Structural Calculation

Revision B

71 Avenue Road, Swiss Cottage,
London, NW8 6HP

18 August 2023

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House in reasonable condition

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Structural Report

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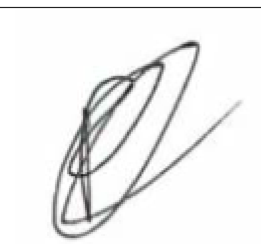
X Zubair Sherwani Structural Engineer		Mehrdad Digitally signed by Mehrdad DN: cn=Mehrdad , c=GB, ou=Adkins Consultants, email=m.mehrabani@adkinsconsultants.com Reason: HOO Form Consent Verification Location: UK Date: 2023.08.18 13:20:51 +01'00'
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This is a supplementary report, to be read in conjunction with the soil report from GROUND AND WATER

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Safety

All site operatives are to be made aware of this information if at any time health and safety concerns arise on site about any matter related to our design. Work is to be halted and we are to be contacted immediately.

Whilst we have made every attempt to design out all risks with our design, some risks relating to the construction, maintenance or demolition of the structure may remain. Residual risks are detailed below with our assessment of how these risks can be managed.

TYPE OF RISK	Method of risk reduction	Risk Level	
		Low	High
Site access	Safe route to works to be put in place by contractor		✓
Demolition	Contractor to survey existing building, plan sequencing, demolition and temporary support		✓
Temporary support	Contractor to provide all necessary temporary support for vertical and lateral loads		✓
Manual handling	Contractor to provide mechanical lifting equipment for items over 20KG and where manual lifting tasks are awkward		✓
Services	Locations of all existing services to be confirmed prior to start of works		✓

Design Notes

Dimensions used in the following calculations are for design only and shall not be used for constructional purposes.

Any construction details and dimensions indicated in the following calculations shall not be varied unless they are substantiated by calculation, and approved in writing by the Engineer.

The Eurocodes, British Standards and Codes of Practice highlighted in the table below have been used in the preparation of these calculations all constructional details must be in accordance with all relevant clauses contained in these same standards or associated standards.

It is the responsibility of the Contractor to design and provide all necessary temporary propping to ensure that the structural integrity of the dwelling is maintained at all times. Mild steel shims to be rammed in tight between top flange of steel and underside of structure above to ensure full load transfer is achieved prior to the removal of any temporary propping.

<u>Steelwork</u>
Steel sections to be to BS 4: Part 1 or BS EN 4848 as appropriate made from steel to BS EN 10025.
All steelwork to be hot rolled Grade S355JR except otherwise stated.
Do not use steel sections which are heavily pitted or rusted.

Loading tables

Timber Stud wall loading – Internal	
Loads	DL (kN/m^2)
Studs	0.12
Noggins	0.06
2 Layers PBD	0.32
Total	**0.50**

Timber Stud Wall loading – External	
Loads	DL (kN/m^2)
Concrete tiles	0.75
2 x 18mm Ply	0.28
Studs	0.12
Noggins	0.06
2 Layers PBD	0.32
Total	**1.53**

Masonry Cavity Wall – External with 100mm bwk, 100mm blk	
Loads	DL (kN/m^2)
103 Brick	2.25
Plaster	**0.25**
100 Block – Light weight	1.55
Total	**4.1**

Masonry Cavity Wall – External with 210mm internal block	
Loads	DL (kN/m^2)
103 Brick	2.25
Plaster	0.25
210 Collar-Jointed – Dense	4.60
Total	**7.10**

Masonry wall – Internal	
Loads	DL (kN/m^2)
103 Brick	2.40
100 Block – Lightweight	1.65
100 Block – Dense	2.30
140 Block – Lightweight	2.00
140 Block – Dense	3.30
210 Collar – Jointed – Lightweight	3.60
210 Collar – Joined – Dense	4.60

Flat Roof- Timber		
Loads	DL (kN/m^2)	LL (kN/m^2)
3 Layer Felt	0.12	
50mm Insulation Board	0.15	
25mm Ply	0.19	
Firings	0.10	
Joists (200x50@400c/c)	0.14	
Ceiling + Services	0.25	
Imposed		0.95
Total	**0.95**	**0.95**

Pitched Roof- Timber		
Loads	DL (kN/m^2)	LL (kN/m^2)
Tile Finish	1.40	
Slate Finish	1.00	
Imposed		0.75

(Tank Allowance = 0.25kN/m^2)

Floor- Timber		
Loads	DL (kN/m^2)	LL (kN/m^2)
25mm Ply	0.22	
Joists (200 x 50@ 400c/c)	0.12	
Ceiling + Services	0.20	
15mm Plasterboard	0.18	
Imposed (residential)		1.50
Total	**0.72**	**1.50**

Floor- Concrete Ground Floor		
Loads	DL (kN/m^2)	LL (kN/m^2)
75mm Screed	1.70	
100mm Insulation	0.10	
150mm Pre-Cast Beam & Block	3.10	
Imposed (residential)		1.50
Total	**4.90**	**1.50**

Project Description and Overview

		Notes
Client:	Mr. Meir N Gareh	
Site Visit Date	7 February 2023	
Existing Property	Masonry	
	Suspended Ground Floor (Timber)	
	Reasonable condition	
Ground Investigation		FROM GROUND AND WATER
Proposed Scheme Area	Basement Development	
Proposed Scheme	Other	
	Concrete frame	
Trees Impact on Proposed Foundations	Significant Front of the 71 Avenue Road is the region where tree protection must be addressed	
Drainage Impact on Proposed Foundations	Standard Detail	

I. Introduction

A. Purpose of the report

This report describes the projected three storey dwelling house along with basement development, covering all temporary and permanent work components that might have an influence on geology, hydrogeology, hydrology, and land stability.

B. Background information on the project

The site is not registered as a historic building; however, the surviving structure is a detached home built before 1871. The building was in disrepair. The home had a tiled roof, and brickwork fence, and was built of masonry.

The change was made during the course of the property's existence. The client intends to demolish the property since doing so will allow for the development of a structure with long-term viability and appropriate cost given its probable lifespan. The surrounding properties should be considered when the building is being done.

C. Overview of the basement design

According to the suggested drawing, a new exterior staircase at the front of the property and a new internal stair the ground-floor hallway would be used to access the new basement.

Through glass units built into the floor, natural daylight will enter the basement from the front and back of the building. The new basement will have a new kitchen, a shower area, and bedrooms as shown in figure 1(Architect, 2019a).

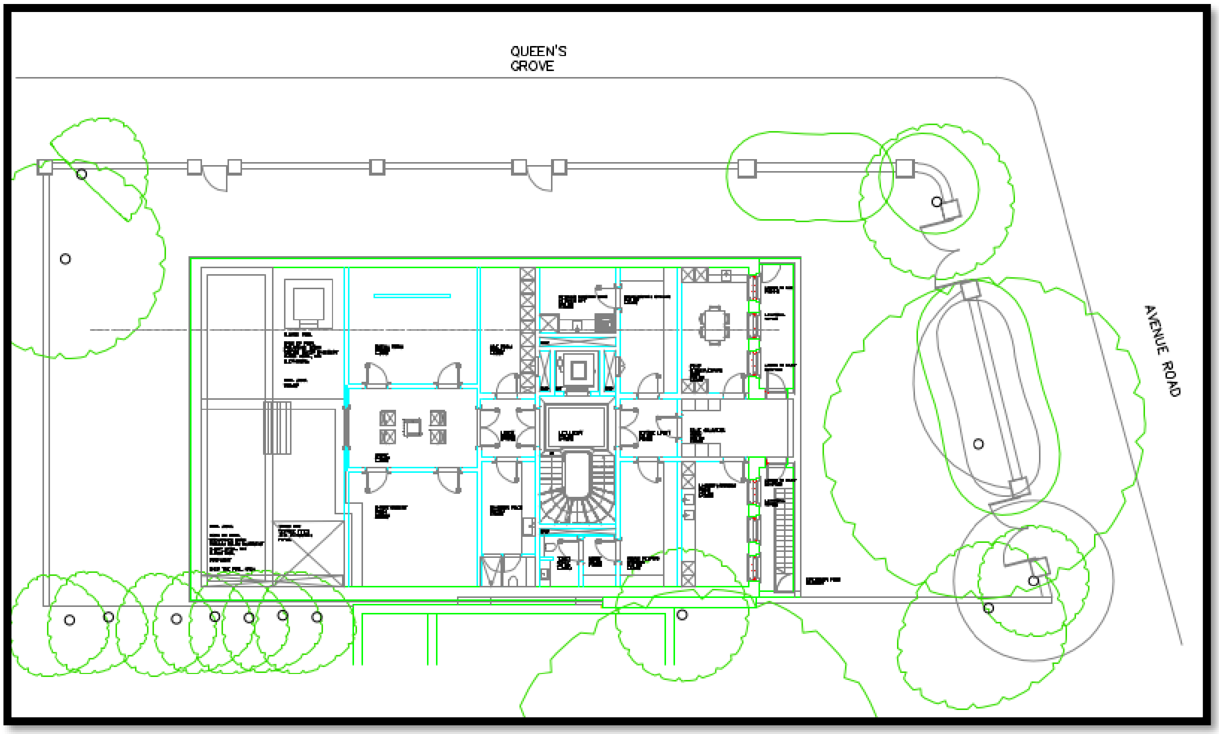

Figure 1 Basement Plan outline in green(Architect, 2019a)

II. Site Analysis

A. Location of the basement

The location is near the intersection of Avenue Road and Queens Grove. No.69 Avenue Road and 37A Queens Grove are located on the east and south sides, respectively. The planned basement and structural renovation at 69 Avenue have been approved. Avenue Road and Queens Grove form the site's northern and western boundaries, respectively. Several valuable trees on the land will be preserved see figure 2.

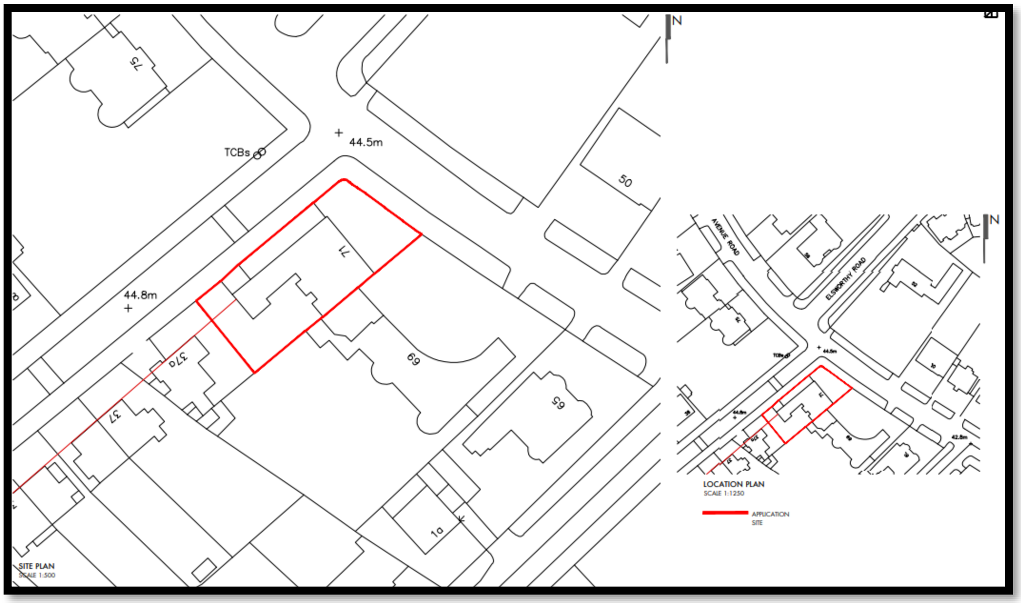

Figure 2 Site plan and location(Architect, 2019c)

B. Soil analysis and foundation requirements

The trial holes were logged by a Ground and Water Limited representative, generally in accordance with BS EN 14688 'Geotechnical Investigation and Testing – Identification and Classification of Soil'. The ground conditions encountered within the trial holes constructed on the site generally conformed to that anticipated from examination of the geology map as the London Clay Formation was encountered below a capping of Made Ground.

The succession of conditions and description of soils encountered in the trial holes in descending order is tabulated below refer to ground and water soil report.

Summary of Strata Encountered			
Strata	**Top Depth (m AOD)**	**Base Depth (m AOD)**	**Thickness (m)**
MADE GROUND (all trial holes): Varicoloured sandy gravelly CLAY. Sand was fine to coarse. Gravel was fine to coarse, angular to sub-rounded concrete, brick and flint.	GL	0.20 – 1.80	0.20 – 1.80
LONDON CLAY FORMATION (WS1 and WS2): Orange brown gravelly silty CLAY. Gravel is fine to coarse, angular to sub-rounded flint.	0.20 – 0.90	1.60 – 3.20	1.40 – 2.30
LONDON CLAY FORMATION (all trial holes): Orange brown silty CLAY with grey mottling.	0.80 – 3.20	9.00 – 5.00	1.80 – 7.40

Groundwater monitoring was undertaken on two occasions to date, where the standing water level was noted to be between 1.60 – 4.40m Below Ground Level. The majority of the area is flat terrain, with no cuts or valleys. As a result, slope instability will be ignored the design. below ground level.

The foundation requirement consists of raft and contagious piles. It has been discussed in details further in the report see figure 3.

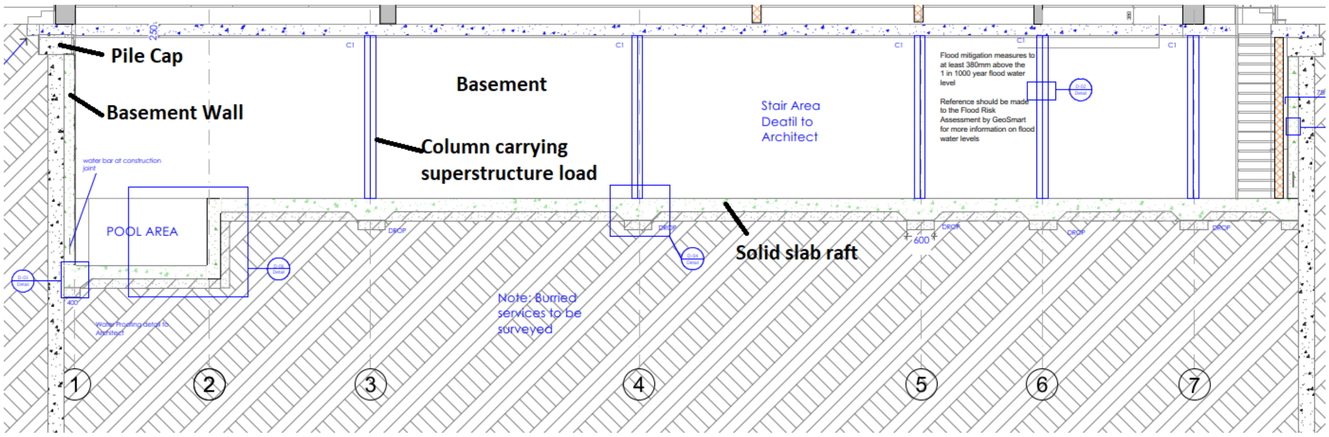

Figure 3 Foundational detail of the basement

No significant instability issues related with granular soils are expected and no instability issues were observed during the ground investigation. If instability is noted within the sands and gravels or within Made Ground, the following could be applied for good workmanship and mitigation of any risk.
It should be noted that these are indicative. Specific measures should be included in a competent Construction Method Statement for the works on this site by the structural engineer and the contractor. Where soft/loose spots are encountered, trench sheets should be left in. Alternatively, a back prop with precast lintels or sacrificial boards should be installed. If the soil support to the ends of the lintels is insufficient, brace the ends of the PC lintels with 150x150 C24 timbers and prop with Acrows diagonally back to the ground. Where voids are present, behind the lintels or trench sheeting, voids behind sacrificial propping should be grouted. Grout should be 3:1 sand/cement and packed into the voids. Prior to casting, a layer of DPM should be installed between trench sheeting (or PC lintels) and new concrete. The lintels should be cut into the soil by 150mm either side of the pin. A site stock of a minimum of 10 lintels should be present to prevent delays due to ordering (Soil report, 2023).

C. Topographical survey and site conditions

Figure 4 depicts existing site limits, particularly in amber. That is the region where tree protection must be addressed (Bell, 2023).

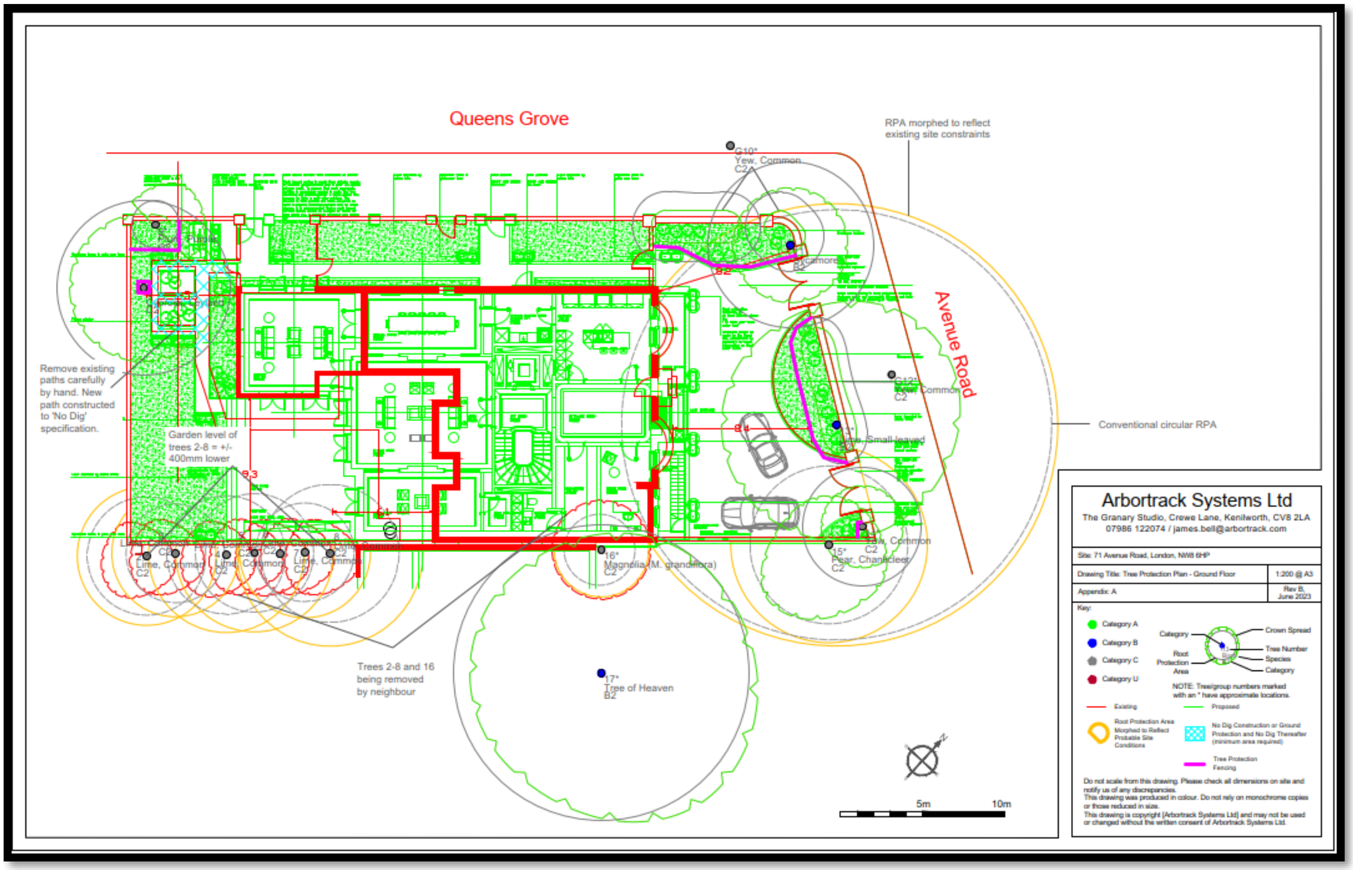

Figure 4 Tree protection area(Bell, 2023)

The RPA is an ideal zone of protection surrounding the roots system (see figure 4) of the trees, and it is indicated on the design. The RPAs will be located within the TPZ and hence totally walled off (see figure 4) unless suitable ground protection is provided.

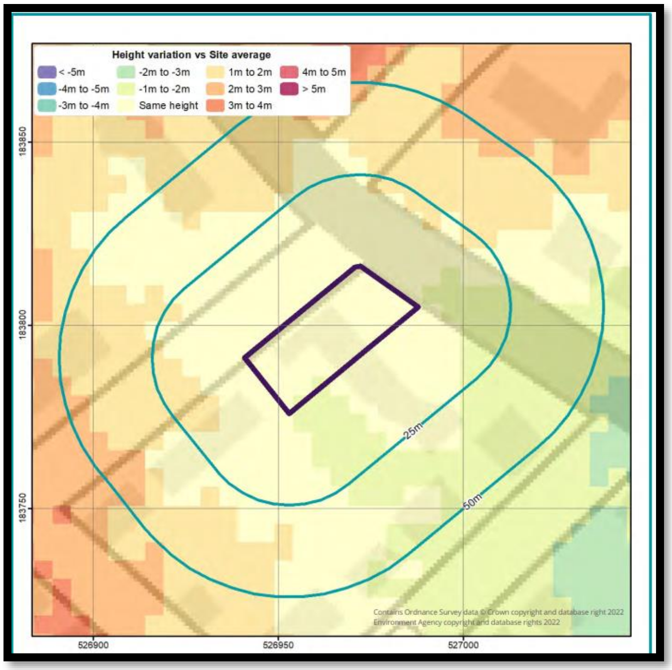

Figure 5 Site topography(GeoSmart, 2022)

The Site's topography has been assessed using LiDAR DTM5 elevation data to detect the general slope and any localised depressions. The mapping compares typical ground levels at the Site to ground levels in the surrounding region. The mapping reveals that the overall Site is mostly flat, with elevations ranging from 44.6 m AOD to 44.0 m AOD. This is based on EA elevation data for the Site at a resolution of 1 m and a vertical accuracy of 0.15 m (GeoSmart, 2022).

D. Local building codes and regulations from the council and Eurocode .

The Eurocodes and British Standards highlighted below in table 1 and have been used in the preparation of these calculations - all constructional details to be in accordance with the relevant clauses contained in these or associated standards whereas The CDM regulations can be found at the end of the report.

Table 1 Eurocodes

Code	Title
Basis for structural design	
BS EN 1990: 2002	Eurocode Basis of structural design
NA to BS EN 1990: 2002	UK National Annex for Eurocode Basis of structural design
Actions on structures	
BS EN 1991-1-1: 2002	Eurocode 1 General actions – Densities, self-weight, imposed loads for buildings
NA to BS EN 1991-1-1: 2002	UK National Annex to Eurocode 1 General actions – Densities, self-weight, imposed loads for buildings
BS EN 1991-1-3: 2003	Eurocode 1 General actions – Snow loads
NA to BS EN 1991-1-3: 2003	UK National Annex to Eurocode 1 General actions – Snow loads
BS EN 1991-1-4: 2005	Eurocode 1 General actions – Wind actions
NA to BS EN 1991-1-4: 2005	UK National Annex to Eurocode 1 General actions – Wind actions
Design of steel structures	
BS EN 1993-1-1: 2005	Eurocode 3 General rules and rules for buildings
NA to BS EN 1993-1-1: 2005	UK National Annex to Eurocode 3 General rules and rules for buildings
Design of timber structures	
BS 5268: Part 2: 2002	Structural use of timber Code of practice for permissible stress design, materials and workmanship
Design of masonry structures	
BS 5628: Part 1: 2005	Code of practice for the use of masonry Structural use of unreinforced masonry
Geotechnical design	
BS EN 1997-1: 2004	Eurocode 7 General rules
NA to BS EN 1997-1: 2004	UK National Annex to Eurocode 7 General rules

III. Basement Design

A. Floor plan layout/ Materials and Construction

To support the building above, the basement will have cast in situ piles all around the perimeter. Reinforced concrete will be used for the basement's main wall. The raft will be placed to support basement columns and partition walls as shown in figure 6 (Adkins, 2023a).

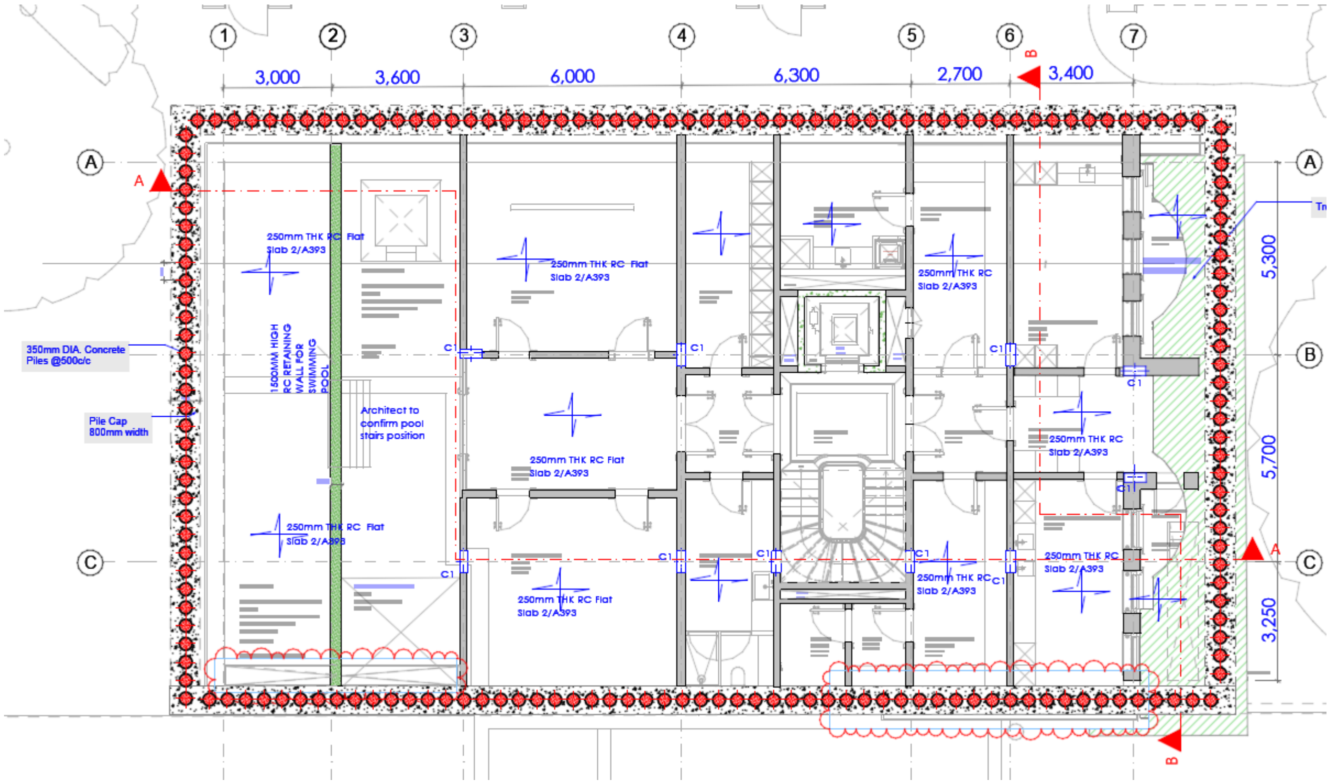

Figure 6 Basement plan with structural elements(Adkins, 2023a)

Before installing the inner skin of the blockworks (100mm wide & 7.3N dense) wall, a 50mm air cavity must be established see illustration in figure 7 (Adkins, 2023b).

