression:

$$\rho_c = \mu m_v H \Delta\sigma_z$$

where μ = the consolidation factor
m_v = coefficient of volume compressibility
$\Delta\sigma_z$ = average increase in vertical pressure within the layer.

The consolidation factor, μ, depends upon both the pore water pressure parameter, A and the shape of the foundation. Skempton and Bjerrum (1957), on the assumption that m_v and A are constant with depth, evolved the equation:

$$\mu = A + (1 - A)\alpha.$$

Values of α are given in table 1.6.

Table 1.6

H/B	Circular footing	Strip footing
0	1.00	1.00
0.25	0.67	0.74
0.5	0.50	0.53
1.0	0.38	0.37
2.0	0.30	0.26
4.0	0.28	0.20
10.0	0.26	0.14
∞	0.25	0

Typical values for μ are:

Soft sensitive clays – possibly greater than 1.0.
Normally consolidated clays – generally less than 1.0.
Average overconsolidated clays – approximately 0.5.
Heavily overconsolidated clays – perhaps as little as 0.25.

In the case of a deep layer of clay only the upper thickness of depth 4B need be analysed. The procedure is to divide the thickness of 4B into a convenient number of horizontal slices and consider the changes in vertical pressure within each one. The total consolidation is taken as the summation of the individual settlements of each slice (see example 1.9).

(b) *Cohesionless soils*

As already discussed, the prediction of the settlement of foundations on sand are based on in situ test results.

(c) *Buoyant, or compensated foundations*

If the predicted settlement of a shallow foundation is too large to be acceptable it is often possible to effect a reduction by placing the foundation at a greater depth.

When the depth of the foundation is such that the weight of excavated soil is equal to the proposed structural load then, at foundation level, the vertical stress on the soil will have the same value both before and after construction. Such a foundation is said to be fully compensated and, in theory, will experience no settlement. It is rarely necessary to design for full compensation as most structures can accept some settlement.

An excavation involves stress relief which can cause swelling in the remaining soil. Swelling effects are negligible in sands but can be of significance in certain types of clay. The extent of swelling is largely dependent upon the availability of moisture and the length of time that the foundation soil is left unloaded. It is therefore necessary, for buoyant foundations in clay, to attempt to keep the excavation dry and to construct the foundation as quickly as possible. If swelling has occurred even a fully compensated foundation will suffer some settlement effects as the soil will recompress when loaded.

There is a physical limit to the depth at which a foundation can be placed. Even if the sides of an excavation can be successfully supported, the weight of material adjacent to the foundation will eventually cause shear failure within the soil.

E42

Skempton (1951) proposed that the limiting, or critical depth of excavation, z_c, for clays could be obtained from the formula:

$$z_c = N_c \frac{c_u}{\gamma}$$

where c_u = undrained shear strength of soil
N_c = bearing capacity coefficient (obtained from fig. 1.2)
γ = unit weight of soil

and that, for a particular depth of excavation, z, the factor of safety against bottom heave was:

$$F = N_c \frac{c_u}{w_s + \gamma z}$$

where w_s = any surcharge load acting on the surface of the soil.

Example 1.8

A building is 50 m square in plan and is to be constructed within a deep bed of soft clay which has an undrained shear strength of 18 kN/m² and a saturated unit weight of 18 kN/m³.

The water table is at the surface of the clay which is overlain by a 1 m thick layer of sand of unit weight 16 kN/m³.

To keep settlement effects within acceptable limits it is necessary to ensure that the net increase in foundation pressure will not exceed 50 kN/m².

Determine the necessary foundation depth if the total applied load will be 300 MN.

$$\text{Gross foundation pressure} = \frac{300 \times 1000}{50 \times 50} = 120 \text{ kN/m}^2$$

$\therefore$ required pressure relief due to soil excavation $= 120 - 50 = 70 \text{ kN/m}^2$.

If D is the required depth of the foundation below the top of the clay bed,

$$1 \times 16 + D \times 18 = 70$$

whence $D = 54/18 = 3.0$ m (and $z = 4$ m).

Although this answers the question, it is necessary to check the factor of safety against bottom heave.

$z/B = 4/50 \approx 0$ $\quad\therefore N_c$ (from fig. 1.2) = 6.2

$$\therefore F = \frac{6.2 \times 18}{70} = 1.6 \text{ which is satisfactory.}$$

Note. The value of F that can be regarded as satisfactory will depend upon the site conditions, e.g. the standard of workmanship and the time taken to complete construction. For most practical problems it should not be less than 1.2.