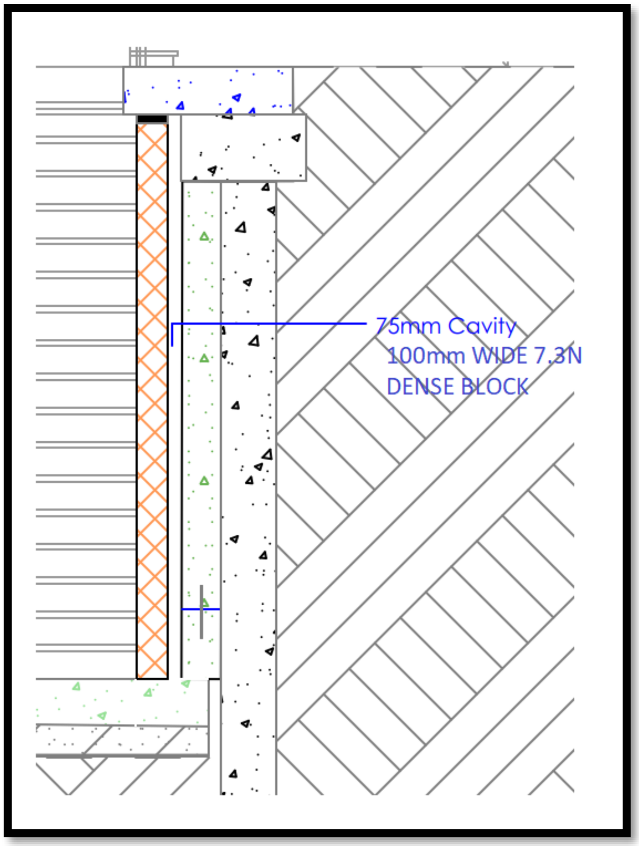

Figure 7 Section detail for outer and inner skin of basement wall (Adkins, 2023b)

The about 250mm thick basement slab that is reinforced with A393 mesh at the top and bottom takes on the partition wall weight of the building above the basement, leaving the outside wall load to be carried on piles. The basement flat slab will be supported by 12 or more 250x600 cross-sectional dimension reinforced columns that are distributed uniformly throughout the design and flush with the thicknesses of the internal walls (Adkins, 2023a).

B. Ceiling height and finishes.

Floor-to-floor height is specified at 4280mm. whereas the height from floor to ceiling is 3600mm as shown in figure 8 (Adkins, 2023b; Architect, 2019b).

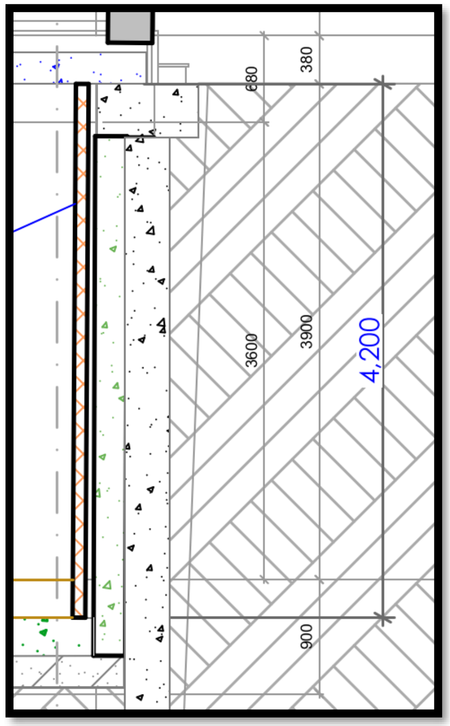

Figure 8 Showing height related dimensions for basement (Adkins, 2023b; Architect, 2019b)

C. Water Proofing system:

When it comes to waterproofing a basement there are several waterproofing systems to consider. Here are three different options to consider:

Tanking System:

A tanking system involves creating a waterproof barrier on the inside or outside of the basement walls. This can be achieved through the application of a waterproofing coating, or by installing a waterproof membrane. The system needs to be properly installed by a qualified contractor, and it's important to ensure that the materials used meet British Standards see figure 9.

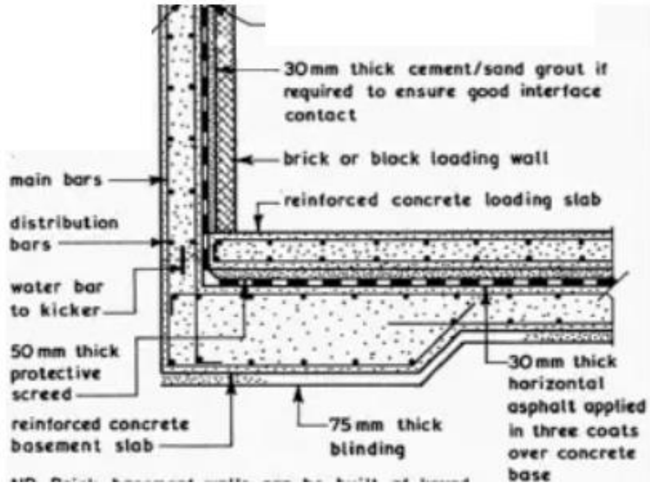

Figure 9 Waterproofing detail for internal tanking system

Assessment and Survey: A professional waterproofing specialist assesses the basement to identify the extent of water ingress and determine the appropriate tanking system.

Surface Preparation: The walls and floors of the basement are prepared by removing any loose materials, such as old paint or plaster, and cleaning the surfaces to ensure proper adhesion of the tanking materials.

Priming: A primer is applied to the prepared surfaces to enhance adhesion and create a suitable bonding layer for the tanking system.

Waterproofing Membrane: It can be done through asphalt or a liquid-applied or sheet waterproofing membrane is installed on the walls and floor of the basement. The membrane acts as a barrier to prevent water penetration. Liquid-applied membranes are often used as they can be easily applied to irregular surfaces and provide seamless protection.

Coving and Fillets: Coving, which is a curved waterproofing detail, is applied at the wall-floor junction to create a smooth transition and ensure complete waterproofing. Fillets, which are triangular-shaped seals, are also installed in corners and other joints to provide additional reinforcement.

Drainage System: A drainage system is typically installed along the basement walls to collect any water that may enter the structure. This system directs the water towards a sump pump or other drainage outlets to remove it from the basement.

Finishing: Once the tanking system is in place, the surfaces can be finished with appropriate materials, such as plasterboard or a suitable render, to create a habitable space.

D. Excavation with permanent embedded retaining walls to deep basement (Contiguous Bored Pile RW)

As piles are normally put at centres 150mm bigger than their diameter, piling produces gaps in structural walls where soil is exposed during excavation. This option is suitable when the retained soil is normally firm to stiff as in our case (not frequently granular) . This is often the fastest and cheapest option for construction.

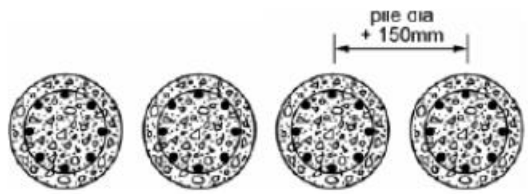

Figure 10 Contagious pile system

E. Dewatering:

As the water table is quite high and hence it is necessary to think about dewatering. During the construction stage of a new basement in London, a sump pump system can be utilized to manage groundwater and maintain a dry working environment. The procedure typically unfolds as follows:

- Excavation and Sump Pit Installation: As the basement excavation progresses, a sump pit is excavated at the lowest point of the construction area. The pit is carefully constructed with sturdy materials, such as concrete, to ensure stability and prevent collapse.

- Sump Pump Installation: Once the sump pit is ready, a sump pump is installed within it. The pump consists of a motor, impeller, and float switch. The motor drives the impeller, which helps in pumping out water, while the float switch automatically activates the pump when the water level rises above a certain threshold.

- Perimeter Drainage System: A perimeter drainage system is implemented around the basement walls. This system involves the installation of perforated pipes along the perimeter, which collect groundwater that enters the construction area through the surrounding soil.

- Pump Activation and Water Removal: As construction progresses, groundwater seeps into the construction area and is channeled towards the perimeter drainage system. The collected water flows into the sump pit. Once the water level reaches the predetermined height, the float switch triggers the sump pump, activating the motor and impeller. The pump then draws water from the pit and pumps it out of the construction area.

- Discharge of Pumped Water: The water pumped out of the sump pit needs to be discharged safely. In compliance with local regulations, the water is directed to an appropriate drainage system or a designated discharge location. This may involve connecting the sump pump to a storm sewer or a surface water drainage system.

- Ongoing Monitoring and Maintenance: Regular monitoring and maintenance of the sump pump system are essential throughout the construction stage. This includes checking the pump's operation, inspecting the sump pit for any debris or blockages, and ensuring the discharge system is functioning properly. Any necessary maintenance tasks, such as cleaning or replacing the pump, are performed to keep the system operating effectively.

By employing a sump pump system during the construction stage, groundwater is effectively managed, preventing water accumulation in the basement and creating a dry working environment. This helps ensure the structural integrity of the basement throughout the construction process.

IV. Structural Design

A. Load calculation and structural analysis (Refer Appendix-A for structural elements design)

Dropping load down to 250mm slab

1. Roof

$$\boxed{DL_{roof} := 1.4 \frac{kN}{m^2}} \qquad \boxed{LL_{roof} := 0.75 \frac{kN}{m^2}}$$

2. Second floor:

$$\boxed{DL_{secondfloor} := 0.72 \frac{kN}{m^2}} \qquad \boxed{LL_{secondfloor} := 1.5 \frac{kN}{m^2}}$$

3. First floor

$$\boxed{DL_{firstfloor} := 0.72 \frac{kN}{m^2}} \qquad \boxed{LL_{firstfloor} := 1.5 \frac{kN}{m^2}}$$

4. DL of walls

$$\boxed{DL_{wallsinternal} := 4.6 \frac{kN}{m^2}} \qquad \boxed{DL_{Externalwall} := 7.10 \frac{kN}{m^2}}$$

Total deadload

$$DL := DL_{roof} + DL_{secondfloor} + DL_{firstfloor} + DL_{wallsinternal} + DL_{Externalwall}$$

$$\boxed{DL = 14.54\ \frac{kN}{m^2}}$$

$$DL_{factored} := 1.35 \cdot DL$$

$$\boxed{DL_{factored} = 19.629\ \frac{kN}{m^2}}$$

Total Live load

$$LL := LL_{roof} + LL_{secondfloor} + LL_{firstfloor}$$

$$\boxed{LL = 3.75\ \frac{kN}{m^2}}$$

$$LL_{factored} := 1.5 \cdot LL$$

$$\boxed{LL_{factored} = 5.625\ \frac{kN}{m^2}}$$

B. Structural elements and Sizes:

The structural elements and the material is tabulated below. For detailed structural design refer to appendix-A.

Structural Element	Material	Sizes
Foundation Raft	Reinforced Concrete	300 mm and 500 mm at edges
Foundation Piles	Reinforced Concrete	350 mm Diameter 500 mm C/C
Foundation Pile Cap	Reinforced Concrete	500 mm Deep and 800mm wide
Swimming Pool Retaining Wall	Reinforced Concrete	300 mm THK.
Columns	Reinforced Concrete	600x250 mm
Ground Floor	Reinforced Concrete Slab	250 mm THK.
Basement Wall	Reinforced Concrete	250 mm THK.

C. Load path

Vertical load transfer	Horizontal load transfer
Roof load transfer: 1.35DL+1.5LL combination of load transfer from roof on to the flat joists and rafters. Flat roof and joists transfer the load on cranked beams and normal beam. Beams will carry the load to the external and internal load bearing walls see figure 10	**Lateral wind load through external walls:** External diaphragm of the building side will undergo through 1kN/m2 of the wind pressure on windward side and suction on the leeward sides. The walls will resist the most of the lateral load and directly ground it on the pile cap see figure 11.
Second floor load: 1.35DL+1.5LL It will be supported by ceiling joist and these ceiling joist will transfer the second floor load to the supporting beam and ultimately to the external and internal load bearing wall see figure 10	**Lateral wind load through internal walls**: The remaining from the above will be taken by joists of slab, transfer to the beam and taken by internal walls and resist by them by sufficient rigidity (EI) it has due to its material see figure 11
First floor load: 1.35DL+1.5LL It will be supported by ceiling joist and these ceiling joist will transfer the First-floor load to the supporting beam and ultimately to the external and internal load bearing wall. External walls supported by pile cap and finally grounded to the hard strata through end bearing piles. Internal load bearing walls will sit on 250mm flat slab see figure 10	**Lift core wall:** External walls transferring the load to the joists and there taken by lift core walls where it directly grounds the lateral load the raft see figure 11.
Ground Floor load: 1.35DL+1.5LL This load will be transferred through slab to column in middle and then to the raft. Whereas external edge will be supported by pile cap and later whole load will be grounded by series of piles to the hard strata. See figure 10	**Soil lateral pressure:** Soil lateral pressure to be taken by piles and ground it to hard strata see figure 11.

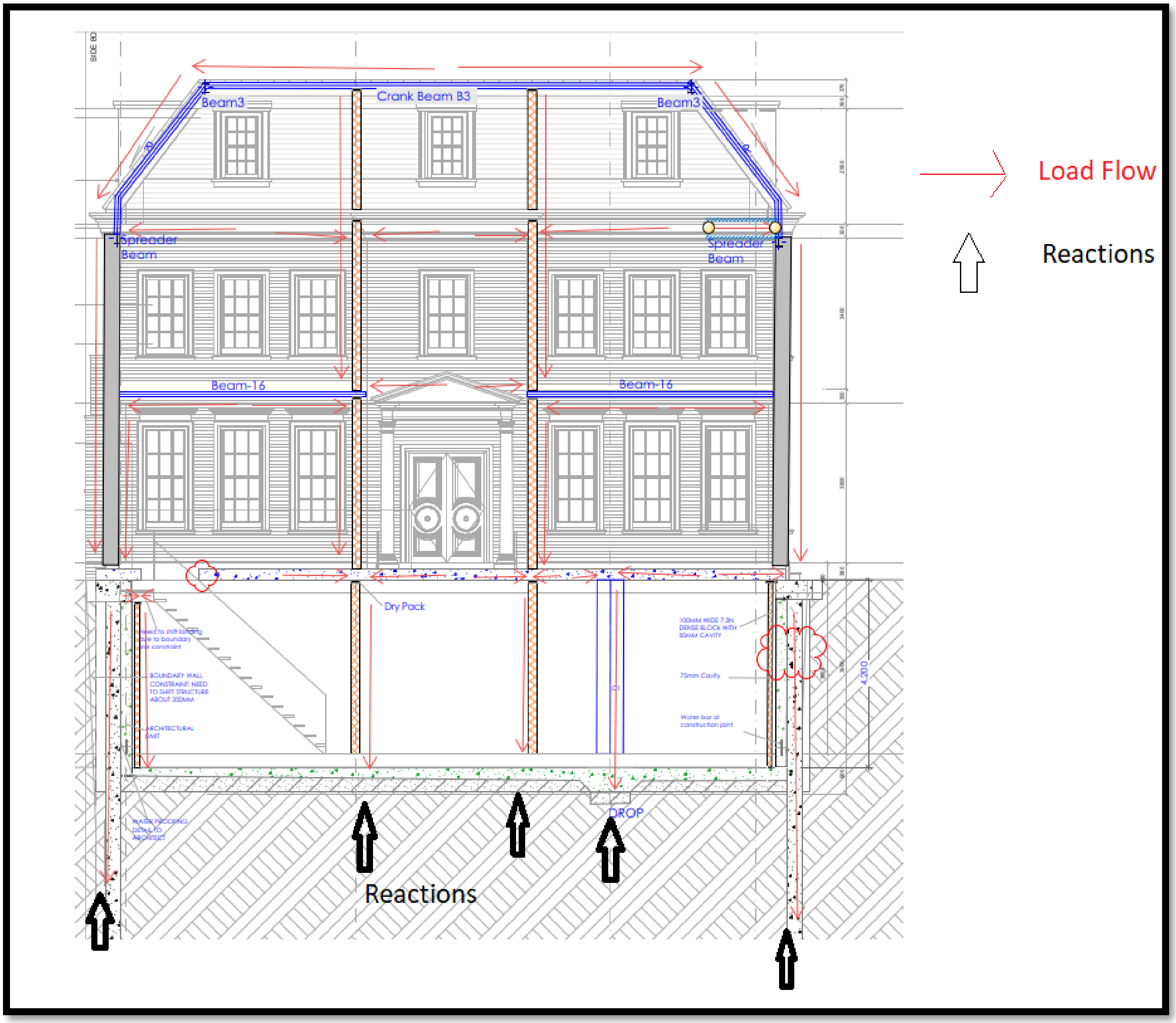

Figure 11 Depicting gravity load flow

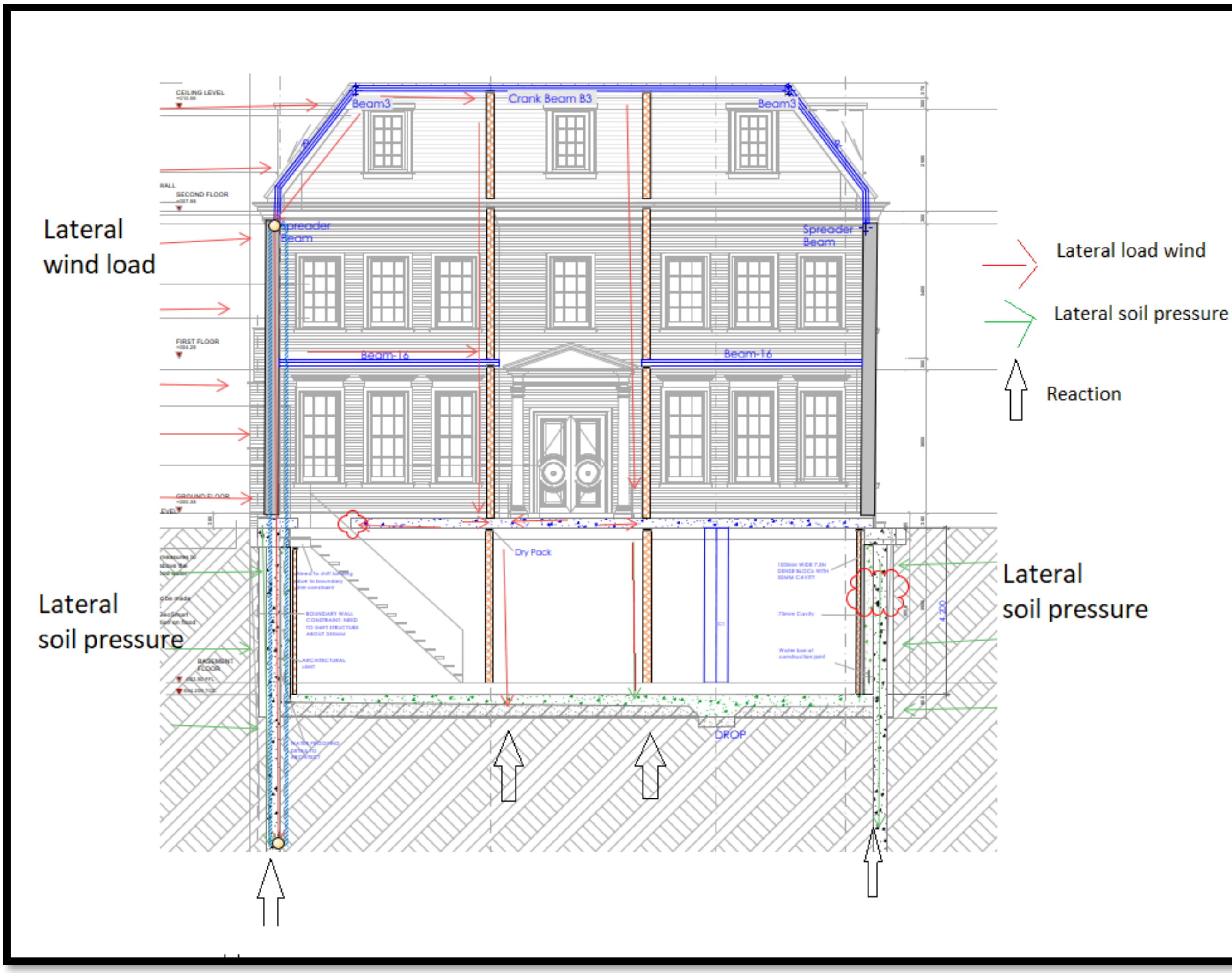

Figure 12 Lateral load flow

Designing a building or structure, there are several elements used and structural forms employed to ensure the safety and stability of the building. The selection of these elements and forms depends on the building's function, location, and intended use. Table 4.1 shows some of the elements and structural forms and the resisting force it offers along with material selection.

Beams: horizontal elements that carry loads across openings or spans between supports

Columns: vertical elements that support the weight of the building

Foundations: elements that transfer the building's weight to the ground

Walls: elements that provide lateral support and divide the building into rooms or compartments

Structural forms

Frame structures: made up of beams and columns, and provide a rigid skeleton for the building

Gravity loads: the weight of the building and its contents

Lateral loads: forces that act horizontally on the building, such as wind, earthquake, or water pressure

Torsion loads: forces that twist or rotate the building, such as from wind or earthquake loads

Shear loads: forces that act perpendicular to the building's length, such as from wind or seismic loads

Axial loads: forces that act along the length of a structural element, such as the weight of a column or beam.

Table 4.1 Elements, structural form and material selection

Elements Used	**Structural form**	**Resisting forces**	**Material selection**
Walls	Plate	Bending, shear, torsion	300 mm brick wall with 100mm insulation(Proposal)
Joists	Beam	Bending and shear	Timber
Beams	Beam	Bending and Shear	Steel
Columns	Strut	Axial	Concrete
slab	Plate	Bending, shear, torsion	Concrete
Pile cap	Plate	Bending, shear, torsion	Concrete
Piles	Strut	Axial	Cast in situ concrete piles
Raft	Plate	Bending	Concrete
Core wall	Core	Shear	Concrete

D. Design of foundation and retaining walls

The foundation for basement will be a raft foundation with meshing at the top and bottom, as well as drops where the column will be positioned. The basement wall will also be reinforced concrete wall as shown figure 10 section A-A . In addition, the swimming pool will feature a retaining wall with a height of 1500 mm. The structure above the basement will be supported by pile caps, which will carry the majority of the super-structural load and ground it to the hard starta by cast in situ concrete piles see figure 11.

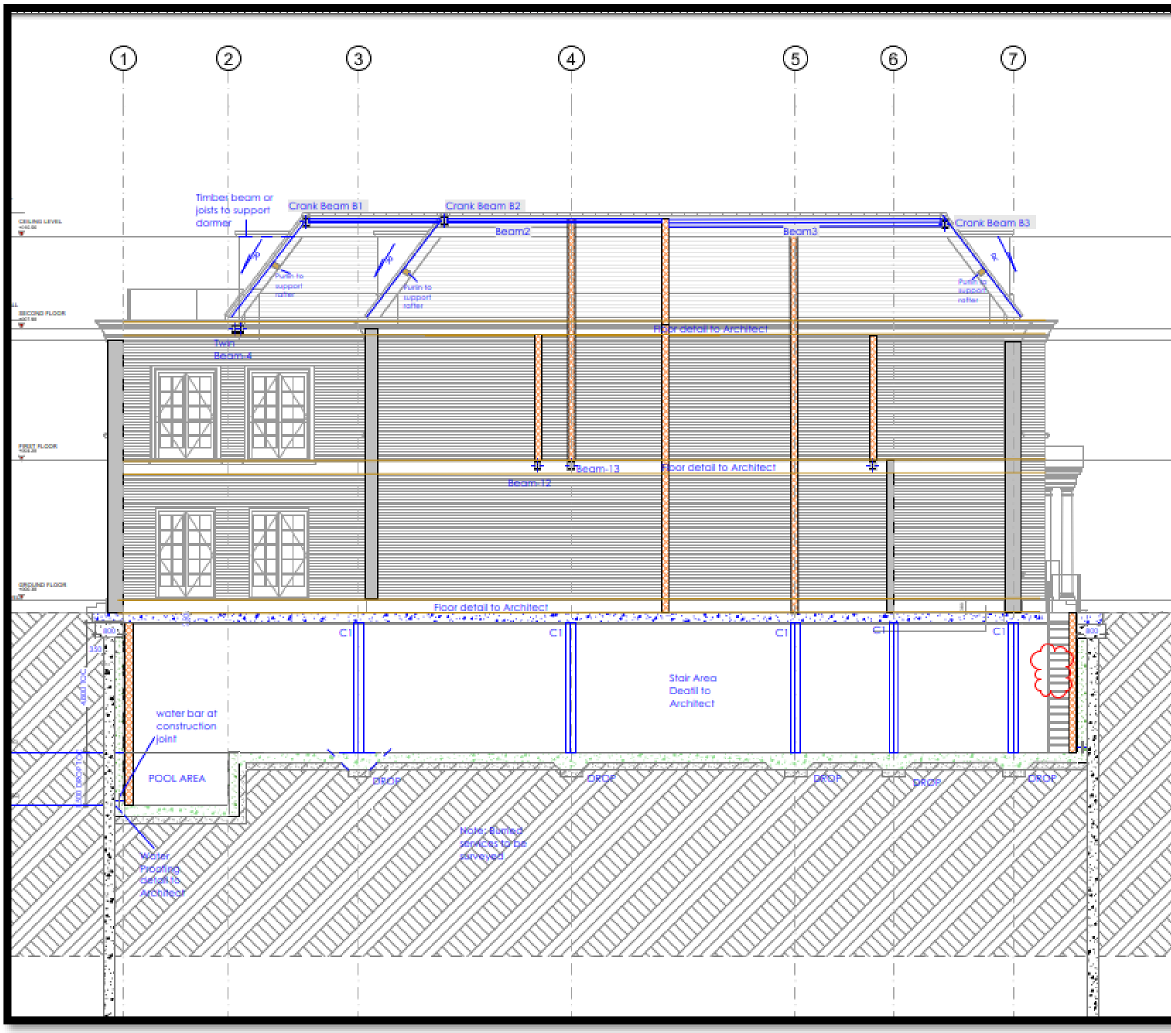

Figure 13 Section A-A(Adkins, 2023b)

E. Design of floor and roof systems

Roof joists will span through steel frameworks that will support the roof on top. The pitch will be supported by rafters, and the load will be passed to the outside walls, which will be composed of masonry. Floors up to the first level are similarly supported by steel beams with joists running across them, transmitting stresses to the exterior or load bearing walls. The bottom floor, on the other hand, is different in that it is supported by a reinforced concrete flat slab that transfers loads to the columns and pile cape.

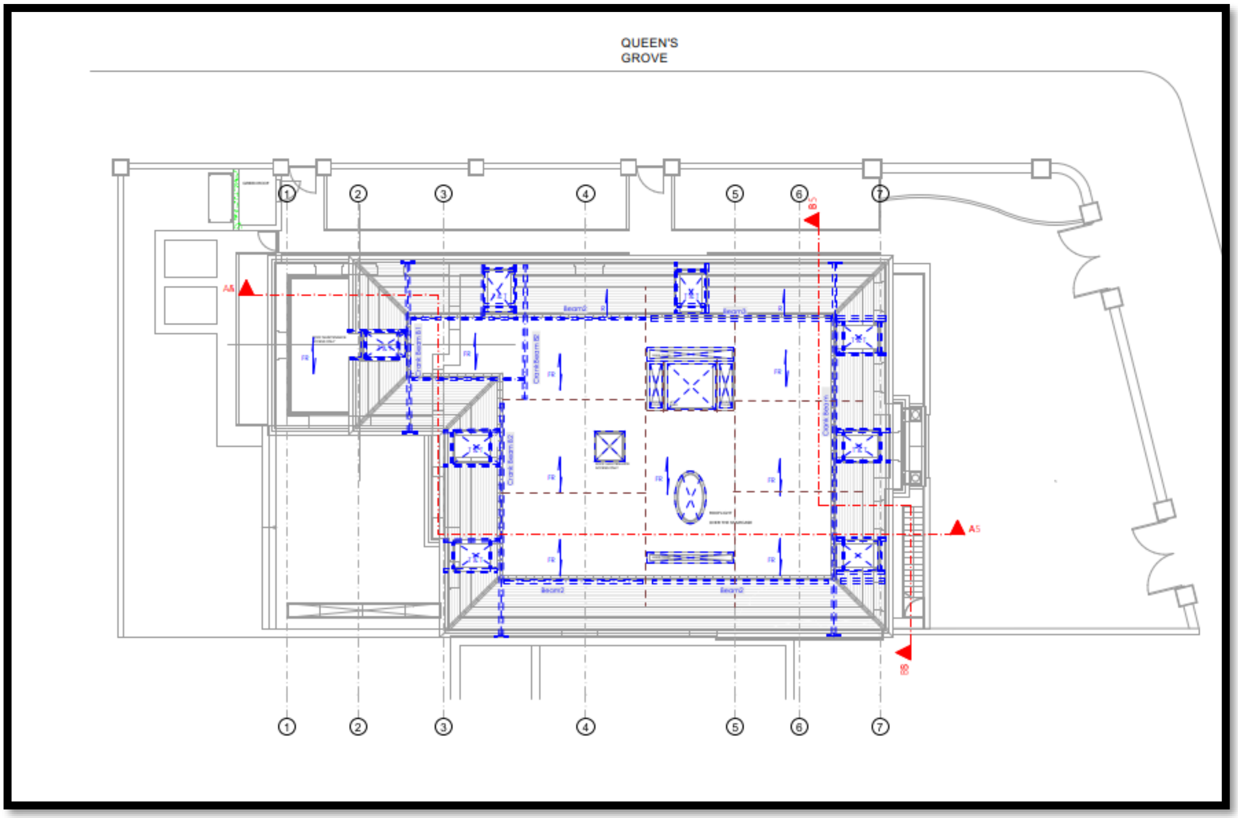

Figure 14-A Roof supporting structural elements

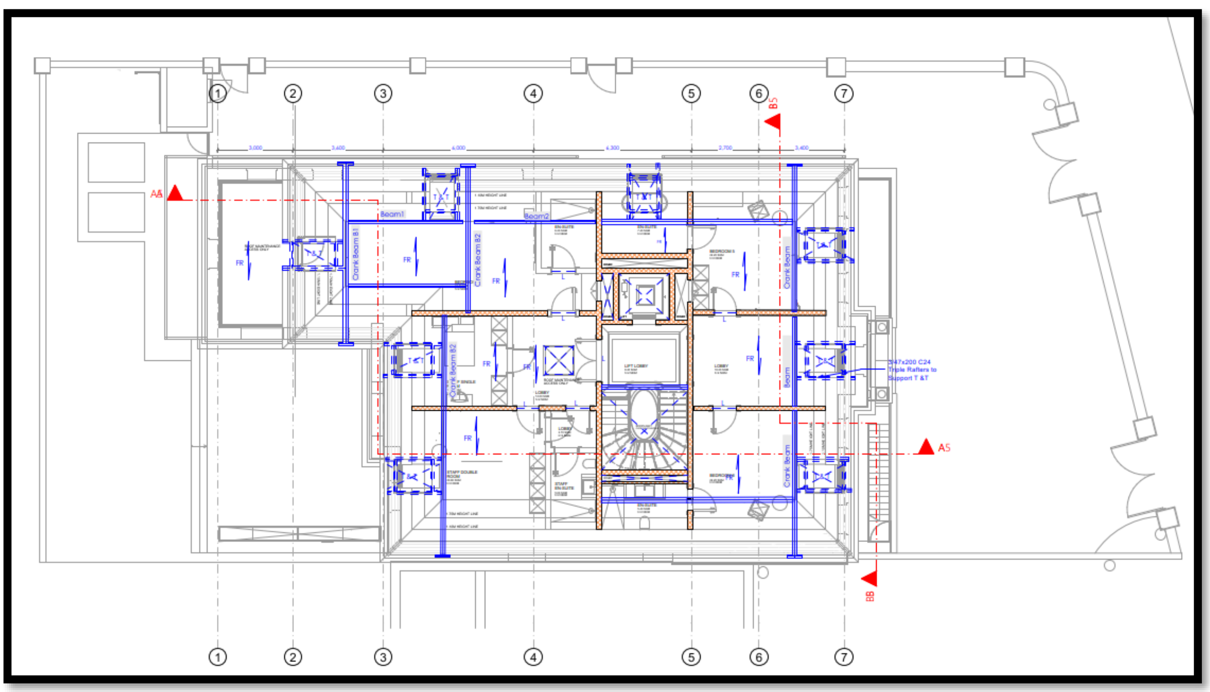

Figure 14-B Second floor

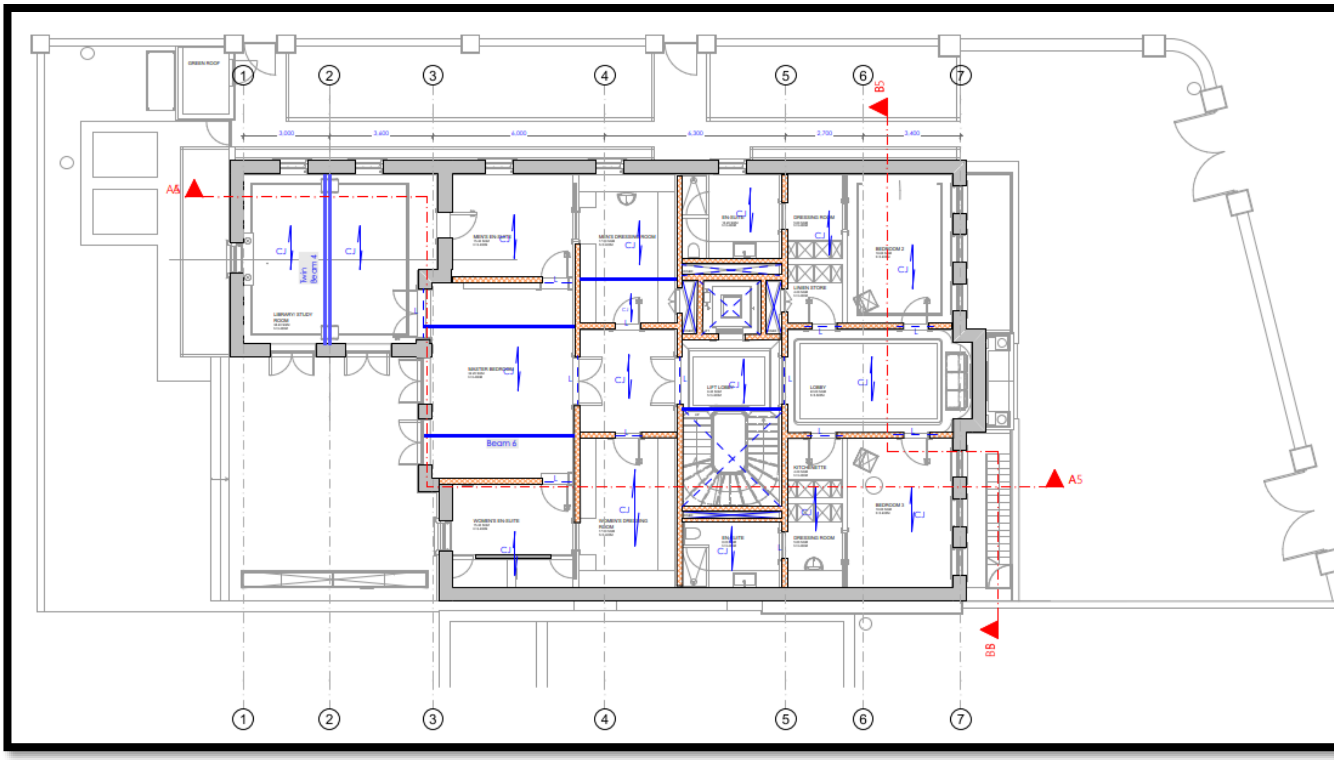

Figure14-C First Floor

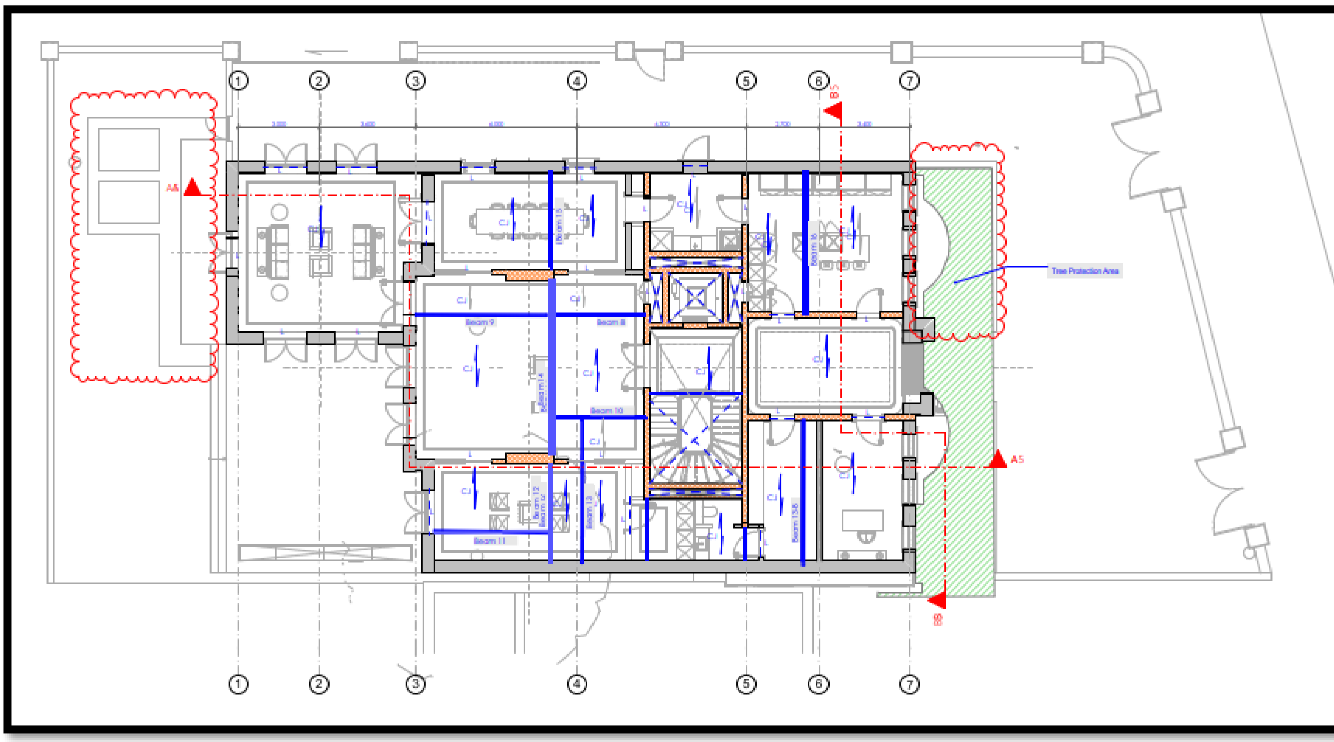

Figure 14-D Ground Floor

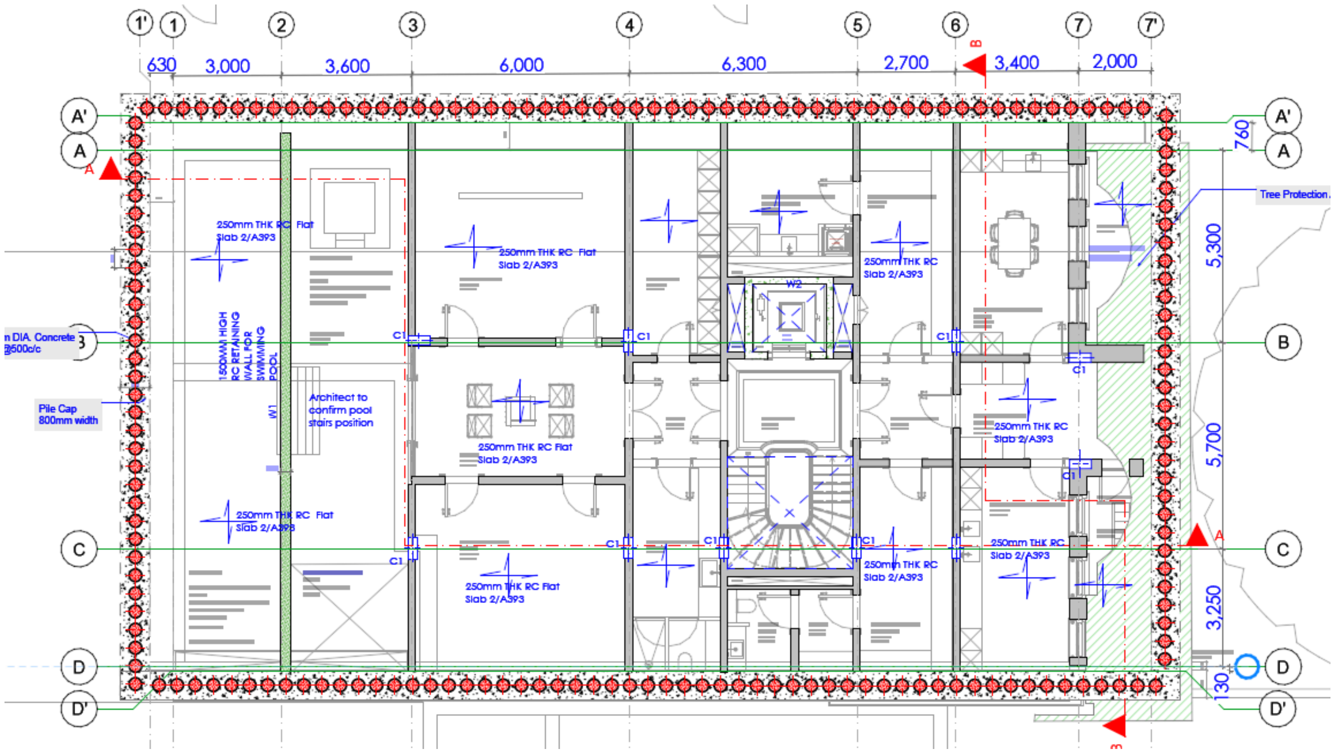

Figure 15-E Basement

F. Design of columns and beams

The beams will be mostly S355 steel, sitting on load-bearing walls with a minimum 100mm bearing and a padstone beneath. However, crank steel beam will sit on 800mm spreader beam on an internal skin of load bearing wall. The columns will bear the load of the ground flat slab and transfer it to the raft. The column will be reinforced concrete with a standard section throughout. as can been in figure 10.

G. Lateral stability

I. Soil Stability – Piles all around the basement must provide lateral stability in terms of soil layers. The longest internal length of contagious piles are 27.630 m and 15.140 m internally refer to grids in figure 15-E. It's feasible that the soil around a basement be stabilized laterally by piles. The suggested piles would be driven into the unstable (London clay) or prone to sliding soil surrounding the basement to build a secure foundation that can withstand lateral stresses. The danger of soil movement around the foundation can be decreased by the piles, which will distribute the weight of the building to deeper, more solid soil layers. The piles will also aid in more uniformly dispersing the structure's weight throughout the earth, which can lessen the possibility of differential settlement or foundation failure.

II. Structural Stability- Reinforced concrete wall that has provided for lift will work as core and will take care of structural stability.

H. Temporary support during the construction

I. Tree protection barrier for protecting tree:

II. Party wall protection: might be sheet piles:

III. Concrete piles cast in situ: Crane might need wooden plank under out riggers in order to provide stability as the soil is clay.

Note: All temporary work needs to be design and reassess by specialist.

VI Basement construction method statement

A. Objective

The objective of this procedure is to provide guidelines for Basement concrete works in the project considering the quality and requirements safety.

B. Scope

The scope of this method statement is the basement construction. It mainly includes area marking, excavation, secant piling, lean concrete & application of waterproofing, screed, rebar works, formwork, lifting and transportation, concrete pouring and curing etc. at 71 Avenue road.

C. Prestart checklist

Before commencing the work, the following shall be carried out.

- ✓ Ensure necessary work permit is obtained for commencement of work on identified location & along with marking.
- ✓ Ensure that work area is clean and safe for work.
- ✓ Make sure that the area is clear of any flammable and combustible materials.
- ✓ PVC/Hard barricade to be used with respect to site conditions along with the proper sign boards inducting the Hazarders substances and work activity and warning lights to be placed for the indication of area during nights.
- ✓ Ensure all site personnel have undergone work safe induction and other relevant training.
- ✓ Ensure that the work is carried out by trained and competent personnel.

D. REFERENCES

- ✓ Avenue road demolition plan (Adkins)
- ✓ Basement Impact assessment Avenue Road-RH (Adkins)
- ✓ Basement Impact assessment Audit (CampbellReith)
- ✓ Demolition Method of Statement (Adkins)

- ✓ Site Inspection Report
- ✓ Flood Risk Assessment 75438R1 RS (GeoSmart)
- ✓ Tree Report
- ✓ PTAL Output
- ✓ Sustainable Drainage Assessment (SuDSmart Plus)
- ✓ Sustainability Statement (Mayfield Morrison Limited)
- ✓ Design and access statement PU Architect

E. ABBREVIATION / DEFINITIONS

- ✓ PPE- Personal Protective Equipment
- ✓ HSE- Health ,Safety & Environment
- ✓ IMTE- Inspection, Measuring & Testing Equipment
- ✓ FQP- Field Quality Plan
- ✓ NGL- Natural Ground Level
- ✓ IFC-Issued For Construction
- ✓ SE-Structural Engineer
- ✓ TOE- Toe of piles
- ✓ LM- Linear meter

F. Tools and Equipment

- ✓ Survey Instruments
- ✓ Concrete pumps
- ✓ Concrete Trucks
- ✓ Concrete Vibrators
- ✓ Slump cone and Tamping rods, Concrete cubes mould, Concrete thermometer
- ✓ Mobile crane
- ✓ Trailer & Trucks
- ✓ Carpenters Tools
- ✓ Steel fixing Tools Steel fabrication equipment/machines
- ✓ Masons Tools
- ✓ Concrete leveling bar/Flat bar
- ✓ Welding machine
- ✓ Power generator

- ✓ Safety barriers/signs light
- ✓ Air compressor
- ✓ Water proofing materials
- ✓ Other tools as work requirements

G. Methodology

The sequence of Basement construction:

Demolition → Site preparation → Survey setting out →Installation of guide wall → Installation of temporary casing → Drilling the pile bore holes →Construction of piles→ Reinforcement cage installation →Concreting →Disposal of drilling spoil →Daily piling report →Bracings to protect contagious pile wall → Excavation → Lean concrete for basement raft → Water proofing → Fabrication and installation of rebar for raft & basement wall → Fabrication and installation of formwork till kicker line → Pouring the concrete → Striping of the form → Curing →formwork for remaining wall → concreting wall →Waterproofing works → slab formwork → fabrication and installation of rebar→ concrete →Backfilling

H. Site preparation:

Survey work to locate and identify the guide line of the area. Make sure the site area is almost in the same level and it is available for the equipment and the vehicle.

Provide proper safe barrier with necessary signs as per the safety standard & requirements

Proper access shall be maintained in the construction area.

I. Survey setting out:

With reference to established benchmarks, lean concrete marking shall be done by using total station and to determine the thickness of the concrete. The location of permanent bored piles shall be set out and pegged by the subcontractor's surveyor based on approved setting out drawings from Adkins consultant and control points at site. The surveying details of each location to be recorded incorporating reduced level and coordinates. Each individually surveyed pile position shall be protected from disturbance prior to commencement of boring works. Two reference points to be installed equidistant at not less than 2.0m from the pile centre location. A pilot hole of about 3-6metre deep shall be drilled at the pile location.

Important levels	Dimensions & Level	Soil type
Existing ground level	+0.00	Made ground
Working platform level	+0.00	Made ground
Maximum excavation level for basement raft	-6.50	London clay formation
Maximum excavation level for piles	10m deep provisional. Exact depth to specialist.	-
TOE level	To specialist	-
Type of shoring	Contagious pile wall 350mm dia @ 500c/c	London clay till -9.00m
Number of piles	176	-
LM of shoring	91.93 m	-

J. Casing of Guide wall by shoring contractor

The guide wall is requires for keeping the alignment and verticality of contagious pile wall within the tolerable limit. Trench excavation shall be carried along the contagious pile wall center line for depth of 500 mm from working platform level. The guide-wall concreting shall be carried out after placing the nominal reinforcement

K. Temporary guide casing

A crawler mounted piling rig specially developed to reduce the possible disturbance to the surrounding soil will be used to install contagious pile. A temporary guide casing of appropriate diameter shall be installed to the pile center through the guide wall shot.

The temporary casing used will be free from significant distortion and shall be uniform cross-section throughout the length. 2 number steel reference pins will be installed equidistance from the pile point. The distance of the steel casing is measured from steel pins and adjustment made so that reference pins are equidistance from the steel casing. The verticality of the steel casing is checked with sprit level in two perpendicular direction.

L. Drilling of pile borehole

After installation of the temporary guide casing the boreholes will be drilled using appropriate drilling tools i.e bucket an auger. In case required the borehole will be drillid using suitable drilling fluid like bentonite until made ground formation.

M. Excavation work:

Excavation shall be performed with mechanical excavator. Slope of excavation shall be at an angle of 34 degrees as per soil investigation report provided by GW. Cross lot bracing to be provided before begin with excavation see figure 15 & 16.

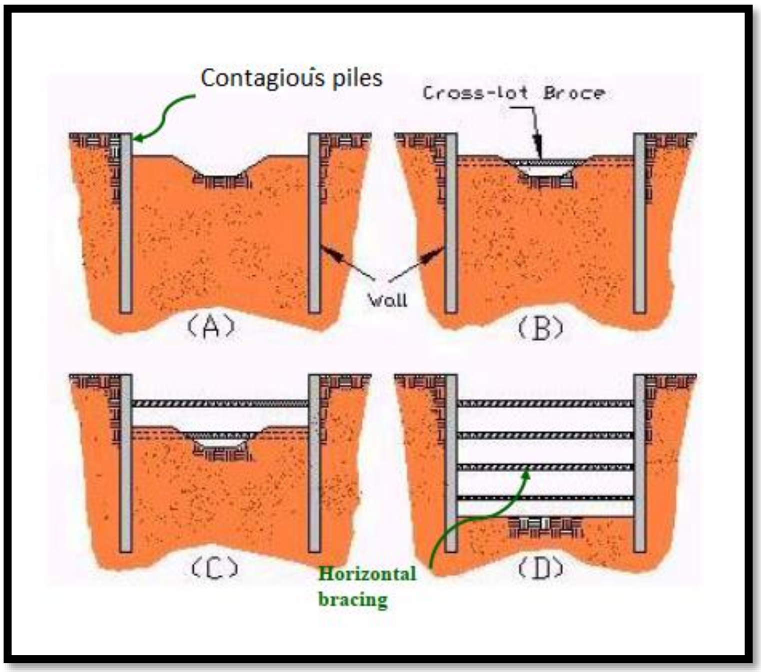

Figure 16 Excavation process after piling wall

The surveyor shall set out the coordinates as per relevant construction drawings and will inform area / excavation Engineer and foreman of these coordinates & levels.

The identification and tree protection will be done as per tree report and earlier stated in section II . Ensure the width of piles and raft as per the approved drawings/IFC AR-A1-ZS-00 to 07.