Foundations of offshore structures

Introduction

The seventies saw the development of the North Sea oil industry which involved the construction of massive production platforms and the installation of wells up to 10 km deep.

The size of the platforms is difficult to comprehend for anyone not involved in the industry but a dramatic description of a concrete production platform, the Norwegian 'Condeep', was given in the *New Civil Engineer* in 1974. The magazine suggested to its readers that the best way to appreciate the size of 'Condeep' was to think of a foundation as thick as the height of Nelson's column and with a plan size equal to the area of Trafalgar Square. On top of this mammoth slab are placed three concrete columns, each as high as the G.P.O. tower, and on top of these columns is placed a concrete platform, the size of a football pitch.

Design loads

Any structure placed in an offshore location must be designed to withstand the maximum forces it is liable to be subjected to during its lifetime. There is sufficient statistical evidence for the North Sea to be able to predict that the maximum wave liable to occur within a one hundred year period will have the following characteristics:

Height: 24–30 m, depending upon water depth at location.
Period: 14–18 s.
Wavelength: 600 m, giving a wave velocity of some 135 km/h or 85 m.p.h.

If the mean sea level is considered to be the line midway between the trough and the crest of the one hundred year wave then it can be

Project	207 SUMATRA RD, NWS	Job ref	1381
Part of structure	SETTLEMENTS	Calc sheet no / rev	E1 44
Drawing ref	Calc by / Date	Check by	Date

Ref	Calculations	Output

NEW BASEMENT SETTLEMENTS

REFER TO PAGES E29 TO E43.

			D	L
TOTAL LOAD ONTO RAFT	=	2716.5kN	2063.7	652.5
ADD RAFT	=			
24 × 0.300m DEEP × 9.5 × 6.15	=	420.7kN	420.7	
LIVE 1.5 × 9.5 × 6.15	=	87.6		87.6
		3224.6 kN	2484.4	740.2

"PARTIALLY" SATURATED CLAY SOIL = 20 kN/m²

G.W.T. APPEARS TO BE BELOW 6m DEEP.

SPT values, say, AT 1m = 120 (N)

AT 4.0m = 170 (N)

EFFECTIVE OVERBURDEN AT 1m

1.0 × 20 = 20 kN/m²

EFFECTIVE OVERBURDEN AT S.B.

4.0 × 20 = 80 kN/m²

FIGURE 1.4

CORRECTED N 20 × 2.0 = N' — 40 kN/m²

N 80 × 0.95 = N' — 76 kN/m²

AVERAGE N' VALUE $\frac{40+76}{2}$ = 58.0 kN/m²

ALLOWABLE DESIGN BEARING FROM 1.13

= 580 kN/m²

"ALLOWABLE" DO NOT INCREASE BY 50%

Project	207 SUMATRA RD, NW6	Job ref	1381
Part of structure	SETTLEMENTS	Calc sheet no / rev	C, 45,
Drawing ref	Calc by: WM / Date: OCT 2012	Check by	Date

Ref	Calculations	Output
	TOTAL FOUNDATION LOAD ALLOWABLE	FOR 25mm SETTLEMENT
	$\frac{3500 \text{ SAY}}{1000} \times 9.5 \times 6.15 = 33.9 \text{MN}/m^2$	
	ACTUAL LOAD $= 3.225 \text{ MN}/m^2$	
	ACTUAL DESIGN SETTLEMENT	
	$\frac{3.225}{33.90} \times 25 = 2.38$ mm.	
	ACTUAL SETTLEMENT TENDS TO BE 1/3 DESIGN VALUE	
	$2.38/3 = 0.8$mm.	NEGLIGABLE.
	HORIZONTAL MOVEMENT RESISTED BY 9.5m x 6.15m "WAFFLE" RAFT FOUNDATION.	
	THEREFORE, NEGLIGABLE HORIZONTAL MOVEMENT AS :—	
	1/ TERRACE HOUSE EXTENDED EAST-WEST.	
	2/ WALL AREAS ADD TO STIFFNESS OF BASEMENT FRAME ETC	
	3/ GROUND & FIRST FLOOR "BOXING" IS MAINTAINED ETC	
	SAY BASEMENT DESIGN OK.	