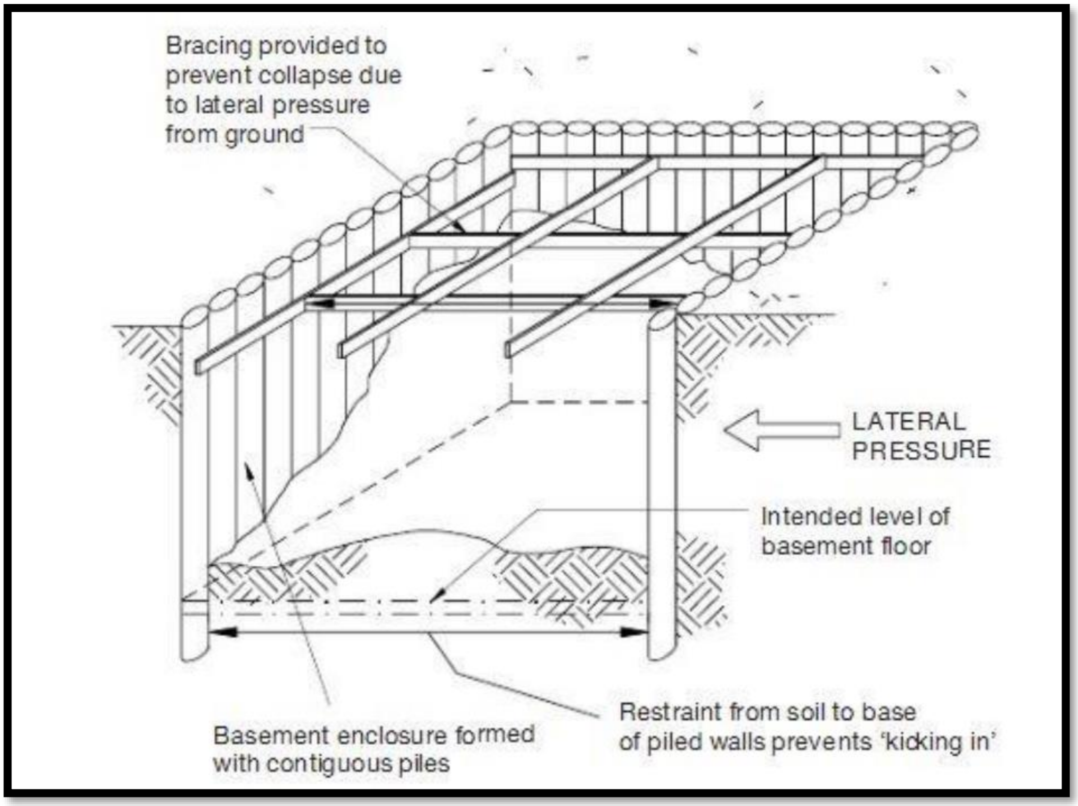

Figure 17 Basement cross-section with structural elements

Area of excavation shall be in general compacted by mini roller where access is feasible. Where no access is available, plate compactor or equivalent machinery shall be used. Suitable hard (guardrails) edge protection two (2M) meters away and around excavations or openings as shown in figure 17.

Figure 18 Edge protection for excavated area by edge safe

Mechanical shovel shall be used for shifting, spreading and levelling of material at dumping area along non-potable water use for dust suppression, compaction.

The excavation slopes can be secured by applying hessian cloth to avoid erosion if Necessary.

Suitable shoring shall be provided – if necessary or the sides shall be benched or sloped back to a safe angle as per standard. Excavate angle of foundations at 34 degrees as well, also as per the soil investigation report requirements.

Safe access/egress to the excavation will be provided as per legal requirements - As shown in figure 18

Figure 19 Safe access to excavated area

Prior to commencing any operations, the client representative will be required to verify all associated technical information such as presence of services, pile coordinates, platform and cut-off levels, validity of drawings etc.

N. Reinforcement work

- ✓ The reinforcing materials to be deformed high yield bars grade Fe 500B according to Eurocode 2 and BS 8110 Yield strength shall be 500Mp. The reinforcing materials Inspected as per IFC and Bar Bending Schedule from the approved drawings, the Bar Bending Schedule is prepared.
- ✓ This enables the weight of standard length and cut to length bars to be taken off. From this take-off, the rebar order can be placed.
- ✓ Cutting and Bending of Bars :The reinforcement will be cut and bent in accordance with the approved bar bending schedules at steel yard.
- ✓ The bending of reinforcements will be done on the bending machine, producing a gradual and even motion. Bars will be bent cold at specified radius.

- ✓ Receipt and Storage of Bars: The rebar received from the supplier will be inspected. All defective materials will be repaired or replaced by the supplier.
- ✓ The storage of rebar will be arranged above ground on padded supports, using timber pieces. The rebar will be covered with dark light proof material to prevent rusting.
- ✓ The mill certificates will be reviewed and listed together with the heat numbers marked on the steel (rebar) and will be correlated against the delivery.

Placing of Reinforcement

- ✓ Minimum concrete cover to reinforcement shall be as per the reinforced concrete
- ✓ works general notes.
- ✓ All splices will be provided as indicated in the approved for construction drawings.
- ✓ Bending or straightening of bars partially embedded in concrete will not be permitted without prior approval of the owner.
- ✓ The reinforcement at obstructions or where it clashes with another reinforcement will not be cut, bent, omitted or modified without prior approval from the Adkins or specialist contractor.
- ✓ The welding of reinforcement will not be permitted.

Fixing of Reinforcement

- ✓ The rebar installation procedure will be supervised by the site engineers of the
- ✓ contractor.
- ✓ Rebar will be tied firmly with approved 1.6mm thick wires at the designated spots.
- ✓ Approved supports, chairs and spacers with various shapes and dimensions will be
- ✓ placed underneath or at sides of rebar to ensure the correct concrete cover required
- ✓ in the contract documents for each structural element.
- ✓ The prefabricated rebar cages shall be tied firmly to keep rebar configuration
- ✓ undisturbed during the hoisting operations.
- ✓ After fixing the reinforcement will be inspected for type, size and accuracy of placing.
- ✓ The areas where the coating will be found to be damaged it will be repaired as per
- ✓ approved method statement.
- ✓ The reinforcement projecting out from work being concreted or already concreted
- ✓ will not be bent out of its correct position. It will be protected from deformations or oher damages.
- ✓ The cover to reinforcement will be maintained by using either plastic spacers or
- ✓ concrete blocks.

For Piles

The reinforcement cage will be fabricated in lay-down sections. The length, type and size of the steel cage will be according to contract drawings and specifications. The cages will be provided with stiffening rings and others accessories to enable handling, lifting and installation without permanent deformations. Cages will be installed into the bored hole using a service crane of the required lifting capacity Concrete spacers wired to the cage shall provide lateral support and ensure adequate concrete cover. Spacers shall be placed at 3 equal levels of each 12m cage with 3 nos at each level.

O. Formwork and supporting system

Cleaning & Treatment

- ✓ Form surfaces shall be clean and dry before the application of approved form release
- ✓ agent.
- ✓ Apply releasing agent over form work surfaces evenly & thinly by using a brush or
- ✓ roller.
- ✓ Formwork shall be erected on level surface free from obstruction and undulations.

Fixing

- ✓ Ensure all reinforcement are tied in position as per drawing before erection of forms.
- ✓ Ensure cover blocks are tied tightly and in position.
- ✓ Forms shall be placed in position as per approved IFC drawing.
- ✓ Chamfers (20mm*20mm) shall be provided on all external exposed corners of
- ✓ concrete members as per approved drawing and specifications.
- ✓ Form work in contact with concrete shall be free from deleterious material,
- ✓ projecting nails, splits & other defects.
- ✓ Ensure all the inserts, cut-outs, pipes and sleeves are provided and concealed
- ✓ service line works are completed.
- ✓ Electrical & Mechanical clearance shall be obtained before closing the shutter.
- ✓ Tie rods, clamps and supports shall be used to keep the shutter in position.
- ✓ The supports must be rigid and firm to maintain level and straightness/verticality of
- ✓ forms.
- ✓ All ties, props, scaffold fittings shall be secured against vibration before & during
- ✓ concreting operations.

- ✓ Joints shall be adequately tightened to prevent grout loss & avoid formation of fines, or other blemishes. Masking taps shall be used to seal the joint completely.
- ✓ Approved PVC Water stop (250MM) shall be placed in the Centre Of the kicker 150mm High as per approved IFC drawing vertically and must be tied firmly on the reinforcement.
- ✓ Alignment and plumb of the form work shall be checked and must ensure prior to concrete.
- ✓ Cover blocks made of same concrete mix of parent material will be used for supporting of reinforcement rebar with correct cover.
- ✓ The formwork system will be inspected by engineer on site and obtain approval prior to casting.

P. Concreting:

- ✓ Concreting will be carried out with the approved design mix/trail mix as specified..
- ✓ The Concrete design and construction shall be in accordance with EN 1992 "DESIGN OF CONCRETE STRUCTURES", unless otherwise noted.
- ✓ It will be ensured that the temperature of the concrete at the time of placing does
- ✓ not exceed 32°C. If the ambient temp of the steel is more than 38°C the forms and
- ✓ reinforcement will be sprayed with water just prior to placing of concrete.
- ✓ Assurance shall be made no sustaining water inside the form work prior to concreting.
- ✓ Slump of concrete shall be as per trail mix approved.
- ✓ Before the truck arrived at the site, all the inspection and testing shall be done by
- ✓ The slump and cube test samples shall be taken from the end of the mixture truck chute or discharge point of the concrete placement hose.
- ✓ During concrete pouring, needle type vibrator shall be used to consolidate the concrete. Sufficient number of vibrators shall be available at site based on the quantity of concrete.
- ✓ The concrete will be placed in layers not exceeding 400 mm in thickness; each layer consolidated separately before the next layer is laid.For piles the concrete of higher slump (=175mm+25mm) shall be used for 'tremie' method. The self-compacting mixed concrete will be discharged through a tremie pipe, which is lowered centrally to the bottom of the bored hole prior to filling it with concrete. Concrete level of the borehole was recorded after each concrete truck discharged and graph will be plotting against theoretical.
- ✓ One length shall be continuously embedded in the concrete during this process to ensure that the discharge of concrete is below the level of the impurities, which might be present in the top part of the rising head of concrete.
- ✓ All testing and sampling of the concrete shall be carried out as instructed by the Engineer or Engineer's representative. A complete record of all cubes taken shall be maintained in a proper form

and slump test results shall be recorded on the 'Delivery Order' and the 'Pile Bore Log'. All compressive concrete tests will be carried out at the supplier's laboratory and independent lab. The client will be notified of the dates of the test by regular issuance in order the tests maybe witnessed.

- ✓ For a continuous assurance of concrete quality and integrity, concrete will be poured to minimum 0.6m above the theoretical cut-off level.
- ✓ All completed piles shall be temporarily barricaded and to be backfilled to ground level with a suitable material the next day.
- ✓ Concrete surface shall be finished by skilled masons with necessary tools & equipment.
- ✓ Control joints, if any, will be cut immediately after the concrete got hardened.
- ✓ After concreting and before setting of concrete, the foundation and other
- ✓ connection points (especially but not limited to anchor bolts) are checked by
- ✓ surveyor to confirm their location, orientation, elevation and condition.
- ✓ Immediately after placement, protect concrete from premature drying or excessively
- ✓ hot or cold temperatures, and mechanical injury as per requirements.
- ✓ One set sample consists six cubes of concrete. Three cubes shall be tested at 7 days
- ✓ and three cubes shall be tested at 28 days.
- ✓ The compressive strength results shall be submitted to the engineer for the approval
- ✓ Before making the cube sample the cube modules will be treated with curing materials.
- ✓ Concrete shall be filled in the cubes in layers and compacted by using compacting rod, the cubes shall be cured at curing tank until testing.
- ✓ The frequency of cubes sampling shall be taken as per the specification or engineer instructions.
- ✓ The finishing of concrete shall be in accordance with construction drawing or job specification.

Q. Stripping off formwork

Stripping shall be done once the concrete reaches its final setting time/ as per the specification. During stripping formwork, all the concrete edges shall be protected from any possible chipping off. The striping for every structure may vary as per the specification as listed below.

- ✓ Slabs (props left under) – 3 days
- ✓ Beams soffits (props left under) – 7 days
- ✓ Removal of props under slab
- ✓ Spanning up to 4.5m – 7 days
- ✓ Spanning over 4.5m – 14 days

- ✓ Removal of props under beams – 14 days
- ✓ Cantilever slab – 14 days

Spoil from piles will be cleared from the boring locations by means of an excavator as boring proceeds. Depending on the volume of spoil excavated, it will be removed to stockpile area or spoil pit, for drying before loading and removed off-site.

R. Curing

In order to control surface cracking the following measures shall be applied

- ✓ Curing and protection of concrete shall be done in accordance with BS EN 206-1.
- ✓ Curing water and compound as per BS EN 206-1.
- ✓ The saturated burlaps shall be covered with a polythene sheets, minimum 0.15 mm
- ✓ (6 mils) in thickness and shall always be kept in contact with the concrete surface.
- ✓ Maximum total dissolved solids in water used for curing shall not exceed 1000 ppm.
- ✓ Water curing shall be continuous until the compressive strength has reached 70% of
- ✓ the specified strength or seven days minimum.
- ✓ As soon as the form work is removed, all surfaces shall be wrapped with wet hessian
- ✓ clothes.
- ✓ The polyethylene sheet shall be placed over the hessian clothes immediately to
- ✓ prevent evaporation from the wet hessian cloth throughout the curing period, the
- ✓ hessian clothe shall be maintained in a permanently wet condition using water.

S. Backfilling

- ✓ Backfilling will commence only after the completion of inspection and test of below grade structures is complete.
- ✓ Backfilling shall commence using similar excavated suitable materials.
- ✓ Care shall be taken to remove stone bolder, plastic paper, Aluminum can, wooden
- ✓ planks, papers and any other deleterious materials.
- ✓ The thickness of the back fill layers shall not be more than 200mm as per project specifications.
- ✓ Each layer will be compacted to a dry density, depending on the area of working.
- ✓ Back filling compaction shall be continue till achieve final level.
- ✓ After reaching the final layer water sprinkling will be done for the compaction. If
- ✓ required, the fill material will be ploughed to uniform water content before compacting.
- ✓ If drying is required, the same will be done by way of natural drying. In case
- ✓ additional dry material is added for drying, care will be taken to see that the layer

- ✓ thickness does not exceed that specified.

VII Conclusion

A. Summary of the basement design

This study covered all temporary and permanent work components that could have an impact on geology, hydrogeology, hydrology, and land stability, and it outlines the proposed three-story home with basement construction. All of the issues relating to structural stability have been addressed in this study. The site team will benefit from the computation for the elements and the detailed technique of statement that have been supplied.

Basement are graded according to BS8102 depending on their specific performance requirement.

The following may be summed up as the primary functional requirements for a basement or basement wall that are successfully addressed in this report:

1.Structural stability

2. Durability

3. Exclusion of moisture

4. Buildability

VI. References

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VII. Appendix

A. Calculations

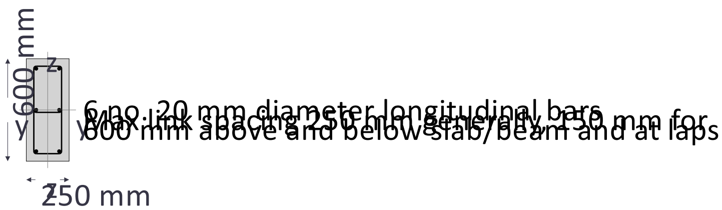
RC COLUMN DESIGN (EN 1992)

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.4.05

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Moment capacity (y)	kNm	301	13	0.04	PASS
Moment capacity (z)	kNm	116	35	0.30	PASS
Biaxial bending				0.31	PASS



Column input details

Column geometry

Overall depth (perpendicular to y axis) h = **600** mm

Overall breadth (perpendicular to z axis) b = **250** mm

Stability in the z direction **Braced**

Stability in the y direction **Braced**

Concrete details

Concrete strength class C32/40

Partial safety factor for concrete (2.4.2.4(1)) γ_C = **1.50**

Coefficient α_{cc} (3.1.6(1)) α_{cc} = **0.85**

Maximum aggregate size d_g = **20** mm

Reinforcement details

Nominal cover to links c_{nom} = **40** mm

Longitudinal bar diameter ϕ = **20** mm

Link diameter ϕ_v = **8** mm

Total number of longitudinal bars N = **6**

No. of bars per face parallel to y axis N_y = **2**

No. of bars per face parallel to z axis N_z = **3**

Area of longitudinal reinforcement	$A_s = N \times \pi \times \phi^2 / 4 =$ **1885** mm^2
Characteristic yield strength	$f_{yk} =$ **500** N/mm^2
Partial safety factor for reinft (2.4.2.4(1))	$\gamma_S =$ **1.15**
Modulus of elasticity of reinft (3.2.7(4))	$E_s =$ **200** kN/mm^2

Fire resistance details

Fire resistance period	R = **60** min
Exposure to fire	**Exposed on more than one side**
Ratio of fire design axial load to design resistance	$\mu_{fi} =$ **0.70**

Axial load and bending moments from frame analysis

Design axial load	$N_{Ed} =$ **660.0** kN
Moment about y axis at top	$M_{topy} =$ **0.0** kNm
Moment about y axis at bottom	$M_{btmy} =$ **0.0** kNm
Moment about z axis at top	$M_{topz} =$ **0.0** kNm
Moment about z axis at bottom	$M_{btmz} =$ **0.0** kNm

Column effective lengths

Effective length for buckling about y axis	$l_{0y} =$ **3650** mm
Effective length for buckling about z axis	$l_{0z} =$ **3650** mm

Calculated column properties

Concrete properties

Area of concrete	$A_c = h \times b =$ **150000** mm^2
Characteristic compression cylinder strength	$f_{ck} =$ **32** N/mm^2
Design compressive strength (3.1.6(1))	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C =$ **18.1** N/mm^2
Mean value of cylinder strength (Table 3.1)	$f_{cm} = f_{ck} + 8$ MPa = **40.0** N/mm^2
Secant modulus of elasticity (Table 3.1)	$E_{cm} = 22000 \text{ MPa} \times (f_{cm} / 10 \text{ MPa})^{0.3} =$ **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3))	$\lambda_{sb} = 0.8$
Stress factor (3.1.7(3))	$\eta = 1.0$

Strain limits

Compression strain limit (Table 3.1)	$\varepsilon_{cu3} =$ **0.00350**
Pure compression strain limit (Table 3.1)	$\varepsilon_{c3} =$ **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2))	$f_{yd} = f_{yk} / \gamma_S =$ **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3))	$c_{min,b} = \max(\phi_v, \phi - \phi_v) =$ **12** mm
Min axis distance for fire (EN1992-1-2 T 5.2a)	$a_{fi} =$ **46** mm
Allowance for deviations from min cover (4.4.1.3)	$\Delta c_{dev} =$ **10** mm
Min allowable nominal cover	$c_{nom\_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev}) =$ **28.0** mm
	PASS - the nominal cover is greater than the minimum required

Effective depths of bars for bending about y axis

Area per bar	$A_{bar} = \pi \times \phi^2 / 4 =$ **314** mm^2
Spacing of bars in faces parallel to z axis (c/c)	$s_z = (h - 2 \times (c_{nom} + \phi_v) - \phi) / (N_z - 1) =$ **242** mm
Layer 1 (in tension face)	$d_{y1} = h - c_{nom} - \phi_v - \phi / 2 =$ **542** mm
Layer 2	$d_{y2} = d_{y1} - s_z =$ **300** mm
Layer 3	$d_{y3} = d_{y2} - s_z =$ **58** mm
2nd moment of area of reinft about y axis	$I_{sy} = 2 \times A_{bar} \times (N_y \times (d_{y1} - h /2)^2) =$ **7359** cm^4
Radius of gyration of reinft about y axis	$i_{sy} = \sqrt{(I_{sy} / A_s)} =$ **198** mm

Effective depth about y axis (5.8.8.3(2)) $d_y = h / 2 + i_{sy} = \mathbf{498}$ mm

Effective depths of bars for bending about z axis

Area of per bar $A_{bar} = \pi \times \phi^2 / 4 = \mathbf{314}$ mm^2

Spacing of bars in faces parallel to y axis (c/c) $s_y = (b - 2 \times (c_{nom} + \phi_v) - \phi) / (N_y - 1) = \mathbf{134}$ mm

Layer 1 (in tension face) $d_{z1} = b - c_{nom} - \phi_v - \phi / 2 = \mathbf{192}$ mm

Layer 2 $d_{z2} = d_{z1} - s_y = \mathbf{58}$ mm

Effective depth about z axis $d_z = d_{z1} = \mathbf{192}$ mm

Column slenderness about y axis

Radius of gyration $i_y = h / \sqrt{(12)} = \mathbf{17.3}$ cm

Slenderness ratio (5.8.3.2(1)) $\lambda_y = l_{0y} / i_y = \mathbf{21.1}$

Column slenderness about z axis

Radius of gyration $i_z = b / \sqrt{(12)} = \mathbf{7.2}$ cm

Slenderness ratio (5.8.3.2(1)) $\lambda_z = l_{0z} / i_z = \mathbf{50.6}$

Design bending moments

Frame analysis moments about y axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (y axis) $e_{iy} = l_{0y} /400 = \mathbf{9.1}$ mm

Min end moment about y axis $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{6.0}$ kNm

Max end moment about y axis $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{6.0}$ kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A $A = \mathbf{0.7}$

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.301}$

Factor B $B = \sqrt{(1 + 2 \times \omega)} = \mathbf{1.266}$

Moment ratio $r_{my} = \mathbf{1.000}$

Factor C $C_y = 1.7 - r_{my} = \mathbf{0.700}$

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) = \mathbf{0.243}$

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} = \mathbf{25.2}$

λ_y<λ_{limy} - Second order effects may be ignored

Frame analysis moments about z axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (z axis) $e_{iz} = l_{0z} /400 = \mathbf{9.1}$ mm

Min end moment about z axis $M_{01z} = \min(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{6.0}$ kNm

Max end moment about z axis $M_{02z} = \max(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{6.0}$ kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A $A = \mathbf{0.7}$

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.301}$

Factor B $B = \sqrt{(1 + 2 \times \omega)} = \mathbf{1.266}$

Moment ratio $r_{mz} = \mathbf{1.000}$

Factor C $C_z = 1.7 - r_{mz} = \mathbf{0.700}$

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) = \mathbf{0.243}$

Slenderness limit $\lambda_{limz} = 20 \times A \times B \times C_z / \sqrt{(n)} = \mathbf{25.2}$

λ_z>=λ_{limz} - Second order effects must be considered

Design bending moments (cl. 6.1(4))

Design moment about y axis $M_{Edy} = \max(M_{02y}, N_{Ed} \times \max(h/30, 20 \text{ mm})) = \mathbf{13.2}$ kNm

Local second order bending moment about z axis (cl. 5.8.8.2 & 5.8.8.3)

Relative humidity of ambient environment $RH = \mathbf{50}$ %

Column perimeter in contact with atmosphere $u = \mathbf{1700}$ mm

Age of concrete at loading	t_0 = **28** day
Parameter n_u	$n_u = 1 + \omega$ = **1.301**
Approx value of n at max moment of resistance	n_{bal} = **0.4**
Axial load correction factor	$K_r = \min(1.0\ ,\ (n_u - n) / (n_u - n_{bal}))$ = **1.000**
Reinforcement design strain	$\varepsilon_{yd} = f_{yd} / E_s$ = **0.00217**
Basic curvature	$curve_{basic\_z} = \varepsilon_{yd} / (0.45 \times d_z)$ = **0.0000252** mm^{-1}
Notional size of column	$h_0 = 2 \times A_c / u$ = **176** mm
Factor α_1 (Annex B.1(1))	$\alpha_1 = (35\ MPa / f_{cm})^{0.7}$ = **0.911**
Factor α_2 (Annex B.1(1))	$\alpha_2 = (35\ MPa / f_{cm})^{0.2}$ = **0.974**
Relative humidity factor (Annex B.1(1))	$\phi_{RH} = [1 + ((1 - RH / 100\%) / (0.1\ mm^{-1/3} \times (h_0)^{1/3})) \times \alpha_1] \times \alpha_2$ = **1.764**
Concrete strength factor (Annex B.1(1))	$\beta_{fcm} = 16.8 \times (1\ MPa)^{1/2} / \sqrt{(f_{cm})}$ = **2.656**
Concrete age factor (Annex B.1(1))	$\beta_{t0} = 1 / (0.1 + (t_0 / 1\ day)^{0.2})$ = **0.488**
Notional creep coefficient (Annex B.1(1))	$\phi_0 = \phi_{RH} \times \beta_{fcm} \times \beta_{t0}$ = **2.289**
Final creep development factor (at t ∞)	$\beta_{c\infty}$ = 1.0
Final creep coefficient (Annex B.1(1))	$\phi_\infty = \phi_0 \times \beta_{c\infty}$ = **2.289**
Ratio of SLS to ULS moments	r_{Mz} = **0.80**
Effective creep ratio (5.8.4(2))	$\phi_{efz} = \phi_\infty \times r_{Mz}$ = **1.831**
Factor β	$\beta_z = 0.35 + f_{ck} / 200\ MPa - \lambda_z / 150$ = **0.173**
Creep factor	$K_{\phi z} = \max(1.0\ ,\ 1 + \beta_z \times \phi_{efz})$ = **1.316**
Modified curvature	$curve_{mod\_z} = K_r \times K_{\phi z} \times curve_{basic\_z}$ = **0.0000331** mm^{-1}
Curvature distribution factor	c = **10**
Deflection	$e_{2z} = curve_{mod\_z} \times l_{0z}^2 / c$ = **44.1** mm
Nominal 2nd order moment	$M_{2z} = N_{Ed} \times e_{2z}$ = **29.1** kNm

Design bending moment about z axis (cl. 5.8.8.2 & 6.1(4))

Equivalent moment from frame analysis	$M_{0ez} = \max(0.6 \times M_{02z} + 0.4 \times M_{01z}, 0.4 \times M_{02z})$ = **6.0** kNm
Design moment	$M_{Edz} = \max(M_{02z}, M_{0ez} + M_{2z}, M_{01z} + 0.5 \times M_{2z}, N_{Ed} \times \max(b/30, 20\ mm))$
	M_{Edz} = **35.1** kNm

<u>Moment capacity about y axis with axial load (660.0 kN)</u>

Moment of resistance of concrete

By iteration:-

Position of neutral axis	y = **225.0** mm
Concrete compression force (3.1.7(3))	$F_{yc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times y\ ,\ h) \times b$ = **816.1** kN
Moment of resistance	$M_{Rdyc} = F_{yc} \times [h / 2 - (\min(\lambda_{sb} \times y\ ,\ h)) / 2]$ = **171.4** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y)$ = **-0.00493**
Stress in layer 1	$\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1})$ = **-434.8** N/mm^2
Force in layer 1	$F_{y1} = N_y \times A_{bar} \times \sigma_{y1}$ = **-273.2** kN
Moment of resistance of layer 1	$M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1})$ = **66.1** kNm
Strain in layer 2	$\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y)$ = **-0.00117**
Stress in layer 2	$\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2})$ = **-233.3** N/mm^2
Force in layer 2	$F_{y2} = 2 \times A_{bar} \times \sigma_{y2}$ = **-146.6** kN
Moment of resistance of layer 2	$M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2})$ = **0.0** kNm
Strain in layer 3	$\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y)$ = **0.00260**
Stress in layer 3	$\sigma_{y3} = \min(f_{yd}, E_s \times \varepsilon_{y3}) - \eta \times f_{cd}$ = **416.6** N/mm^2
Force in layer 3	$F_{y3} = N_y \times A_{bar} \times \sigma_{y3}$ = **261.8** kN
Moment of resistance of layer 3	$M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3})$ = **63.4** kNm

Resultant concrete/steel force F_y = **658.1** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis M_{Rdy} = **300.8** kNm

PASS - The moment capacity about the y axis exceeds the design bending moment

<u>Moment capacity about z axis with axial load (660.0 kN)</u>

Moment of resistance of concrete

By iteration:-

Position of neutral axis z = **95.3** mm

Concrete compression force (3.1.7(3)) $F_{zc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times z , b) \times h$ = **829.2** kN

Moment of resistance $M_{Rdzc} = F_{zc} \times [b / 2 - (\min(\lambda_{sb} \times z , b)) / 2]$ = **72.1** kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z)$ = **-0.00355**

Stress in layer 1 $\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1})$ = **-434.8** N/mm^2

Force in layer 1 $F_{z1} = N_z \times A_{bar} \times \sigma_{z1}$ = **-409.8** kN

Moment of resistance of layer 1 $M_{Rdz1} = F_{z1} \times (b / 2 - d_{z1})$ = **27.5** kNm

Strain in layer 2 $\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z)$ = **0.00137**

Stress in layer 2 $\sigma_{z2} = \min(f_{yd}, E_s \times \varepsilon_{z2}) - \eta \times f_{cd}$ = **255.7** N/mm^2

Force in layer 2 $F_{z2} = N_z \times A_{bar} \times \sigma_{z2}$ = **241.0** kN

Moment of resistance of layer 2 $M_{Rdz2} = F_{z2} \times (b / 2 - d_{z2})$ = **16.1** kNm

Resultant concrete/steel force F_z = **660.4** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis M_{Rdz} = **115.7** kNm

PASS - The moment capacity about the z axis exceeds the design bending moment

<u>Biaxial bending</u>

Determine if a biaxial bending check is required (5.8.9(3))

Ratio of column slenderness ratios $ratio_{\lambda} = \max(\lambda_y, \lambda_z) / \min(\lambda_y, \lambda_z)$ = **2.40**

Eccentricity in direction of y axis $e_y = M_{Edz} / N_{Ed}$ = **53.3** mm

Eccentricity in direction of z axis $e_z = M_{Edy} / N_{Ed}$ = **20.0** mm

Equivalent depth $h_{eq} = i_y \times \sqrt{(12)}$ = **600** mm

Equivalent width $b_{eq} = i_z \times \sqrt{(12)}$ = **250** mm

Relative eccentricity in direction of y axis $e_{rel\_y} = e_y / b_{eq}$ = **0.213**

Relative eccentricity in direction of z axis $e_{rel\_z} = e_z / h_{eq}$ = **0.033**

Ratio of relative eccentricities $ratio_e = \min(e_{rel\_y}, e_{rel\_z}) / \max(e_{rel\_y}, e_{rel\_z})$ = **0.156**

$ratio_{\lambda}$ > 2 - Biaxial bending check is required

Biaxial bending (5.8.9(4))

Design axial resistance of section $N_{Rd} = (A_c \times f_{cd}) + (A_s \times f_{yd})$ = **3539.5** kN

Ratio of applied to resistance axial loads $ratio_N = N_{Ed} / N_{Rd}$ = **0.186**

Exponent a a = **1.07**

Biaxial bending utilisation $UF = (M_{Edy} / M_{Rdy})^a + (M_{Edz} / M_{Rdz})^a$ = **0.314**

PASS - The biaxial bending capacity is adequate

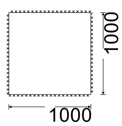
<u>RC GROUND FLOOR SLAB DESIGN</u>

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.21

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Short span					
Reinf. at midspan	mm^2/m	393	322	0.820	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Reinf. at support	mm^2/m	393	322	0.820	PASS
Bar spacing at support	mm	200	300	0.667	PASS
Shear at cont. supp	kN/m	113.7	17.5	0.154	PASS
Deflection ratio		4.88	60.00	0.081	PASS
Long span					
Reinf. at midspan	mm^2/m	393	338	0.860	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Reinf. at support	mm^2/m	393	307	0.780	PASS
Bar spacing at support	mm	200	300	0.667	PASS
Shear at cont. supp	kN/m	109.2	17.5	0.160	PASS
Cover					
Min cover top	mm	40	20	0.500	PASS
Min cover bottom	mm	30	20	0.667	PASS



Slab definition

Slab reference name **Ground floor RC flat slab**

Type of slab **Two way spanning with restrained edges**

Overall slab depth h = **250** mm

Shorter effective span of panel l_x = **1000** mm

Longer effective span of panel l_y = **1000** mm

Support conditions **Four edges continuous (interior panel)**

Top outer layer of reinforcement **Short span direction**

Bottom outer layer of reinforcement **Long span direction**

Loading

Characteristic permanent action G_k = **19.6** kN/m^2

Characteristic variable action Q_k = **5.6** kN/m^2

Partial factor for permanent action γ_G = **1.35**

Partial factor for variable action γ_Q = **1.50**

Quasi-permanent value of variable action ψ_2 = **0.30**

Design ultimate load $q = \gamma_G \times G_k + \gamma_Q \times Q_k$ = **34.9** kN/m^2

Quasi-permanent load $q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k$ = **21.3** kN/m^2

Concrete properties

Concrete strength class C32/40

Characteristic cylinder strength	f_{ck} = **32** N/mm^2
Partial factor (Table 2.1N)	γ_C = **1.50**
Compressive strength factor (cl. 3.1.6)	α_{cc} = **0.85**
Design compressive strength (cl. 3.1.6)	f_{cd} = **18.1** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **3.0** N/mm^2
Maximum aggregate size	d_g = **20** mm

Reinforcement properties

Characteristic yield strength	f_{yk} = **500** N/mm^2
Partial factor (Table 2.1N)	γ_S = **1.15**
Design yield strength (fig. 3.8)	$f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Concrete cover to reinforcement

Nominal cover to outer top reinforcement	c_{nom_t} = **40** mm
Nominal cover to outer bottom reinforcement	c_{nom_b} = **30** mm
Fire resistance period to top of slab	R_{top} = **60** min
Fire resistance period to bottom of slab	R_{btm} = **60** min
Axia distance to top reinft (Table 5.8)	a_{fi_t} = **10** mm
Axia distance to bottom reinft (Table 5.8)	a_{fi_b} = **10** mm
Min. top cover requirement with regard to bond	c_{min,b_t} = **10** mm
Min. btm cover requirement with regard to bond	c_{min,b_b} = **10** mm
Reinforcement fabrication	**Not subject to QA system**
Cover allowance for deviation	Δc_{dev} = **10** mm
Min. required nominal cover to top reinft	$c_{nom\_t\_min}$ = **20.0** mm
Min. required nominal cover to bottom reinft	$c_{nom\_b\_min}$ = **20.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient	β_{sx_p} = **0.0240**
Design bending moment	$M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2$ = **0.8** kNm/m
Reinforcement provided	A393 mesh
Area provided	A_{sx_p} = 393 mm^2/m
Effective depth to tension reinforcement	$d_{x\_p} = h - c_{nom\_b} - \phi_{y\_p} - \phi_{x\_p} / 2$ = **205.0** mm
K factor	$K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck})$ = **0.001**
Redistribution ratio	δ = 1.0
K' factor	$K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21$ = **0.208**

K < K' - Compression reinforcement is not required

Lever arm	$z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **194.7** mm
Area of reinforcement required for bending	$A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z)$ = **10** mm^2/m
Minimum area of reinforcement required	$A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p})$ = **322** mm^2/m
Area of reinforcement required	$A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min})$ = **322** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress	$\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q$ = **6.7** N/mm^2
Maximum allowable spacing (Table 7.3N)	$s_{max\_x\_p}$ = **300** mm
Actual bar spacing	s_{x_p} = **200** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient	β_{sy_p} = **0.0240**

Design bending moment $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2 =$ **0.8** kNm/m
Reinforcement provided A393 mesh
Area provided A_{sy_p} = 393 mm^2/m
Effective depth to tension reinforcement $d_{y\_p} = h - c_{nom\_b} - \phi_{y\_p} / 2 =$ **215.0** mm
K factor $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck}) =$ **0.001**
Redistribution ratio $\delta = 1.0$
K' factor $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **204.2** mm
Area of reinforcement required for bending $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z) =$ **9** mm^2/m
Minimum area of reinforcement required $A_{sy\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p}) =$ **338** mm^2/m
Area of reinforcement required $A_{sy\_p\_req} = \max(A_{sy\_p\_m}, A_{sy\_p\_min}) =$ **338** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times \min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q =$ **6.4** N/mm^2
Maximum allowable spacing (Table 7.3N) $s_{max\_y\_p}$ = **300** mm
Actual bar spacing s_{y_p} = **200** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at continuous support in short span direction (cl.6.1)

Bending moment coefficient β_{sx_n} = **0.0310**
Design bending moment $M_{x\_n} = \beta_{sx\_n} \times q \times l_x^2 =$ **1.1** kNm/m
Reinforcement provided A393 mesh
Area provided A_{sx_n} = 393 mm^2/m
Effective depth to tension reinforcement $d_{x\_n} = h - c_{nom\_t} - \phi_{x\_n} / 2 =$ **205.0** mm
K factor $K = M_{x\_n} / (b \times d_{x\_n}^2 \times f_{ck}) =$ **0.001**
Redistribution ratio $\delta = 1.0$
K' factor $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm $z = \min(0.95 \times d_{x\_n}, d_{x\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **194.7** mm
Area of reinforcement required for bending $A_{sx\_n\_m} = M_{x\_n} / (f_{yd} \times z) =$ **13** mm^2/m
Minimum area of reinforcement required $A_{sx\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_n}, 0.0013 \times b \times d_{x\_n}) =$ **322** mm^2/m
Area of reinforcement required $A_{sx\_n\_req} = \max(A_{sx\_n\_m}, A_{sx\_n\_min}) =$ **322** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sx\_n} = (f_{yk} / \gamma_S) \times \min((A_{sx\_n\_m}/A_{sx\_n}), 1.0) \times q_{SLS} / q =$ **8.6** N/mm^2
Maximum allowable spacing (Table 7.3N) $s_{max\_x\_n}$ = **300** mm
Actual bar spacing s_{x_n} = **200** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at continuous support in long span direction (cl.6.1)

Bending moment coefficient β_{sy_n} = **0.0320**
Design bending moment $M_{y\_n} = \beta_{sy\_n} \times q \times l_x^2 =$ **1.1** kNm/m
Reinforcement provided A393 mesh
Area provided A_{sy_n} = 393 mm^2/m
Effective depth to tension reinforcement $d_{y\_n} = h - c_{nom\_t} - \phi_{x\_n} - \phi_{y\_n} / 2 =$ **195.0** mm
K factor $K = M_{y\_n} / (b \times d_{y\_n}^2 \times f_{ck}) =$ **0.001**
Redistribution ratio $\delta = 1.0$

K' factor — $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

K < K' - Compression reinforcement is not required

Lever arm — $z = \min(0.95 \times d_{y\_n}, d_{y\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **185.2** mm

Area of reinforcement required for bending — $A_{sy\_n\_m} = M_{y\_n} / (f_{yd} \times z) =$ **14** mm^2/m

Minimum area of reinforcement required — $A_{sy\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_n}, 0.0013 \times b \times d_{y\_n}) =$ **307** mm^2/m

Area of reinforcement required — $A_{sy\_n\_req} = \max(A_{sy\_n\_m}, A_{sy\_n\_min}) =$ **307** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress — $\sigma_{sy\_n} = (f_{yk} / \gamma_S) \times \min((A_{sy\_n\_m}/A_{sy\_n}), 1.0) \times q_{SLS} / q =$ **9.4** N/mm^2

Maximum allowable spacing (Table 7.3N) — $s_{max\_y\_n} =$ **300** mm

Actual bar spacing — $s_{y\_n} =$ **200** mm

PASS - The reinforcement spacing is acceptable

Shear capacity check at short span continuous support

Shear force — $V_{x\_n} = q \times l_x / 2 =$ **17.5** kN/m

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200\text{ mm} / d_{x\_n})^{0.5}) =$ **1.988**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sx\_n} / (b \times d_{x\_n})) =$ **0.0019**

Minimum shear resistance (Exp. 6.3N) — $V_{Rd,c\_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{x\_n}$

$V_{Rd,c\_min} =$ **113.7** kN/m

Shear resistance constant (cl. 6.2.2) — $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C =$ **0.12** N/mm^2

Shear resistance (Exp. 6.2a)

$V_{Rd,c\_x\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\text{ N/mm}^2))^{0.333} \times b \times d_{x\_n}) =$ **113.7** kN/m

PASS - Shear capacity is adequate

Shear capacity check at long span continuous support

Shear force — $V_{y\_n} = q \times l_x / 2 =$ **17.5** kN/m

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200\text{ mm} / d_{y\_n})^{0.5}) =$ **2.000**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sy\_n} / (b \times d_{y\_n})) =$ **0.0020**

Minimum shear resistance (Exp. 6.3N) — $V_{Rd,c\_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{y\_n}$

$V_{Rd,c\_min} =$ **109.2** kN/m

Shear resistance constant (cl. 6.2.2) — $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C =$ **0.12** N/mm^2

Shear resistance (Exp. 6.2a)

$V_{Rd,c\_y\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\text{ N/mm}^2))^{0.333} \times b \times d_{y\_n}) =$ **109.2** kN/m

PASS - Shear capacity is adequate

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio — $\rho_0 = (f_{ck} / 1\text{ N/mm}^2)^{0.5} / 1000 =$ **0.0057**

Required tension reinforcement ratio — $\rho = \max(0.0035, A_{sx\_p\_req} / (b \times d_{x\_p})) =$ **0.0035**

Required compression reinforcement ratio — $\rho' = A_{scx\_p\_req} / (b \times d_{x\_p}) =$ **0.0000**

Stuctural system factor (Table 7.4N) — $K_\delta =$ **1.5**

Basic limit span-to-depth ratio (Exp. 7.16)

$ratio_{lim\_x\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}] =$ **50.21**

Mod span-to-depth ratio limit

$ratio_{lim\_x} = \min(40 \times K_\delta, \min(1.5, (500\text{ N/mm}^2 / f_{yk}) \times (A_{sx\_p} / A_{sx\_p\_m})) \times ratio_{lim\_x\_bas}) =$ **60.00**

Actual span-to-eff. depth ratio — $ratio_{act\_x} = l_x / d_{x\_p} =$ **4.88**

PASS - Actual span-to-effective depth ratio is acceptable

<u>Reinforcement summary</u>

Midspan in short span direction — **A393 mesh B2**

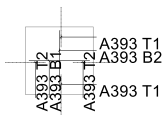
Midspan in long span direction — **A393 mesh B1**

Continuous support in short span direction — **A393 mesh T1**

Continuous support in long span direction **A393 mesh T2**

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.



RC RAFT FOUNDATION

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 1.0.00

Design summary

Overall design status PASS

Overall design utilisation 0.904

Raft

Description	**Unit**	**Provided**	**Required**	**Utilisation**	**Result**
Top reinforcement	mm^2/m	1571	401	0.255	PASS
Bottom reinforcement	mm^2/m	1571	342	0.218	PASS
Shear resistance	kN/m	153.6	36.5	0.238	PASS
Span to depth ratio		60.0	7.5	0.125	PASS

Retaining wall

Description	**Unit**	**Provided**	**Required**	**Utilisation**	**Result**
Bearing pressure (edge)	kN/m^2	180	95	0.527	PASS
Top reinforcement (edge)	mm^2	628	294	0.468	PASS
Bottom reinforcement (edge)	mm^2	628	270	0.430	PASS
Shear resistance (edge)	kN	82.1	60.9	0.742	PASS
Boot reinforcement (edge boot)	mm^2/m	314	200	0.638	PASS
Shear resistance (edge boot)	kN/m	70.3	15.9	0.227	PASS
Bearing pressure (corner boot)	kN/m^2	180	58	0.320	PASS
Top reinforcement (corner)	mm^2	628	294	0.468	PASS
Shear resistance (corner)	kN	82.1	41.6	0.507	PASS
Span to depth ratio (corner)		16.0	2.6	0.162	PASS

Internal beam

Description	Unit	Provided	Required	Utilisation	Result
Top slab reinf. for transfer	mm^2/m	1571	384	0.244	PASS
Bot slab reinf. for transfer	mm^2/m	1571	325	0.207	PASS
Bearing pressure	kN/m^2	180	153	0.848	PASS
Top reinforcement	mm^2	628	441	0.701	PASS
Bottom reinforcement	mm^2	628	406	0.646	PASS
Shear reinforcement	mm^2/m	628	568	0.904	PASS

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

Concrete strength class — C35/45

Aggregate type — Quartzite

Aggregate adjustment factor - cl.3.1.3(2) — AAF = **1.0**

Characteristic compressive cylinder strength — f_{ck} = **35** N/mm^2

Mean value of compressive cylinder strength — $f_{cm} = f_{ck} + 8\ N/mm^2 = \mathbf{43}\ N/mm^2$

Mean value of axial tensile strength — $f_{ctm} = 0.3\ N/mm^2 \times (f_{ck}/ 1\ N/mm^2)^{2/3} = \mathbf{3.2}\ N/mm^2$

Secant modulus of elasticity of concrete — $E_{cm} = 22\ kN/mm^2 \times (f_{cm} / 10\ N/mm^2)^{0.3} \times AAF = \mathbf{34077}\ N/mm^2$

Ultimate strain - Table 3.1 — $\varepsilon_{cu2} = \mathbf{0.0035}$

Shortening strain - Table 3.1 — $\varepsilon_{cu3} = \mathbf{0.0035}$

Effective compression zone height factor — $\lambda = \mathbf{0.80}$

Effective strength factor — $\eta = \mathbf{1.00}$

Coefficient k_1 — $k_1 = \mathbf{0.40}$

Coefficient k_2 — $k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2}) = \mathbf{1.00}$

Coefficient k_3 — $k_3 = \mathbf{0.40}$

Coefficient k_4 — $k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2}) = \mathbf{1.00}$

Partial factor for concrete -Table 2.1N — $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) — $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength - exp.3.15 — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{19.8}\ N/mm^2$

Compressive strength coefficient - cl.3.1.6(1) — $\alpha_{ccw} = \mathbf{1.00}$

Design compressive concrete strength - exp.3.15 — $f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C = \mathbf{23.3}\ N/mm^2$

Maximum aggregate size — h_{agg} = **20** mm

Monolithic simple support moment factor — $\beta_1 = \mathbf{0.25}$

Density of reinforced concrete — $\rho_{conc} = \mathbf{24.5}\ kN/m^3$

Reinforcement details

Characteristic yield strength of reinforcement — $f_{yk} = \mathbf{500}\ N/mm^2$

Partial factor for reinforcing steel - Table 2.1N — $\gamma_S = \mathbf{1.15}$

Design yield strength of reinforcement — $f_{yd} = f_{yk} / \gamma_S = \mathbf{435}\ N/mm^2$

Soil properties

Allowable bearing pressure — $q_{allow} = \mathbf{180.0}\ kN/m^2$

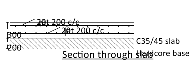
Soil classification — A - Single consistent firm sub-soil

Density of hardcore/compacted fill — $\rho_{fill} = \mathbf{20.0}\ kN/m^3$

Angle of dispersal through fill from horizontal — $\alpha_{fill} = \mathbf{60.0}$ deg

Assumed diameter of depression — ϕ_{dep_basic} = **1500** mm

Slab details

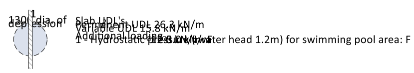


Section through slab

Slab thickness — h_{slab} = **300** mm

Hardcore thickness	h_{fill_slab} = **200** mm
Diameter of depression modified for hardcore	$\phi_{dep\_slab} = \phi_{dep\_basic} - h_{fill\_slab\_b1}$ = **1300** mm
Nominal cover to slab top reinforcement	$c_{nom\_slab\_t}$ = **40** mm
Nominal cover to slab bottom reinforcement	$c_{nom\_slab\_b}$ = **75** mm
Top reinforcement provided	20ϕ at 200 c/c
Area of top reinforcement provided	$A_{s,prov\_t\_slab}$ = **1571** mm^2/m
Average effective depth of top reinforcement	$d_{avg\_t\_slab} = h_{slab} - c_{nom\_slab\_t} - \phi_{t\_slab}$ = **240** mm
Minimum effective depth of top reinforcement	$d_{min\_t\_slab} = h_{slab} - c_{nom\_slab\_t} - 1.5 \times \phi_{t\_slab}$ = **230** mm
Bottom reinforcement provided	20ϕ at 200 c/c
Area of bottom reinforcement provided	$A_{s,prov\_b\_slab}$ = **1571** mm^2/m
Average effective depth of bottom reinforcement	$d_{avg\_b\_slab} = h_{slab} - c_{nom\_slab\_b} - \phi_{b\_slab}$ = **205** mm
Minimum effective depth of bottom reinforcement	$d_{min\_b\_slab} = h_{slab} - c_{nom\_slab\_b} - 1.5 \times \phi_{b\_slab}$ = **195** mm

Raft loading



Slab loading details

No.	Load type	Permanent	Variable	Width x	Width y
1	Hydrostatic pressure (water head 1.2m) for swimming pool area	12.0 kN/m	5.0 kN/m	-	150 mm

Slab UDL loading

Permanent UDL to slab	w_{G_slab} = **26.3** kN/m^2
Variable UDL to slab	w_{Q_slab} = **15.8** kN/m^2

Slab design check

Slab self weight	$w_{slab} = \rho_{conc} \times h_{slab}$ = **7.4** kN/m^2
Slab fill self weight	$w_{fill\_slab} = \rho_{fill} \times h_{fill\_slab}$ = **4.0** kN/m^2
Total uniform load at formation level	$F_{slab} = w_{slab} + w_{fill\_slab} + w_{G\_slab} + w_{Q\_slab}$ = **53.5** kN/m^2

Bearing pressure beneath load 1

Ultimate applied load	$F_{ult\_1\_slab} = 1.35 \times F_{Gk,line\_1\_slab} + 1.50 \times F_{Qk,line\_1\_slab}$ = **23.7** kN/m
Net bearing pressure available to resist load	$q_{net\_1\_slab} = q_{allow} - F_{slab}$ = **126.5** kN/m^2
Net ultimate bearing pressure available	$q_{net\_ult\_1\_slab} = q_{net\_1\_slab} \times F_{ult\_1\_slab} / (F_{Gk,line\_1\_slab} + F_{Qk,line\_1\_slab})$ = **176.4** kN/m^2
Loaded width required at formation	$b_{req\_1\_slab} = F_{ult\_1\_slab} / q_{net\_ult\_1\_slab}$ = **134** mm
Effective loaded width at underside of slab	$b_{req\_eff\_1\_slab} = \max(b_{long\_1\_slab}, b_{req\_1\_slab} - 2 \times h_{fill\_slab} / \tan(\alpha_{fill}))$ = **150** mm
Eff. net ult. bearing pressure available at u/s slab	$q_{net\_ult\_eff\_1\_slab} = q_{net\_ult\_1\_slab} \times b_{req\_1\_slab} / b_{req\_eff\_1\_slab}$ = **158.0** kN/m^2
Cantilever bending moment	$M_{u\_1\_slab} = q_{net\_ult\_1\_slab} \times ((b_{req\_eff\_1\_slab} - b_{long\_1\_slab}) / 2)^2 / 2$ = **0.0** kNm/m

Reinforcement required in bottom of slab to distribute the load

Cantilever bending moment	$M_{u\_max\_slab} = \max(M_{u\_1\_slab})$ = **0.0** kNm/m
	$K_{max\_slab} = M_{u\_max\_slab} / (d_{min\_b\_slab}^2 \times f_{ck})$ = **0.000**

	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**
Lever arm	$z_{max\_slab} = \min(0.5 \times d_{min\_b\_slab} \times [1 + (1 - 2 \times K_{b\_slab} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{min\_b\_slab})$ = **185** mm
Minimum area of reinforcement required	$A_{s,min\_max\_slab} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min\_b\_slab}$ = **325** mm^2/m
Area of tension reinforcement required	$A_{s,req\_max\_slab} = M_{u\_max\_slab} / (f_{yd} \times z_{b\_slab})$ = **0** mm^2/m

PASS - Area of reinforcement provided to distribute the load is adequate. The allowable bearing pressure will not be exceeded

Self weight and UDL forces in slab

Effective span of slab	$l_{eff\_slab} = \phi_{dep\_slab} + d_{avg\_t\_slab}$ = **1540** mm
Ultimate applied self weight and UDL's	$F_{ult\_slab} = 1.35 \times (w_{slab} + w_{G\_slab}) + 1.50 \times w_{Q\_slab}$ = **69.1** kN/m^2
Moment at edge of depression	$M_{neg\_slab} = F_{ult\_slab} \times l_{eff\_slab}^2 / 32$ = **5.1** kNm/m
Moment at centre of depression	$M_{pos\_slab} = F_{ult\_slab} \times l_{eff\_slab}^2 \times (1 + \nu) / 64$ = **3.1** kNm/m
Shear force at edge of depression	$V_{slab} = F_{ult\_slab} \times l_{eff\_slab} / 4$ = **26.6** kN/m

Forces in slab due to load 1

Approximate equivalent UDL	$F_{equiv\_1\_slab} = F_{ult\_1\_slab} / (2 \times 0.3 \times l_{eff\_slab})$ = **25.6** kN/m^2
Moment at edge of depression	$M_{neg\_1\_slab} = F_{equiv\_1\_slab} \times l_{eff\_slab}^2 / 32$ = **1.9** kNm/m
Moment at centre of depression	$M_{pos\_1\_slab} = F_{equiv\_1\_slab} \times l_{eff\_slab}^2 \times (1 + \nu) / 64$ = **1.1** kNm/m
Shear force at edge of depression	$V_{1\_slab} = F_{equiv\_1\_slab} \times l_{eff\_slab} / 4$ = **9.9** kN/m

Total forces in slab

Moment at edge of depression	$M_{neg\_total\_slab} = M_{neg\_slab} + M_{neg\_1\_slab}$ = **7.0** kNm/m
Moment at centre of depression	$M_{pos\_total\_slab} = M_{pos\_slab} + M_{pos\_1\_slab}$ = **4.2** kNm/m
Shear force at edge of depression	$V_{total\_slab} = V_{slab} + V_{1\_slab}$ = **36.5** kN/m

Reinforcement required in top of slab for bending

	$K_{total\_t\_slab} = M_{neg\_total\_slab} / (d_{avg\_t\_slab}^2 \times f_{ck})$ = **0.003**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**
Lever arm	$z_{total\_t\_slab} = \min(0.5 \times d_{avg\_t\_slab} \times [1 + (1 - 2 \times K_{total\_t\_slab} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{avg\_t\_slab})$ = **228** mm
Minimum area of reinforcement required	$A_{s,min\_total\_t\_slab} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{avg\_t\_slab}$ = **401** mm^2/m
Area of tension reinforcement required	$A_{s,req\_total\_t\_slab} = M_{neg\_total\_slab} / (f_{yd} \times z_{total\_t\_slab})$ = **71** mm^2/m

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of slab for bending

	$K_{total\_b\_slab} = M_{pos\_total\_slab} / (d_{avg\_b\_slab}^2 \times f_{ck})$ = **0.003**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**
Lever arm	$z_{total\_b\_slab} = \min(0.5 \times d_{avg\_b\_slab} \times [1 + (1 - 2 \times K_{total\_b\_slab} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{avg\_b\_slab})$ = **195** mm
Minimum area of reinforcement required	$A_{s,min\_total\_b\_slab} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{avg\_b\_slab}$ = **342** mm^2/m
Area of tension reinforcement required	$A_{s,req\_total\_b\_slab} = M_{pos\_total\_slab} / (f_{yd} \times z_{total\_b\_slab})$ = **50** mm^2/m

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor	$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2)$ = **0.516**
Max allowable design shear force (cl.6.2.2)	$V_{Ed,max\_slab} = 0.5 \times d_{min\_t\_slab} \times v_1 \times f_{cwd}$ = **1385** kN/m

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio	$\rho_{l\_slab} = \min(A_{s,prov\_t\_slab} / d_{min\_t\_slab}, 0.02) = \mathbf{0.00683}$
k coefficient	$k_{v\_slab} = \min(1 + (200mm / d_{min\_t\_slab})^{0.5}, 2) = \mathbf{1.933}$
C_{Rd} coefficient	$C_{Rd,c\_slab} = 0.18 / \gamma_C = \mathbf{0.120}$
Minimum shear stress	$v_{min\_slab} = 0.035\ N/mm^2 \times k_{v\_slab}^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5} = \mathbf{0.556}\ N/mm^2$
Minimum design shear resistance	$V_{Rd,c,min\_slab} = v_{min\_slab} \times d_{min\_t\_slab} = \mathbf{127.9}\ kN/m$
Design shear resistance	$V_{Rd,c\_slab} = \max(C_{Rd,c\_slab} \times k_{v\_slab} \times 1N/mm^2 \times (100 \times \rho_{l\_slab} \times f_{ck}/1N/mm^2)^{1/3} \times d_{min\_t\_slab}, V_{Rd,c,min\_slab}) = \mathbf{153.6}\ kN/m$

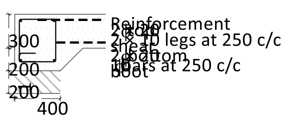
PASS - Design shear resistance exceeds design shear

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_{0\_slab} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000 = \mathbf{0.00592}$
Required tension reinforcement ratio	$\rho_{slab} = A_{s,req\_total\_b\_slab} / d_{min\_b\_slab} = \mathbf{0.00026}$
Required compression reinforcement ratio	$\rho'_{slab} = \mathbf{0}$
Structural system factor - Table 7.4N	$K_{b,defl\_slab} = \mathbf{1.5}$
Basic allowable span to depth ratio	$\delta_{basic\_slab} = K_{b,defl\_slab} \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{0\_slab} / \rho_{slab} + 3.2 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho_{0\_slab} / \rho_{slab} - 1)^{1.5}] = \mathbf{3290.716}$
Reinforcement factor - exp.7.17	$K_{s\_slab} = \min(A_{s,prov\_b\_slab} / A_{s,req\_total\_b\_slab} \times 500\ N/mm^2 / f_{yk}, 1.5) = \mathbf{1.500}$
Flange width factor	$F1 = \mathbf{1.000}$
Long span supporting brittle partition factor	$F2 = \mathbf{1.000}$
Allowable span to depth ratio	$\delta_{allow\_slab} = \min(\delta_{basic\_slab} \times K_{s\_slab} \times F1 \times F2, 40 \times K_{b,defl\_slab}) = \mathbf{60.000}$
Actual span to depth ratio	$\delta_{actual\_slab} = l_{eff\_slab} / d_{avg\_b\_slab} = \mathbf{7.512}$

PASS - Actual span to depth ratio is within the allowable limit

Retaining wall

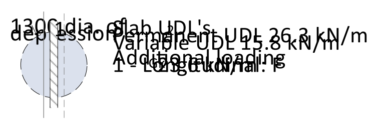


Section through beam

Depth of beam	$h_{edge} = \mathbf{500}\ mm$
Width of beam	$b_{edge} = \mathbf{400}\ mm$
Diameter of depression modified for hardcore	$\phi_{dep\_edge} = \phi_{dep\_basic} - h_{fill\_edge\_b1} = \mathbf{1300}\ mm$
Angle of chamfer	$\alpha_{edge} = \mathbf{45}\ deg$
Depth of boot	$h_{boot\_edge} = \mathbf{200}\ mm$
Width of boot	$b_{boot\_edge} = \mathbf{1}\ mm$
Depth of harcore fill	$h_{fill\_edge} = \mathbf{200}\ mm$
Effective width of beam bearing	$b_{bearing\_edge} = b_{edge} + b_{boot\_edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) = \mathbf{601}\ mm$
Nominal cover to edge beam top reinforcement	$c_{nom\_edge\_t} = \mathbf{40}\ mm$
Nominal cover to edge beam bottom reinforcement	$c_{nom\_edge\_b} = \mathbf{75}\ mm$

Top reinforcement provided	$2 \times 20\phi$
Area of top reinforcement provided	$A_{s,prov\_t\_edge}$ = **628** mm^2
Effective depth of top reinforcement	$d_{t\_edge} = h_{edge} - c_{nom\_edge\_t} - \phi_{v\_edge} - \phi_{t\_edge} / 2$ = **440** mm
Bottom reinforcement provided	$2 \times 20\phi$
Area of bottom reinforcement provided	$A_{s,prov\_b\_edge}$ = **628** mm^2
Effective depth of bottom reinforcement	$d_{b\_edge} = h_{edge} - c_{nom\_edge\_b} - \phi_{v\_edge} - \phi_{b\_edge} / 2$ = **405** mm
Shear reinforcement provided	2×10 legs at 250 c/c
Area of shear reinforcement provided	$A_{sv,prov\_edge}$ = **628** mm^2/m
Boot reinforcement provided	10ϕ bars at 250 c/c
Area of boot reinforcement provided	$A_{s,prov\_boot\_edge}$ = **314** mm^2/m
Effective depth of boot reinforcement	$d_{boot\_edge} = h_{boot\_edge} - c_{nom\_edge\_b} - \phi_{boot\_edge} / 2$ = **120** mm

Retaining wall loading



Edge beam loading details

	Load type	Permanent	Variable	Width x	Width y	Centroid
1	Longitudinal	23.6 kN/m	0.0 kN/m	150 mm	-	200 mm

Edge beam design check

Edge beam self weight: $w_{edge} = \rho_{conc} \times (h_{edge} \times b_{edge} + (h_{edge} - h_{slab\_b1})^2 / (2 \times \tan(\alpha_{edge})) + h_{slab\_b1} \times (h_{edge} - h_{slab\_b1}) / \tan(\alpha_{edge}) + h_{boot\_edge} \times b_{boot\_edge})$ = **6.9** kN/m

Edge beam fill self weight: $w_{fill\_edge} = \rho_{fill} \times h_{fill\_edge}$ = **4.0** kN/m^2

Total uniform load at formation level: $F_{edge} = w_{edge} / b_{bearing\_edge} + w_{fill\_edge} + w_{G\_slab} + w_{Q\_slab}$ = **57.5** kN/m^2

Centroid of longitudinal and equivalent line loads from outside face of raft

Load $\times$ distance of load 1: $M_{centroid\_1\_edge} = w_{ult\_1\_edge} \times x_{1\_edge}$ = **6.4** kN

Total load $\times$ distance: $M_{centroid\_total\_edge} = M_{centroid\_1\_edge}$ = **6.4** kN

Total ultimate long. and equiv. line loads: $w_{equiv\_total\_edge} = w_{ult\_1\_edge}$ = **31.9** kN/m

Centroid of loads: $x_{bar\_edge} = M_{centroid\_total\_edge} / w_{equiv\_total\_edge}$ = **200.0** mm

Bearing pressure

Total unfactored longitudinal and eff. line loads: $w_{equiv\_total\_sls\_edge} = F_{Gk,line\_1\_edge} + F_{Qk,line\_1\_edge}$ = **23.6** kN/m

Allowable bearing width: $b_{brg\_allow\_edge} = 2 \times x_{bar\_edge} + 2 \times h_{fill\_slab\_b1} / \tan(\alpha_{fill})$ = **631** mm

Bearing pressure due to line and point loads: $F_{brg\_edge} = w_{equiv\_total\_sls\_edge} / b_{brg\_allow\_edge}$ = **37.4** kN/m^2

Total applied bearing pressure: $F_{brg\_total\_edge} = F_{brg\_edge} + F_{edge}$ = **94.9** kN/m^2

PASS - Allowable bearing resistance exceeds applied bearing pressure

Edge beam bending check

Divider for moments due to UDL's — β_{udl} = **10.0**

Divider for moments due to point loads — β_{pl} = **6.0**

Applied forces

Span of beam — $l_{edge} = \phi_{dep\_edge} + d_{t\_edge}$ = **1740** mm

Ultimate self weight UDL — $w_{ult\_edge} = 1.35 \times w_{edge}$ = **9.3** kN/m

Approximate ultimate slab self weight UDL — $w_{ult\_slab\_edge}$ = max(0kN/m, $1.35 \times w_{slab} \times (\phi_{dep\_edge} / 2 \times 3/4 - (b_{edge} + (h_{edge} - h_{slab\_b1}) / \tan(\alpha_{edge}))))$ = **0.0** kN/m

Self weight and slab bending moment — $M_{sw\_edge} = (w_{ult\_edge} + w_{ult\_slab\_edge}) \times l_{edge}^2 / \beta_{udl}$ = **2.8** kNm

Self weight and slab shear force — $V_{sw\_edge} = (w_{ult\_edge} + w_{ult\_slab\_edge}) \times l_{edge} / 2$ = **8.1** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) — $w_{ult\_udl\_edge} = (1.35 \times w_{G\_slab} + 1.50 \times w_{Q\_slab}) \times \phi_{dep\_edge} / 2 \times 3 / 4$ = **28.9** kN/m

Bending moment — $M_{udl\_edge} = w_{ult\_udl\_edge} \times l_{edge}^2 / \beta_{udl}$ = **8.7** kNm

Shear force — $V_{udl\_edge} = w_{ult\_udl\_edge} \times l_{edge} / 2$ = **25.1** kN

Moment and shear load 1

Bending moment — $M_{1\_edge} = w_{ult\_1\_edge} \times l_{edge}^2 / \beta_{udl}$ = **9.6** kNm

Shear force — $V_{1\_edge} = w_{ult\_1\_edge} \times l_{edge} / 2$ = **27.7** kN

Resultant moments and shears

Total bending moment (sagging and hogging) — $M_{total\_edge} = M_{udl\_edge} + M_{sw\_edge} + M_{1\_edge}$ = **21.2** kNm

Maximum shear force — $V_{total\_edge} = V_{udl\_edge} + V_{sw\_edge} + V_{1\_edge}$ = **60.9** kN

Reinforcement required in top of beam

Width of section in compression zone — $b_{t\_comp\_edge} = b_{edge} + b_{boot\_edge}$ = **401** mm

$K_{t\_edge} = M_{total\_edge} / (b_{t\_comp\_edge} \times d_{t\_edge}^2 \times f_{ck})$ = **0.008**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

Lever arm — $z_{t\_edge} = \min(0.5 \times d_{t\_edge} \times [1 + (1 - 2 \times K_{t\_edge} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{t\_edge})$ = **418** mm

Minimum area of reinforcement required — $A_{s,min\_t\_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{t\_edge}$ = **294** mm^2

Area of tension reinforcement required — $A_{s,req\_t\_edge} = M_{total\_edge} / (f_{yd} \times z_{t\_edge})$ = **117** mm^2

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of beam

Effective flange outstand — $b_{eff,1\_edge} = 0.1 \times l_{edge}$ = **174** mm

Effective flange width — $b_{eff\_edge} = b_{eff,1\_edge} + b_{edge} + (h_{edge} - h_{slab\_b1}) / \tan(\alpha_{edge})$ = **774** mm

$K_{b\_edge} = M_{total\_edge} / (b_{eff\_edge} \times d_{b\_edge}^2 \times f_{ck})$ = **0.005**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

Lever arm — $z_{b\_edge} = \min(0.5 \times d_{b\_edge} \times [1 + (1 - 2 \times K_{b\_edge} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{b\_edge})$ = **385** mm

Depth of neutral axis — $x = 2 \times (d_{b\_edge} - z_{b\_edge}) / \lambda$ = **51** mm

$\lambda x \le h_{slab}$ - Compression block lies wholly within the depth of slab

Minimum area of reinforcement required — $A_{s,min\_b\_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{b\_edge}$ = **270** mm^2

Area of tension reinforcement required — $A_{s,req\_b\_edge} = M_{total\_edge} / (f_{yd} \times z_{b\_edge})$ = **127** mm^2

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor	$v_1 = 0.6 \times (1 - f_{ck} / 250\ N/mm^2) = \mathbf{0.516}$
Max allowable design shear force (cl.6.2.2)	$V_{Ed,max\_edge} = 0.5 \times d_{t\_int} \times b_{edge} \times v_1 \times f_{cwd} = \mathbf{1059.5}$ kN

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio	$\rho_{l\_edge} = \min(A_{s,prov\_t\_edge} / (b_{edge} \times d_{t\_int}), 0.02) = \mathbf{0.00357}$
k coefficient	$k_{v\_edge} = \min(1 + (200mm / d_{t\_int})^{0.5}, 2) = \mathbf{1.674}$
C_{Rd} coefficient	$C_{Rd,c\_edge} = 0.18 / \gamma_C = \mathbf{0.120}$
Minimum shear stress	$v_{min\_edge} = 0.035\ N/mm^2 \times k_{v\_edge}^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5} = \mathbf{0.449}$ N/mm^2
Minimum design shear resistance	$V_{Rd,c,min\_edge} = v_{min\_edge} \times d_{t\_int} \times b_{edge} = \mathbf{78.9}$ kN
Design shear resistance	$V_{Rd,c\_edge} = \max(C_{Rd,c\_edge} \times k_{v\_edge} \times 1N/mm^2 \times (100 \times \rho_{l\_edge} \times f_{ck}/1N/mm^2)^{1/3} \times d_{t\_int} \times b_{edge}, V_{Rd,c,min\_edge}) = \mathbf{82.1}$ kN

PASS - Design shear resistance exceeds design shear

Boot design check

Effective cantilever span	$l_{eff\_boot\_edge} = b_{boot\_edge} + d_{boot\_edge} / 2 = \mathbf{61}$ mm
Approximate ultimate bearing pressure	$q_{approx\_ult\_edge} = 1.45 \times q_{allow} = \mathbf{261.0}\ kN/m^2$
Cantilever moment	$M_{boot\_edge} = q_{approx\_ult\_edge} \times l_{eff\_boot\_edge}^2 / 2 = \mathbf{0.5}$ kNm/m
Shear force	$V_{boot\_edge} = q_{approx\_ult\_edge} \times l_{eff\_boot\_edge} = \mathbf{15.9}$ kN/m
	$K_{boot\_edge} = M_{boot\_edge} / (d_{boot\_edge}^2 \times f_{ck}) = \mathbf{0.001}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$
Lever arm	$z_{boot\_edge} = \min(0.5 \times d_{boot\_edge} \times [1 + (1 - 2 \times K_{boot\_edge} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{boot\_edge}) = \mathbf{114}$ mm
Minimum area of reinforcement required	$A_{s,min\_boot\_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{boot\_edge} = \mathbf{200}\ mm^2/m$
Area of tension reinforcement required	$A_{s,req\_boot\_edge} = M_{boot\_edge} / (f_{yd} \times z_{boot\_edge}) = \mathbf{10}\ mm^2/m$

PASS - Area of reinforcement provided in boot exceeds area of reinforcement required

Shear resistance with no shear reinforcement

Strength reduction factor	$v_1 = 0.6 \times (1 - f_{ck} / 250\ N/mm^2) = \mathbf{0.516}$
Max allowable design shear force (cl.6.2.2)	$V_{Ed,max\_boot\_edge} = 0.5 \times d_{boot\_edge} \times v_1 \times f_{cwd} = \mathbf{722}$ kN/m

PASS - Design shear force in boot is less than the maximum allowable shear force

Reinforcement ratio	$\rho_{l\_boot\_edge} = \min(A_{s,prov\_boot\_edge} / d_{boot\_edge}, 0.02) = \mathbf{0.00262}$
k coefficient	$k_{v\_boot\_edge} = \min(1 + (200mm / d_{boot\_edge})^{0.5}, 2) = \mathbf{2.000}$
C_{Rd} coefficient	$C_{Rd,c\_boot\_edge} = 0.18 / \gamma_C = \mathbf{0.120}$
Minimum shear stress	$v_{min\_boot\_edge} = 0.035\ N/mm^2 \times k_{v\_boot\_edge}^{3/2} \times (f_{ck} / 1N/mm^2)^{0.5} = \mathbf{0.586}\ N/mm^2$
Minimum design shear resistance	$V_{Rd,c,min\_boot\_edge} = v_{min\_boot\_edge} \times d_{boot\_edge} = \mathbf{70.3}$ kN/m
Design shear resistance	$V_{Rd,c\_boot\_edge} = \max(C_{Rd,c\_boot\_edge} \times k_{v\_boot\_edge} \times 1N/mm^2 \times (100 \times \rho_{l\_boot\_edge} \times f_{ck}/1N/mm^2)^{1/3} \times d_{boot\_edge}, V_{Rd,c,min\_boot\_edge}) = \mathbf{70.3}$ kN/m

PASS - Design shear resistance exceeds design shear

Retaining wall - corner

Total uniform load at formation level	$F_{corn} = w_{edge} / b_{bearing\_edge} + w_{fill\_edge} + w_{G\_slab} + w_{Q\_slab} = \mathbf{57.5}\ kN/m^2$

PASS - Allowable bearing resistance exceeds applied bearing pressure

Applied forces

Span of beam	$l_{corn} = \phi_{dep\_corn} / \sqrt{(2)} + d_{t\_edge} / 2 = \mathbf{1139}$ mm
Ultimate self weight UDL	$w_{ult\_corn} = 1.35 \times w_{edge} = \mathbf{9.3}$ kN/m
Approximate ultimate slab self weight UDL	$w_{ult\_slab\_corn} = \max(0kN/m, 1.35 \times w_{slab} \times (\phi_{dep\_corn} / (\sqrt{(2)} \times 2) - (b_{edge} + (h_{edge} - h_{slab\_b1}) / \tan(\alpha_{edge})))) = \mathbf{0.0}$ kN/m
Self weight and slab bending moment	$M_{sw\_corn} = (w_{ult\_corn} + w_{ult\_slab\_corn}) \times l_{corn}^2 / 2 = \mathbf{6.0}$ kNm

Self weight and slab shear force	$V_{sw\_corn} = (w_{ult\_corn} + w_{ult\_slab\_corn}) \times l_{corn} =$ **10.6** kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{ult\_udl\_corn} = (1.35 \times w_{G\_slab} + 1.50 \times w_{Q\_slab}) \times \phi_{dep\_corn} / \sqrt{(2)} =$ **54.4** kN/m
Bending moment	$M_{udl\_corn} = w_{ult\_udl\_corn} \times l_{corn}^2 / 6 =$ **11.8** kNm
Shear force	$V_{udl\_corn} = w_{ult\_udl\_corn} \times l_{corn} / 2 =$ **31.0** kN

Resultant moments and shears

Bending moment	$M_{total\_x\_corn} = M_{udl\_corn} + M_{sw\_corn} =$ **17.8** kNm
Shear force	$V_{total\_x\_corn} = V_{udl\_corn} + V_{sw\_corn} =$ **41.6** kN
Bending moment	$M_{total\_y\_corn} = M_{udl\_corn} + M_{sw\_corn} =$ **17.8** kNm
Shear force	$V_{total\_y\_corn} = V_{udl\_corn} + V_{sw\_corn} =$ **41.6** kN
Average bending moment	$M_{total\_corn} = (M_{total\_x\_corn} + M_{total\_y\_corn}) / 2 =$ **17.8** kNm
Average shear force	$V_{total\_corn} = (V_{total\_x\_corn} + V_{total\_y\_corn}) / 2 =$ **41.6** kN

Reinforcement required in top of beam

	$K_{t\_corn} = M_{total\_corn} / (b_{t\_comp\_edge} \times d_{t\_edge}^2 \times f_{ck}) =$ **0.007**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) =$ **0.207**
Lever arm	$z_{t\_corn} = \min(0.5 \times d_{t\_edge} \times [1 + (1 - 2 \times K_{t\_corn} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{t\_edge}) =$ **418** mm
Minimum area of reinforcement required	$A_{s,min\_t\_corn} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{t\_edge} =$ **294** mm^2
Area of tension reinforcement required	$A_{s,req\_t\_corn} = M_{total\_corn} / (f_{yd} \times z_{t\_corn}) =$ **98** mm^2

PASS - Area of reinforcement provided exceeds area of reinforcement required

Shear resistance with no shear reinforcement

Strength reduction factor	$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) =$ **0.516**
Max allowable design shear force (cl.6.2.2)	$V_{Ed,max\_corn} = 0.5 \times d_{t\_edge} \times b_{edge} \times v_1 \times f_{cwd} =$ **1059.5** kN

PASS - Design shear force in slab is less than the maximum allowable shear force

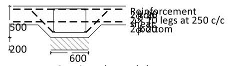
Reinforcement ratio	$\rho_{l\_corn} = \min(A_{s,prov\_t\_edge} / (b_{edge} \times d_{t\_edge}), 0.02) =$ **0.00357**
k coefficient	$k_{v\_corn} = \min(1 + (200\text{mm} / d_{t\_edge})^{0.5}, 2) =$ **1.674**
C_{Rd} coefficient	$C_{Rd,c\_corn} = 0.18 / \gamma_C =$ **0.120**
Minimum shear stress	$v_{min\_corn} = 0.035 \text{ N/mm}^2 \times k_{v\_corn}^{3/2} \times (f_{ck} / 1\text{N/mm}^2)^{0.5} =$ **0.449** N/mm^2
Minimum design shear resistance	$V_{Rd,c,min\_corn} = v_{min\_corn} \times d_{t\_edge} \times b_{edge} =$ **78.9** kN
Design shear resistance	$V_{Rd,c\_corn} = \max(C_{Rd,c\_corn} \times k_{v\_corn} \times 1\text{N/mm}^2 \times (100 \times \rho_{l\_corn} \times f_{ck}/1\text{N/mm}^2)^{1/3} \times d_{t\_edge} \times b_{edge}, V_{Rd,c,min\_corn}) =$ **82.1** kN

PASS - Design shear resistance exceeds design shear

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_{0\_t\_corn} = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 =$ **0.00592**
Required tension reinforcement ratio	$\rho_{t\_corn} = A_{s,req\_t\_corn} / (b_{edge} \times d_{t\_edge}) =$ **0.00056**
Required compression reinforcement ratio	$\rho'_{t\_corn} =$ **0**
Structural system factor - Table 7.4N	$K_{b,defl\_t\_corn} =$ **0.4**
Basic allowable span to depth ratio	$\delta_{basic\_t\_corn} = K_{b,defl\_t\_corn} \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_{0\_t\_corn} / \rho_{t\_corn} + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_{0\_t\_corn} / \rho_{t\_corn} - 1)^{1.5}] =$ **268.769**
Reinforcement factor - exp.7.17	$K_{s\_t\_corn} = \min(A_{s,prov\_t\_edge} / A_{s,req\_t\_corn} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5) =$ **1.500**
Flange width factor	F1 = **1.000**
Long span supporting brittle partition factor	F2 = **1.000**
Allowable span to depth ratio	$\delta_{allow\_t\_corn} = \min(\delta_{basic\_t\_corn} \times K_{s\_t\_corn} \times F1 \times F2, 40 \times K_{b,defl\_t\_corn}) =$ **16.000**
Actual span to depth ratio	$\delta_{actual\_t\_corn} = l_{corn} / d_{t\_edge} =$ **2.589**

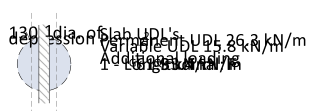
Internal beam 1 details



Section through beam

Depth of beam	h_{int} = **500** mm
Width of beam	b_{int} = **600** mm
Diameter of depression modified for hardcore	$\phi_{dep\_int} = \phi_{dep\_basic} - h_{fill\_int\_b1}$ = **1300** mm
Angle of chamfer	α_{int} = **45** deg
Depth of harcore fill	h_{fill_int} = **200** mm
Nominal cover to internal beam top reinforcement	$c_{nom\_int\_t}$ = **40** mm
Nom. cover to internal beam bottom reinforcement	$c_{nom\_int\_b}$ = **75** mm
Top reinforcement provided	$2 \times 20\phi$
Area of top reinforcement provided	$A_{s,prov\_t\_int}$ = **628** mm^2
Effective depth of top reinforcement	$d_{t\_int} = h_{int} - c_{nom\_int\_t} - \phi_{v\_int} - \phi_{t\_int} / 2$ = **440** mm
Bottom reinforcement provided	$2 \times 20\phi$
Area of bottom reinforcement provided	$A_{s,prov\_b\_int}$ = **628** mm^2
Effective depth of bottom reinforcement	$d_{b\_int} = h_{int} - c_{nom\_int\_b} - \phi_{v\_int} - \phi_{b\_int} / 2$ = **405** mm
Shear reinforcement provided	2×10 legs at 250 c/c
Area of shear reinforcement provided	$A_{sv,prov\_int}$ = **628** mm^2/m

Loading


Internal beam loading details

	Load type	Permanent	Variable	Width x	Width y	Centroid
1	Longitudinal	61.9 kN/m	33.4 kN/m	250 mm	-	0 mm

Internal beam design check

Int beam self weight

$w_{int} = \rho_{conc} \times (h_{int} \times b_{int} + (h_{int} - h_{slab\_b1})^2 / \tan(\alpha_{int}) + 2 \times h_{slab\_b1} \times (h_{int} - h_{slab\_b1}) / \tan(\alpha_{int}))$ = **11.3** kN/m

Int beam fill self weight

$w_{fill\_int} = \rho_{fill} \times h_{fill\_int}$ = **4.0** kN/m^2

Total uniform load at formation level $F_{int} = w_{int} / (b_{int} + 2 \times (h_{int} - h_{slab\_b1}) / \tan(\alpha_{int})) + w_{fill\_int} + w_{G\_slab} + w_{Q\_slab} = \mathbf{57.4}$ kN/m^2

Equivalent and total beam line loads

Total factored long and effective line loads $w_{equiv\_total\_int} = w_{ult\_1\_int} = \mathbf{133.5}$ kN/m

Total unfactored long. and effective line loads $w_{equiv\_total\_sls\_int} = F_{Gk,line\_1\_int} + F_{Qk,line\_1\_int} = \mathbf{95.2}$ kN/m

Centroid of loads from centreline of internal beam

Load × distance of load 1 $M_{centroid\_1\_int} = w_{ult\_1\_int} \times x_{1\_int} = \mathbf{0.0}$ kN

Total load × distance $M_{centroid\_total\_int} = M_{centroid\_1\_int} = \mathbf{0.0}$ kN

Centroid of loads $x_{bar\_int} = M_{centroid\_total\_int} / w_{equiv\_total\_int} = \mathbf{0.0}$ mm

Moment due to eccentricity to be resisted by slab $M_{Ed\_ecc\_int} = w_{equiv\_total\_int} \times \mathrm{abs}(x_{bar\_int}) = \mathbf{0.0}$ kNm/m

Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other

Moment due to depression under slab $M_{Ed\_neg\_slab\_total\_int} = M_{neg\_total\_slab} + M_{Ed\_ecc\_int} / 2 = \mathbf{7.0}$ kNm/m

$K_{slab\_t\_int} = M_{Ed\_neg\_slab\_total\_int} / (d_{min\_t\_slab}^2 \times f_{ck}) = \mathbf{0.004}$

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

Lever arm $z_{slab\_t\_int} = \min(0.5 \times d_{min\_t\_slab} \times [1 + (1 - 2 \times K_{slab\_t\_int} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{min\_t\_slab}) = \mathbf{219}$ mm

Minimum area of reinforcement required $A_{s,min\_slab\_t\_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min\_t\_slab} = \mathbf{384}$ mm^2/m

Area of tension reinforcement required $A_{s,req\_slab\_t\_int} = M_{Ed\_neg\_slab\_total\_int} / (f_{yd} \times z_{slab\_t\_int}) = \mathbf{74}$ mm^2/m

PASS - Area of reinforcement provided in top to span local depression is adequate

Moment to be resisted in bot due to ecc. $M_{Ed\_pos\_slab\_total\_int} = M_{Ed\_ecc\_int} / 2 = \mathbf{0.0}$ kNm/m

$K_{slab\_b\_int} = M_{Ed\_pos\_slab\_total\_int} / (d_{min\_b\_slab}^2 \times f_{ck}) = \mathbf{0.000}$

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

Lever arm $z_{slab\_b\_int} = \min(0.5 \times d_{min\_b\_slab} \times [1 + (1 - 2 \times K_{slab\_b\_int} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{min\_b\_slab}) = \mathbf{185}$ mm

Minimum area of reinforcement required $A_{s,min\_slab\_b\_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min\_b\_slab} = \mathbf{325}$ mm^2/m

Area of tension reinforcement required $A_{s,req\_slab\_b\_int} = M_{Ed\_pos\_slab\_total\_int} / (f_{yd} \times z_{slab\_b\_int}) = \mathbf{0}$ mm^2/m

PASS - Area of reinforcement provided in top to span local depression is adequate

Bearing pressure

Allowable bearing width $b_{brg\_allow\_int} = b_{int} + 2 \times (h_{int} - h_{slab\_b1}) / \tan(\alpha_{int}) = \mathbf{1000}$ mm

Bearing pressure due to line and point loads $F_{brg\_int} = w_{equiv\_total\_sls\_int} / b_{brg\_allow\_int} = \mathbf{95.2}$ kN/m^2

Total applied bearing pressure $F_{brg\_total\_int} = F_{brg\_int} + F_{int} = \mathbf{152.6}$ kN/m^2

PASS - Allowable bearing resistance exceeds applied bearing pressure

Internal beam bending check

Divider for moments due to UDL's $\beta_{udl} = \mathbf{10.0}$

Divider for moments due to point loads $\beta_{pl} = \mathbf{6.0}$

Applied forces

Span of beam $l_{int} = \phi_{dep\_int} + d_{t\_int} = \mathbf{1740}$ mm

Ultimate self weight UDL $w_{ult\_int} = 1.35 \times w_{int} = \mathbf{15.2}$ kN/m

Approximate ultimate slab self weight UDL $w_{ult\_slab\_int} = \max(0\text{kN/m}, 1.35 \times w_{slab} \times (\phi_{dep\_int} \times 3/4 - (b_{int} + 2 \times (h_{int} - h_{slab\_b1}) / \tan(\alpha_{int})))) = \mathbf{0.0}$ kN/m

Self weight and slab bending moment $M_{sw\_int} = (w_{ult\_int} + w_{ult\_slab\_int}) \times l_{int}^2 / \beta_{udl} = \mathbf{4.6}$ kNm

Self weight and slab shear force $V_{sw\_int} = (w_{ult\_int} + w_{ult\_slab\_int}) \times l_{int} / 2 = \mathbf{13.2}$ kN

Moments due to applied uniformly distributed loads

Ultimate udl (approx) — $w_{ult\_udl\_int} = (1.35 \times w_{G\_slab} + 1.50 \times w_{Q\_slab}) \times \phi_{dep\_int} \times 3 / 4 =$ **57.7** kN/m

Bending moment — $M_{udl\_int} = w_{ult\_udl\_int} \times l_{int}^2 / \beta_{udl} =$ **17.5** kNm

Shear force — $V_{udl\_int} = w_{ult\_udl\_int} \times l_{int} / 2 =$ **50.2** kN

Moment and shear load 1

Bending moment — $M_{1\_int} = w_{ult\_1\_int} \times l_{int}^2 / \beta_{udl} =$ **40.4** kNm

Shear force — $V_{1\_int} = w_{ult\_1\_int} \times l_{int} / 2 =$ **116.2** kN

Resultant moments and shears

Total bending moment (sagging and hogging) — $M_{total\_int} = M_{udl\_int} + M_{sw\_int} + M_{1\_int} =$ **62.5** kNm

Maximum shear force — $V_{total\_int} = V_{udl\_int} + V_{sw\_int} + V_{1\_int} =$ **179.6** kN

Reinforcement required in top of beam

$K_{t\_int} = M_{total\_int} / (b_{int} \times d_{t\_int}^2 \times f_{ck}) =$ **0.015**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) =$ **0.207**

Lever arm — $z_{t\_int} = \min(0.5 \times d_{t\_int} \times [1 + (1 - 2 \times K_{t\_int} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{t\_int}) =$ **418** mm

Minimum area of reinforcement required — $A_{s,min\_t\_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{int} \times d_{t\_int} =$ **441** mm^2

Area of tension reinforcement required — $A_{s,req\_t\_int} = M_{total\_int} / (f_{yd} \times z_{t\_int}) =$ **344** mm^2

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of beam

Effective flange outstand — $b_{eff,1\_int} = 0.1 \times l_{int} =$ **174** mm

Effective flange width — $b_{eff\_int} = 2 \times b_{eff,1\_int} + b_{int} + 2 \times (h_{int} - h_{slab\_b1}) / \tan(\alpha_{int}) =$ **1348** mm

$K_{b\_int} = M_{total\_int} / (b_{eff\_int} \times d_{b\_int}^2 \times f_{ck}) =$ **0.008**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) =$ **0.207**

Lever arm — $z_{b\_int} = \min(0.5 \times d_{b\_int} \times [1 + (1 - 2 \times K_{b\_int} / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d_{b\_int}) =$ **385** mm

Depth of neutral axis — $x = 2 \times (d_{b\_int} - z_{b\_int}) / \lambda =$ **51** mm

$\lambda x <= h_{slab}$ - Compression block lies wholly within the depth of slab

Minimum area of reinforcement required — $A_{s,min\_b\_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{int} \times d_{b\_int} =$ **406** mm^2

Area of tension reinforcement required — $A_{s,req\_b\_int} = M_{total\_int} / (f_{yd} \times z_{b\_int}) =$ **374** mm^2

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor — $v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) =$ **0.516**

Max allowable design shear force (cl.6.2.2) — $V_{Ed,max\_int} = 0.5 \times d_{t\_int} \times b_{int} \times v_1 \times f_{cwd} =$ **1589.3** kN

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio — $\rho_{l\_int} = \min(A_{s,prov\_t\_int} / (b_{int} \times d_{t\_int}), 0.02) =$ **0.00238**

k coefficient — $k_{v\_int} = \min(1 + (200\text{mm} / d_{t\_int})^{0.5}, 2) =$ **1.674**

C_{Rd} coefficient — $C_{Rd,c\_int} = 0.18 / \gamma_C =$ **0.120**

Minimum shear stress — $v_{min\_int} = 0.035 \text{ N/mm}^2 \times k_{v\_int}^{3/2} \times (f_{ck} / 1\text{N/mm}^2)^{0.5} =$ **0.449** N/mm^2

Minimum design shear resistance — $V_{Rd,c,min\_int} = v_{min\_int} \times d_{t\_int} \times b_{int} =$ **118.4** kN

Design shear resistance — $V_{Rd,c\_int} = \max(C_{Rd,c\_int} \times k_{v\_int} \times 1\text{N/mm}^2 \times (100 \times \rho_{l\_int} \times f_{ck}/1\text{N/mm}^2)^{1/3} \times d_{t\_int} \times b_{int}, V_{Rd,c,min\_int}) =$ **118.4** kN

Design shear reinforcement required

Angle of comp. shear strut for maximum shear — $\theta_{max} = 45$ deg

Compression chord coefficient - cl.6.2.3(3) — $\alpha_{cw} =$ **1.00**

Minimum area of shear reinforcement - exp.9.5N	$A_{sv,min\_int} = 0.08\ N/mm^2 \times b_{int} \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk} =$ **568** mm^2/m
Min lever arm in shear zone	$z_{min\_int} = \min(z_{t\_int}, z_{b\_int}) =$ **385** mm
Maximum design shear resistance - exp.6.9	$V_{Rd,max\_int} = \alpha_{cw} \times b_{int} \times z_{min\_int} \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) =$ **1389.7** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear stress	$v_{Ed\_int} = V_{total\_int} / (b_{int} \times z_{min\_int}) =$ **0.778** N/mm^2
Angle of concrete compression strut - cl.6.2.3	$\theta_{int} = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed\_int} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ deg), 45deg) =$ **21.8** deg
Area of shear reinforcement required - exp.6.8	$A_{sv,des\_int} = v_{Ed\_int} \times b_{int} / (f_{yd} \times \cot(\theta_{int})) =$ **430** mm^2/m
Area of shear reinforcement required	$A_{sv,req\_int} = \max(A_{sv,min\_int}, A_{sv,des\_int}) =$ **568** mm^2/m
Shear reinforcement provided	2 × 10 legs at 250 c/c
Area of shear reinforcement provided	$A_{sv,prov\_int} =$ **628** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

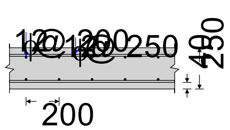
RC WALL DESIGN (EN 1992)

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.1.06

Design summary

Description	Unit	Allowable	Actua	Utilisation	Result
Moment capacity	kNm/m	57.00	7.16	0.13	PASS
Crack width	mm	0.30	0.09	0.29	PASS



Wall input details

Wall geometry

Thickness	h = **250** mm
Length	b = **1000** mm/m
Clear height between restraints	l = **3772** mm
Stability about minor axis	**Braced**

Concrete details

Concrete strength class	**C32/40**
Partial safety factor for concrete (2.4.2.4(1))	$\gamma_C =$ **1.50**
Coefficient α_{cc} (3.1.6(1))	$\alpha_{cc} =$ **0.85**
Maximum aggregate size	$d_g =$ **20** mm

Reinforcement details

Reinforcement in outer layer	**Horizontal**
Nominal cover to outer layer	$c_{nom} =$ **40** mm
Vertical bar diameter	$\phi_v =$ **12** mm

Spacing of vertical reinforcement	s_v = **200** mm
Area of vertical reinforcement (per face)	A_{sv} = **565** mm^2/m
Horizontal bar diameter	ϕ_h = **12** mm
Spacing of horizontal reinforcement	s_h = **250** mm
Area of horizontal reinforcement (per face)	A_{sh} = **452** mm^2/m
Characteristic yield strength	f_{yk} = **500** N/mm^2
Partial safety factor for reinft (2.4.2.4(1))	γ_S = **1.15**
Modulus of elasticity of reinft (3.2.7(4))	E_s = **200.0** kN/mm^2

Fire resistance details

Fire resistance period	R = **60** min
Exposure to fire	**Exposed on two sides**
Ratio of fire design axial load to design resistance	μ_{fi} = **0.70**

Axial load and bending moments from frame analysis

Design axial load	N_{Ed} = **23.6** kN/m
Moment about minor axis at top	M_{top} = **5.0** kNm/m
Moment about minor axis at bottom	M_{btm} = **7.0** kNm/m

Wall effective length factor

Effective length factor for buckling about minor axis	f = **0.70**

Crack width details

Axial load due to quasi-permanent SLS.	N_{Ed_SLS} = **23.6** kN/m
Moment at top due to quasi-permanent SLS.	M_{top_SLS} = **5.0** kNm/m
Moment at btm due to quasi-permanent SLS.	M_{btm_SLS} = **7.0** kNm/m
Duration of applied loading	**Long term**
Maximum allowable crack width	w_{k_max} = **0.3** mm

<u>**Calculated wall properties**</u>

Concrete properties

Area of concrete	$A_c = h \times b$ = **250000** mm^2/m
Characteristic compression cylinder strength	f_{ck} = **32** N/mm^2
Design compressive strength (3.1.6(1))	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2
Mean value of cylinder strength (Table 3.1)	$f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2
Mean value of tensile strength	f_{ctm} = **3.02** N/mm^2
Secant modulus of elasticity (Table 3.1)	E_{cm} = 22000 MPa × (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3))	λ_{sb} = 0.8
Stress factor (3.1.7(3))	η = 1.0

Strain limits

Compression strain limit (Table 3.1)	ε_{cu3} = **0.00350**
Pure compression strain limit (Table 3.1)	ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2))	$f_{yd} = f_{yk} / \gamma_S$ = **434.8** MPa

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (4.4.1.2(3))	$C_{min,b} = \max(\phi_h, \phi_v - \phi_h)$ = **12** mm
Min axis distance for fire (EN1992-1-2 T 5.4)	a_{fi} = **10** mm
Allowance for deviations from min cover (4.4.1.3)	Δc_{dev} = **5** mm
Min allowable nominal cover	$c_{nom\_min} = \max(a_{fi} - \phi_h / 2, c_{min,b} + \Delta c_{dev})$ = **17** mm
	PASS - the nominal cover is greater than the minimum required

Effective depth of vertical bars

Effective depth $d = h - c_{nom} - \phi_h - \phi_v / 2 =$ **192** mm

Depth to compression face bars $d' = c_{nom} + \phi_h + \phi_v / 2 =$ **58** mm

Wall effective length

Wall effective length $l_0 = f \times l =$ **2640** mm

Column slenderness

Radius of gyration about minor axis $i = h / \sqrt{(12)} =$ **7.2** cm

Minor axis slenderness ratio (5.8.3.2(1)) $\lambda = l_0 / i =$ **36.6**

<u>Design bending moments</u>

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections $e_i = l_0 /400 =$ **6.6** mm

Minimum end moment about minor axis $M_{01} = \min(\text{abs}(M_{top}), \text{abs}(M_{btm})) + e_i \times N_{Ed} =$ **5.2** kNm/m

Maximum end moment about minor axis $M_{02} = \max(\text{abs}(M_{top}), \text{abs}(M_{btm})) + e_i \times N_{Ed} =$ **7.2** kNm/m

Slenderness limit for buckling about minor axis (cl. 5.8.3.1)

Factor A $A =$ **0.7**

Mechanical reinforcement ratio $\omega = 2 \times A_{sv} \times f_{yd} / (A_c \times f_{cd}) =$ **0.108**

Factor B $B = \sqrt{(1 + 2 \times \omega)} =$ **1.103**

Moment ratio $r_m = M_{01} / M_{02} =$ **0.720**

Factor C $C = 1.7 - r_m =$ **0.980**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) =$ **0.005**

Slenderness limit $\lambda_{lim} = 20 \times A \times B \times C / \sqrt{(n)} =$ **209.8**

***$\lambda<\lambda_{lim}$* - Second order effects may be ignored**

Design bending moment

Design moment about minor axis $M_{Ed} = \max(M_{02}, N_{Ed} \times \max(h/30, 20 \text{ mm})) =$ **7.2** kNm/m

<u>Moment capacity about minor axis with axial load N_{Ed}</u>

Moment of resistance of concrete

By iteration:-

Position of neutral axis $z =$ **35.5** mm

Concrete compression force (3.1.7(3)) $F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0 \text{ mm}), h) \times b =$ **514.3** kN/m

Moment of resistance $M_{Rdc} = F_c \times [h / 2 - (\min(\lambda_{sb} \times z, h)) / 2] =$ **57.0** kNm/m

Moment of resistance of reinforcement

Strain in tension face bars $\varepsilon = \varepsilon_{cu3} \times (1 - d / z) =$ **-0.01545**

Stress in tension face bars $\sigma = \text{if}(\varepsilon< 0, \max(-1 \times f_{yd}, E_s \times \varepsilon), \min(f_{yd}, E_s \times \varepsilon)) =$ **-434.8** N/mm^2

Force in tension face bars $F_s = \text{if}(d > \lambda_{sb} \times z, A_{sv} \times \sigma, A_{sv} \times (\sigma - \eta \times f_{cd})) =$ **-245.9** kN/m

Strain in compression face bars $\varepsilon' = \varepsilon_{cu3} \times (1 - d' / z) =$ **-0.00223**

Stress in compression face bars $\sigma' = \text{if}(\varepsilon'< 0, \max(-1 \times f_{yd}, E_s \times \varepsilon'), \min(f_{yd}, E_s \times \varepsilon')) =$ **-434.8** N/mm^2

Force in compression face bars $F_s' = \text{if}(d' > \lambda_{sb} \times z, A_{sv} \times \sigma', A_{sv} \times (\sigma' - \eta \times f_{cd})) =$ **-245.9** kN/m

Resultant force on cross-section $F_{res} = N_{Ed} - (F_c + F_s + F_s') =$ **0.954** kN/m

PASS - This is less than 1 kN therefore say OK

Moment of resistance of tension face bars $M_{Rds} = F_s \times (d - h / 2) =$ **-16.5** kNm/m

Moment of resistance of compression face bars $M_{Rds}' = F_s' \times (h / 2 - d') =$ **-16.5** kNm/m

Combined moment of resistance

Moment of resistance about minor axis $M_{Rd} = M_{Rdc} + M_{Rds}' - M_{Rds} =$ **57.0** kNm/m

PASS - The moment capacity exceeds the design bending moment

Crack widths

Slenderness limit (cl. 5.8.3.1)

Min 1st order moment about minor axis	M_{01_SLS}=min(abs(M_{top_SLS}),abs(M_{btm_SLS}))+e_i×N_{Ed_SLS} = **5.2** kNm/m
Max 1st order moment about minor axis	M_{02_SLS}=max(abs(M_{top_SLS}),abs(M_{btm_SLS}))+e_i×N_{Ed_SLS} = **7.2** kNm/m
Moment ratio	r_{m_SLS} = M_{01_SLS} / M_{02_SLS} = **0.720**
Factor C	C_{SLS} = 1.7 - r_{m_SLS} = **0.980**
Relative normal force	n_{SLS} = N_{Ed_SLS} / (A_c × f_{cd}) = **0.005**
Slenderness limit	λ_{lim_SLS} = 20 × A × B × C_{SLS} / √(n_{SLS}) = **209.8**

***λ<λ_{lim_SLS}* - Second order effects may be ignored**

Design bending moment (cl. 7.3.4)

Design moment about minor axis	M_{Ed_SLS} = M_{02_SLS} = **7.2** kNm/m
Cover to tension reinforcement	c = h - d - ϕ_v / 2 = **52.0** mm
Ratio of steel to concrete modulii	α_e = E_s / E_{cm} = **6.0**
Area of reinft in concrete units	$A_{s,eff}$ = 2 × α_e × A_{sv} = **6783** mm²/m
Combined area of steel/conc in conc units	A_{eff} = b × h + $A_{s,eff}$ = **256783** mm²/m
Reinforcement ratio per face	ρ = A_{sv} /(b × d) = **0.003**
Neutral axis depth with pure bending	x_b = d × [-2×α_e×ρ + √(4×α_e^2×ρ^2 + 2×α_e×ρ×(1+d'/d))] = **35.0** mm
Second moment of area of cracked section	I_c = b×x_b^3/3 + α_e×ρ×b×d×[(x_b-d')² + (d-x_b)²] = **99686665** mm⁴/m
Strain in tension face steel due to bending	ε_{sb} = M_{Ed_SLS} × (x_b - d) / (E_{cm} × I_c) = **-0.00034**
Strain in comp face steel due to bending	ε_{sb}' = M_{Ed_SLS} × (x_b - d') / (E_{cm} × I_c) = **-0.00005**
Strain due to axial load	ε_{axial} = N_{Ed_SLS} / (A_{eff} × E_{cm}) = **0.00000**
Resultant strain in tension face steel	ε_s = ε_{sb} + ε_{axial} = **-0.00034**
Resultant strain in comp face steel	ε_s' = ε_{sb}' + ε_{axial} = **-0.00005**
Stress in tension steel	σ_s = min(f_{yd}, abs(E_s × ε_s)) = **67.1** MPa
Depth to neutral axis	x = [(ε_s' × d) - (ε_s × d')] / (ε_s' - ε_s) = **36.2** mm
Effective depth of concrete in tension	$h_{c,ef}$ = min(2.5×(h-d), (h-x)/3, h/2) = **71.3** mm
Effective area of concrete in tension	$A_{c,eff}$ = $h_{c,ef}$ × b = **71257** mm²/m
Load duration factor	k_t = **0.4**
Reinforcement ratio	$\rho_{p,eff}$ = A_{sv} / $A_{c,eff}$ = **0.008**
Mean value of conc tensile strength	$f_{ct,eff}$ = f_{ctm} = **3.02** MPa
Difference between reinft and concrete strains	ε_{diff} = max([σ_s-k_t×$f_{ct,eff}$×(1+α_e×$\rho_{p,eff}$)/$\rho_{p,eff}$]/E_s, 0.6×σ_s/E_s) = **0.00020**
Greater tensile strain	ε_1 = ε_s × (h - x) / (d - x) = **-0.00046**
Lesser tensile strain	ε_2 = min(0, ε_s' × x / (x - d')) = **0.00000**
Factor k_1	k_{1cs} = **0.8**
Factor k_2	k_{2cs} = (ε_1 + ε_2) / (2 × ε_1) = **0.500**
Factor k_3	k_{3cs} = 3.40= **3.4**
Factor k_4	k_{4cs} = **0.425**
Maximum crack spacing	$s_{r,max}$ = k_{3cs} × c + k_{1cs} × k_{2cs} × k_{4cs} × ϕ_v / $\rho_{p,eff}$ = **433.9** mm
Crack width	w_k = $s_{r,max}$ × ε_{diff} = **0.087** mm
Allowable crack width	w_{k_max} = **0.3** mm

PASS - The maximum crack width is less than the maximum allowable

RETAINING WALL ANALYSIS FOR SWIMMING POOL

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

Tedds calculation version 2.9.16

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **1800** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1676** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **0** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **1800** mm
Height of water	h_{water} = **0** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

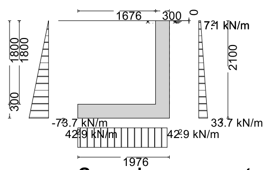
Soil type	Organic clay
Moist density	γ_{mr} = **15** kN/m^3
Saturated density	γ_{sr} = **15** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic cohesion	$c'_{b.k}$ = **0** kN/m^2
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

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m^2



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem}$ = **1976** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover}$ = **1800** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water}$ = **0** mm
Length of surcharge load	$l_{sur} = l_{heel}$ = **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **1976** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **2100** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **1050** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **0.54** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1826** mm
Area of wall base	$A_{base} = l_{base} \times t_{base}$ = **0.593** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2$ = **988** mm
Area of base soil	$A_{pass} = d_{cover} \times l_{toe}$ = **3.017** m^2
- Distance to vertical component	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass}$ = **838** mm
- Distance to horizontal component	$x_{pass\_h} = (d_{cover} + h_{base}) / 3$ = **700** mm
Area of excavated base soil	$A_{exc} = h_{pass} \times l_{toe}$ = **3.017** m^2
- Distance to vertical component	$x_{exc\_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc}$ = **838** mm
- Distance to horizontal component	$x_{exc\_h} = (h_{pass} + h_{base}) / 3$ = **700** mm

Design approach 1

Partial factors on actions - Table A.3 - Combination 1

Partial factor set	A1
Permanent unfavourable action	γ_G = **1.350**
Permanent favourable action	γ_{Gf} = **1.000**
Variable unfavourable action	γ_Q = **1.500**

Variable favourable action γ_{Qf} = **0.000**

Partial factors for soil parameters – Table A.4 - Combination 1

Soil parameter set M1

Angle of shearing resistance $\gamma_{\phi'}$ = **1.00**

Effective cohesion $\gamma_{c'}$ = **1.00**

Weight density γ_{γ} = **1.00**

Water properties

Design water density $\gamma_{w}' = \gamma_{w} / \gamma_{\gamma}$ = **9.8** kN/m^3

Retained soil properties

Design moist density $\gamma_{mr}' = \gamma_{mr} / \gamma_{\gamma}$ = **15** kN/m^3

Design saturated density $\gamma_{sr}' = \gamma_{sr} / \gamma_{\gamma}$ = **15** kN/m^3

Design effective shear resistance angle $\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'})$ = **18** deg

Design wall friction angle $\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'})$ = **9** deg

Base soil properties

Design soil density $\gamma_{b}' = \gamma_{b} / \gamma_{\gamma}$ = **18** kN/m^3

Design effective shear resistance angle $\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'})$ = **18** deg

Design wall friction angle $\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'})$ = **9** deg

Design base friction angle $\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'})$ = **12** deg

Design effective cohesion $c'_{b.d} = c'_{b.k} / \gamma_{c'}$ = **0** kN/m^2

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times [1 + \sqrt{[}\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))]]^2)$ = **0.483**

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[}\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]]^2)$ = **2.359**

Sliding check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_{Gf} \times A_{stem} \times \gamma_{stem}$ = **13.5** kN/m

Wall base $F_{base} = \gamma_{Gf} \times A_{base} \times \gamma_{base}$ = **14.8** kN/m

Base soil $F_{exc\_v} = \gamma_{Gf} \times A_{exc} \times \gamma_{b}'$ = **54.3** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} - F_{water\_u} + F_{exc\_v}$ = **82.6** kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times \text{Surcharge}_Q \times h_{eff}$ = **15** kN/m

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_{w}') \times (h_{sat} + h_{base})^2 / 2$ = **7.4** kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_{w}' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **29.2** kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **0** kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h}$ = **51.6** kN/m

Check stability against sliding

Base soil resistance $F_{exc\_h} = \gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_{b}' \times (h_{pass} + h_{base})^2 / 2$ = **92.5** kN/m

Base friction $F_{friction} = F_{total\_v} \times \tan(\delta_{bb.d})$ = **17.6** kN/m

Resistance to sliding $F_{rest} = F_{exc\_h} + F_{friction}$ = **110** kN/m

Factor of safety $FoS_{sl} = F_{rest} / F_{total\_h}$ = **2.132**

PASS - Resistance to sliding is greater than sliding force

Overturning check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_{Gf} \times A_{stem} \times \gamma_{stem}$ = **13.5** kN/m

Wall base $F_{base} = \gamma_{Gf} \times A_{base} \times \gamma_{base} = \mathbf{14.8}$ kN/m

Base soil $F_{exc\_v} = \gamma_{Gf} \times A_{exc} \times \gamma_b' = \mathbf{54.3}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} - F_{water\_u} + F_{exc\_v} = \mathbf{82.6}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times Surcharge_Q \times h_{eff} = \mathbf{15}$ kN/m

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{7.4}$ kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{29.2}$ kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m

Base soil $F_{exc\_h} = \max(-\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (h_{pass} + h_{base})^2 / 2, -(F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h})) = \mathbf{-51.6}$ kN/m

Total $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{exc\_h} = \mathbf{0}$ kN/m

Overturning moments on wall

Surcharge load $M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} = \mathbf{15.8}$ kNm/m

Saturated retained soil $M_{sat\_OT} = F_{sat\_h} \times x_{sat\_h} = \mathbf{5.2}$ kNm/m

Water $M_{water\_OT} = F_{water\_h} \times x_{water\_h} + 0$ kNm/m $= \mathbf{20.4}$ kNm/m

Moist retained soil $M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m

Total $M_{total\_OT} = M_{sur\_OT} + M_{sat\_OT} + M_{water\_OT} + M_{moist\_OT} = \mathbf{41.4}$ kNm/m

Restoring moments on wall

Wall stem $M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{24.7}$ kNm/m

Wall base $M_{base\_R} = F_{base} \times x_{base} = \mathbf{14.6}$ kNm/m

Base soil $M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} = \mathbf{81.6}$ kNm/m

Total $M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} = \mathbf{120.9}$ kNm/m

Check stability against overturning

Factor of safety $FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{2.922}$

PASS - Maximum restoring moment is greater than overturning moment

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem} = \mathbf{18.2}$ kN/m

Wall base $F_{base} = \gamma_G \times A_{base} \times \gamma_{base} = \mathbf{20}$ kN/m

Base soil $F_{pass\_v} = \gamma_G \times A_{pass} \times \gamma_b' = \mathbf{73.3}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{pass\_v} = \mathbf{111.5}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times Surcharge_Q \times h_{eff} = \mathbf{15}$ kN/m

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{7.4}$ kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{29.2}$ kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m

Base soil $F_{pass\_h} = \max(-\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2, -(F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h})) = \mathbf{-51.6}$ kN/m

Total $F_{total\_h} = \max(F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} \times \tan(\delta_{bb.d}), 0 \text{ kN/m}) = \mathbf{0}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{33.3}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{19.8}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-15.8}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-5.2}$ kNm/m

Water	$M_{water} = -F_{water\_h} \times x_{water\_h} =$ **-20.4** kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} =$ **0** kNm/m
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} =$ **97.6** kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_{sat} + M_{water} + M_{moist} + M_{pass} =$ **109.2** kNm/m

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} =$ **979** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 =$ **-9** mm
Loaded length of base	$l_{load} = 2 \times \bar{x} =$ **1958** mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{load} =$ **57** kN/m^2
Bearing pressure at heel	$q_{heel} =$ **0** kN/m^2
Effective overburden pressure	$q = (t_{base} + d_{cover}) \times \gamma_b' - (t_{base} + d_{cover} + h_{water}) \times \gamma_w' =$ **17.2** kN/m^2
Design effective overburden pressure	$q' = q / \gamma_\gamma =$ **17.2** kN/m^2
Bearing resistance factors	$N_q = Exp(\pi \times \tan(\phi'_{b.d})) \times (\tan(45 deg + \phi'_{b.d} / 2))^2 =$ **5.258**
	$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) =$ **13.104**
	$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) =$ **2.767**
Foundation shape factors	$s_q = 1$
	$s_\gamma = 1$
	$s_c = 1$
Load inclination factors	$H = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} =$ **0** kN/m
	$V = F_{total\_v} =$ **111.5** kN/m
	$m = 2$
	$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m =$ **1**
	$i_\gamma = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} =$ **1**
	$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) =$ **1**

Net ultimate bearing capacity

$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_b' - \gamma_w') \times l_{load} \times N_\gamma \times s_\gamma \times i_\gamma =$ **112.6** kN/m^2

Factor of safety $FoS_{bp} = n_f / \max(q_{toe}, q_{heel}) =$ **1.977**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Design approach 1

Partial factors on actions - Table A.3 - Combination 2

Partial factor set	A2
Permanent unfavourable action	$\gamma_G =$ **1.000**
Permanent favourable action	$\gamma_{Gf} =$ **1.000**
Variable unfavourable action	$\gamma_Q =$ **1.300**
Variable favourable action	$\gamma_{Qf} =$ **0.000**

Partial factors for soil parameters – Table A.4 - Combination 2

Soil parameter set	M2
Angle of shearing resistance	$\gamma_{\phi'} =$ **1.25**
Effective cohesion	$\gamma_{c'} =$ **1.25**
Weight density	$\gamma_\gamma =$ **1.00**

Water properties

Design water density $\gamma_w' = \gamma_w / \gamma_\gamma =$ **9.8** kN/m^3

Retained soil properties

Design moist density	$\gamma_{mr}' = \gamma_{mr} / \gamma_\gamma =$ **15** kN/m^3
Design saturated density	$\gamma_{sr}' = \gamma_{sr} / \gamma_\gamma =$ **15** kN/m^3
Design effective shear resistance angle	$\phi'_{r.d} = atan(\tan(\phi'_{r.k}) / \gamma_{\phi'}) =$ **14.6** deg
Design wall friction angle	$\delta_{r.d} = atan(\tan(\delta_{r.k}) / \gamma_{\phi'}) =$ **7.2** deg

Base soil properties

Design soil density	$\gamma_b' = \gamma_b / \gamma_\gamma =$ **18** kN/m^3
Design effective shear resistance angle	$\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'}) =$ **14.6** deg
Design wall friction angle	$\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'}) =$ **7.2** deg
Design base friction angle	$\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'}) =$ **9.7** deg
Design effective cohesion	$c'_{b.d} = c'_{b.k} / \gamma_{c'} =$ **0** kN/m^2

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times [1 + \sqrt{[\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))]}]^2) =$ **0.553**
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]}]^2) =$ **1.965**

Sliding check

Vertical forces on wall

Wall stem	$F_{stem} = \gamma_{Gf} \times A_{stem} \times \gamma_{stem} =$ **13.5** kN/m
Wall base	$F_{base} = \gamma_{Gf} \times A_{base} \times \gamma_{base} =$ **14.8** kN/m
Base soil	$F_{exc\_v} = \gamma_{Gf} \times A_{exc} \times \gamma_b' =$ **54.3** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} - F_{water\_u} + F_{exc\_v} =$ **82.6** kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times \text{Surcharge}_Q \times h_{eff} =$ **15** kN/m
Saturated retained soil	$F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 =$ **6.3** kN/m
Water	$F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **21.6** kN/m
Moist retained soil	$F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **0** kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} =$ **42.9** kN/m

Check stability against sliding

Base soil resistance	$F_{exc\_h} = \gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (h_{pass} + h_{base})^2 / 2 =$ **77.4** kN/m
Base friction	$F_{friction} = F_{total\_v} \times \tan(\delta_{bb.d}) =$ **14** kN/m
Resistance to sliding	$F_{rest} = F_{exc\_h} + F_{friction} =$ **91.4** kN/m
Factor of safety	$FoS_{sl} = F_{rest} / F_{total\_h} =$ **2.132**

PASS - Resistance to sliding is greater than sliding force

Overturning check

Vertical forces on wall

Wall stem	$F_{stem} = \gamma_{Gf} \times A_{stem} \times \gamma_{stem} =$ **13.5** kN/m
Wall base	$F_{base} = \gamma_{Gf} \times A_{base} \times \gamma_{base} =$ **14.8** kN/m
Base soil	$F_{exc\_v} = \gamma_{Gf} \times A_{exc} \times \gamma_b' =$ **54.3** kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} - F_{water\_u} + F_{exc\_v} =$ **82.6** kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times \text{Surcharge}_Q \times h_{eff} =$ **15** kN/m
Saturated retained soil	$F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 =$ **6.3** kN/m
Water	$F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 =$ **21.6** kN/m
Moist retained soil	$F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **0** kN/m
Base soil	$F_{exc\_h} = \max(-\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (h_{pass} + h_{base})^2 / 2, -(F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h})) =$ **-42.9** kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{exc\_h} =$ **0** kN/m

Overturning moments on wall

Surcharge load	$M_{sur\_OT} = F_{sur\_h} \times x_{sur\_h} = \mathbf{15.7}$ kNm/m
Saturated retained soil	$M_{sat\_OT} = F_{sat\_h} \times x_{sat\_h} = \mathbf{4.4}$ kNm/m
Water	$M_{water\_OT} = F_{water\_h} \times x_{water\_h} + 0$ kNm/m $= \mathbf{15.1}$ kNm/m
Moist retained soil	$M_{moist\_OT} = F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m
Total	$M_{total\_OT} = M_{sur\_OT} + M_{sat\_OT} + M_{water\_OT} + M_{moist\_OT} = \mathbf{35.3}$ kNm/m

Restoring moments on wall

Wall stem	$M_{stem\_R} = F_{stem} \times x_{stem} = \mathbf{24.7}$ kNm/m
Wall base	$M_{base\_R} = F_{base} \times x_{base} = \mathbf{14.6}$ kNm/m
Base soil	$M_{exc\_R} = F_{exc\_v} \times x_{exc\_v} - F_{exc\_h} \times x_{exc\_h} = \mathbf{75.5}$ kNm/m
Total	$M_{total\_R} = M_{stem\_R} + M_{base\_R} + M_{exc\_R} = \mathbf{114.8}$ kNm/m

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total\_R} / M_{total\_OT} = \mathbf{3.256}$

PASS - Maximum restoring moment is greater than overturning moment

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem} = \mathbf{13.5}$ kN/m
Wall base	$F_{base} = \gamma_G \times A_{base} \times \gamma_{base} = \mathbf{14.8}$ kN/m
Base soil	$F_{pass\_v} = \gamma_G \times A_{pass} \times \gamma_b' = \mathbf{54.3}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{pass\_v} = \mathbf{82.6}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_Q \times Surcharge_Q \times h_{eff} = \mathbf{15}$ kN/m
Saturated retained soil	$F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{6.3}$ kN/m
Water	$F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{21.6}$ kN/m
Moist retained soil	$F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m
Base soil	$F_{pass\_h} = \max(-\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2, -(F_{sat\_h} + F_{moist\_h} + F_{water\_h} + F_{sur\_h})) = \mathbf{-42.9}$ kN/m
Total	$F_{total\_h} = \max(F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} - F_{total\_v} \times \tan(\delta_{bb.d}), 0$ kN/m$) = \mathbf{0}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{24.7}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{14.6}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-15.7}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-4.4}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-15.1}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m
Base soil	$M_{pass} = F_{pass\_v} \times x_{pass\_v} - F_{pass\_h} \times x_{pass\_h} = \mathbf{75.5}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_{sat} + M_{water} + M_{moist} + M_{pass} = \mathbf{79.6}$ kNm/m

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{963}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{-25}$ mm
Loaded length of base	$l_{load} = 2 \times \bar{x} = \mathbf{1926}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{load} = \mathbf{42.9}$ kN/m^2
Bearing pressure at heel	$q_{heel} = \mathbf{0}$ kN/m^2
Effective overburden pressure	$q = (t_{base} + d_{cover}) \times \gamma_b' - (t_{base} + d_{cover} + h_{water}) \times \gamma_w' = \mathbf{17.2}$ kN/m^2

Design effective overburden pressure	$q' = q / \gamma_{\gamma} = \mathbf{17.2}$ kN/m^2
Bearing resistance factors	$N_q = Exp(\pi \times \tan(\phi'_{b.d})) \times (\tan(45 deg + \phi'_{b.d} / 2))^2 = \mathbf{3.784}$
	$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) = \mathbf{10.711}$
	$N_{\gamma} = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) = \mathbf{1.447}$
Foundation shape factors	$s_q = 1$
	$s_{\gamma} = 1$
	$s_c = 1$
Load inclination factors	$H = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{0}$ kN/m
	$V = F_{total\_v} = \mathbf{82.6}$ kN/m
	$m = 2$
	$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m = \mathbf{1}$
	$i_{\gamma} = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} = \mathbf{1}$
	$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) = \mathbf{1}$

Net ultimate bearing capacity

$$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_b' - \gamma_w') \times l_{load} \times N_{\gamma} \times s_{\gamma} \times i_{\gamma} = \mathbf{76.5} \text{ kN/m}^2$$

Factor of safety	$FoS_{bp} = n_f / \max(q_{toe}, q_{heel}) = \mathbf{1.783}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

SWIMMING POOL RETAINING WALL 1 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 2.9.16

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2
Partial factor for concrete - Table 2.1N	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm
Ultimate strain - Table 3.1	$\varepsilon_{cu2} = \mathbf{0.0035}$
Shortening strain - Table 3.1	$\varepsilon_{cu3} = \mathbf{0.0035}$
Effective compression zone height factor	$\lambda = \mathbf{0.80}$
Effective strength factor	$\eta = \mathbf{1.00}$
Bending coefficient k_1	$K_1 = \mathbf{0.40}$
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$
Bending coefficient k_3	$K_3 = \mathbf{0.40}$
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$

Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = \mathbf{500}$ N/mm^2
Modulus of elasticity of reinforcement	$E_s = \mathbf{200000}$ N/mm^2
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = \mathbf{1.15}$
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = \mathbf{435}$ N/mm^2

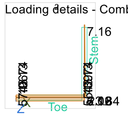
Cover to reinforcement

Front face of stem	$c_{sf} = \mathbf{40}$ mm

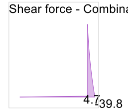
Rear face of stem c_{sr} = **75** mm

Top face of base c_{bt} = **40** mm

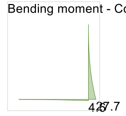
Bottom face of base c_{bb} = **75** mm



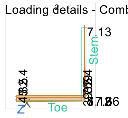
Loading details - Combination No.1 - kN/m



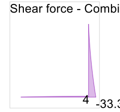
Shear force - Combination No.1 - kN/m


Bending moment - Combination No.1 - kNm/m

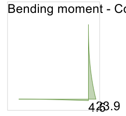
Loading details - Combination No.2 - kN/m



Shear force - Combination No.2 - kN/m



Bending moment - Combination No.2 - kNm/m



Check stem design at base of stem

Depth of section | $h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 | $M = \mathbf{27.7}$ kNm/m

Depth to tension reinforcement | $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{219}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.019}$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm | $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{208}$ mm

Depth of neutral axis | $x = 2.5 \times (d – z) = \mathbf{27}$ mm

Area of tension reinforcement required | $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{306}$ mm^2/m

Tension reinforcement provided | 12 dia.bars @ 200 c/c

Area of tension reinforcement provided | $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{565}$ mm^2/m

Minimum area of reinforcement - exp.9.1N | $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{330}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) | $A_{sr.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.583}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio | $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = \mathbf{0.005}$

Required tension reinforcement ratio	$\rho = A_{sr.req} / d = \mathbf{0.001}$
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$
Structural system factor - Table 7.4N	$K_b = \mathbf{0.4}$
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$
Actual span to depth ratio	$h_{stem} / d = \mathbf{8.2}$

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

Crack control - Section 7.3

Limiting crack width	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment	$M_{sls} = \mathbf{16.6}$ kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{140.9}\ N/mm^2$
Load duration	Long term
Load duration factor	$k_t = \mathbf{0.4}$
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{90875}\ mm^2/m$
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm} = \mathbf{2.9}\ N/mm^2$
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.006}$
Modular ratio	$\alpha_e = E_s / E_{cm} = \mathbf{6.091}$
Bond property coefficient	$k_1 = \mathbf{0.8}$
Strain distribution coefficient	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{583}$ mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.246}$ mm
	$w_k / w_{max} = \mathbf{0.821}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	$V = \mathbf{39.8}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.956}$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.003}$
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524}\ N/mm^2$
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} = \mathbf{114.8}$ kN/m
	$V / V_{Rd.c} = \mathbf{0.346}$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{300}\ mm^2/m$
Maximum spacing of reinforcement – cl.9.6.3(2)	$s_{sx\_max} = \mathbf{400}$ mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{393}\ mm^2/m$

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

Check base design at toe

Depth of section	$h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 2 — $M = \mathbf{4.6}$ kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{219}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.003}$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{208}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{27}$ mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{51}$ mm^2/m

Tension reinforcement provided — 12 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{565}$ mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{330}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.583}$

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width — $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1 — $\psi_2 = \mathbf{0.6}$

Serviceability bending moment — $M_{sls} = \mathbf{3}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{25.6}$ N/mm^2

Load duration — Long term

Load duration factor — $k_t = \mathbf{0.4}$

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{90875}$ mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.006}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

Bond property coefficient — $k_1 = \mathbf{0.8}$

Strain distribution coefficient — $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{583}$ mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.045}$ mm

$w_k / w_{max} = \mathbf{0.149}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{4.7}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.956}$

Longitudinal reinforcement ratio — $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{114.8}$ kN/m

$V / V_{Rd.c} = \mathbf{0.041}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **113** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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

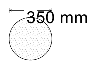
PILE ANALYSIS FOR 100KN FORCE AND COHESIVE STRATA

In accordance with EN 1997-1:2004 incorporating Corrigendum dated February 2009 and the UK national annex

Tedds calculation version 1.0.07

Design summary

Description	Unit	Actual	Allowable	Utilisation	Result
Axial, compression	**kN**	**100**	**422.9**	**0.236**	**PASS**



Pile details

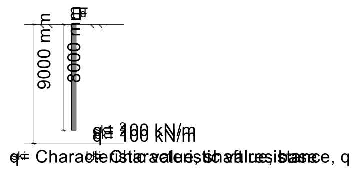
Installation method | Driven
Shape | 350 mm diameter
Length | L = **8000** mm

Material details

Material | Concrete
Concrete strength class | C25/30
Part. factor, concrete (EN1992-1-1 cl. 2.4.2.4(1)) | γ_C = **1.50**
Coefficient α_{cc} (EN1992-1-1 cl. 3.1.6(1)) | α_{cc} = **0.85**
Characteristic compression cylinder strength | f_{ck} = **25** N/mm^2
Design comp. strength (EN1992-1-1 cl. 3.1.6(1)) | $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **14.2** N/mm^2
Mean value of cyl. strength (EN1992-1-1 Table 3.1) | $f_{cm} = f_{ck}$ + 8 MPa = **33.0** N/mm^2
Secant mod. of elasticity (EN1992-1-1 Table 3.1) | E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **31.5** kN/mm^2
Modulus of elasticity | $E = E_{cm}$ = **31.5** kN/mm^2

Geometric properties

Pile section depth | h = **350** mm
Bearing area | $A_{bearing} = \pi \times h^2 / 4$ = **0.096** m^2
Pile perimeter | $Perim_{pile} = \pi \times h$ = **1.1** m
Moment of inertia | $I = \pi \times h^4 / 64$ = **73662** cm^4



Action details

Characteristic perm. unfav. action, compression $G_{c,k,unfav}$ = **100** kN

Characteristic perm. fav. action, compression $G_{c,k,fav}$ = **0** kN

Characteristic variable unfav. action, compression $Q_{c,k}$ = **0** kN

Characteristic perm. unfav. action, tension $G_{t,k,unfav}$ = **0** kN

Characteristic perm. fav. action, tension $G_{t,k,fav}$ = **0** kN

Characteristic variable unfav. action, tension $Q_{t,k}$ = **0** kN

Geotechnical partial and model factors:

Design approach 1:

Model factor on axial resistance γ_{model} = **1.40**

Permanent unfavourable, A1 (Table A.3) $\gamma_{G,unfav,A1}$ = **1.35**

Permanent favourable, A1 (1) $\gamma_{G,fav,A1}$ = **1.00**

Variable unfavourable, A1 (Table A.3) $\gamma_{Q,A1}$ = **1.50**

Permanent unfavourable, A2 (Table A.3) $\gamma_{G,unfav,A2}$ = **1.00**

Permanent favourable, A2 (Table A.3) $\gamma_{G,fav,A2}$ = **1.00**

Variable unfavourable, A2 (Table A.3) $\gamma_{Q,A2}$ = **1.30**

Characteristic axial resistance

Characteristic axial base resistance $R_{bk} = A_{bearing} \times q_{bk}$ = **9.6** kN

Characteristic axial shaft resistance per stratum

Stratum 1 $R_{sk1} = q_{sk1} \times Perim_{pile} \times (L - D_{strata1})$ =**879.6** kN

Characteristic total axial shaft resistance $R_{sk} = R_{sk1}$ = **879.6** kN

Axial compressive resistance

Load combination 1: A1 + M1 + R1

Design compression action $F_{c,d,C1} = \gamma_{G,unfav,A1} \times G_{c,k,unfav} - \gamma_{G,fav,A1} \times G_{c,k,fav} + \gamma_{Q,A1} \times Q_{c,k}$

$F_{c,d,C1}$ = **135** kN

Partial resistance factor, bearing (Table A.NA.6) $\gamma_{b,R1}$ = **1.00**

Partial resistance factor, shaft (Table A.NA.6) $\gamma_{s,R1}$ = **1.00**

Design compressive resistance $R_{c,d,C1} = (R_{bk} / \gamma_{b,R1} + R_{sk} / \gamma_{s,R1}) / \gamma_{model}$ = **635.2** kN

$F_{c,d,C1} / R_{c,d,C1}$ = **0.213**

PASS - Design compressive resistance exceeds design load

Load combination 2: A2 + M1 + R4

Design compression action $F_{c,d,C2} = \gamma_{G,unfav,A2} \times G_{c,k,unfav} - \gamma_{G,fav,A2} \times G_{c,k,fav} + \gamma_{Q,A2} \times Q_{c,k}$

$F_{c,d,C2}$ = **100** kN

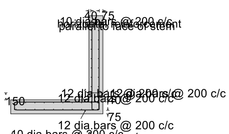
Partial resistance factor, bearing (Table A.NA.6) $\gamma_{b,R4}$ = **1.70**

Partial resistance factor, shaft (Table A.NA.6) $\gamma_{s,R4}$ = **1.50**

Design compressive resistance $R_{c,d,C2} = (R_{bk} / \gamma_{b,R4} + R_{sk} / \gamma_{s,R4}) / \gamma_{model}$ = **422.9** kN

$F_{c,d,C2}$ / R $_{c,d,C2}$ = **0.236**

PASS - Design compressive resistance exceeds design load



Reinforcement details

A. Drawings

End of Report.

General Notes

- CDM REGULATIONS 2015

The client must abide by the Construction Design and Management Regulations 2015. The client must appoint a contractor, if more than one contractor is to be involved, the client will need to appoint (in writing) a principal designer (to plan, manage and coordinate the planning and design work) and a principal contractor (to plan, manage and coordinate the construction and ensure there are arrangements in place for managing and organising the project).

Domestic clients
The domestic client is to appoint a principal designer and a principal contractor when there is more than one contractor, if not your duties will automatically transferred to the contractor or principal contractor.

The designer can take on the duties, provided there is a written agreement between you and the designer to do so.

The Health and Safety Executive is to be notified as soon as possible before construction work starts if the works:

(a) Last longer than 30 working days and has more than 20 workers working simultaneously at any point in the project.
Or:
(b) Exceeds 500 person days.

T. PARTY WALL ACT
The owner, should they need to do so under the requirements of the Party Wall Act 1996, has a duty to serve a Party Structure Notice on any adjoining owner if building work on, to or near an existing Party Wall involves any of the following:
• Support of beam
• Insertion of DPC through wall
• Raising a wall or cutting off projections
• Demolition and rebuilding
• Underpinning
• Insertion of lead flashings
• Excavations within 3 metres of an existing structure where the new foundations will go deeper than adjoining foundations, or within 6 metres of an existing structure where the new foundations are within a 45 degree line of the adjoining foundations.
A Party Wall Agreement is to be in place prior to start of works on site.

U. MATERIALS AND WORKMANSHIP
All works are to be carried out in a workmanlike manner. All materials and workmanship must comply with Regulation 7 of the Building Regulations, all relevant British Standards, European Standards, Agreement Certificates, Product Certification of Schemes (Kite Marks) etc. Products conforming to a European technical standard or harmonised European product should have a CE marking.

V. These drawings/documents and the forms of construction indicated do not constitute the approved documents until Building Regulation approval and/or Planning Permission has been granted. If any construction work is commenced prior these approvals being granted it is undertaken at the client's/contractor's own risk.

W. This document should be read in conjunction with all other relevant engineering details, drawings & specifications.

X. The contractor shall make arrangements for the safe, secure and proper storage of all materials on site. The contractor is to avoid damage to public and private property and is to make good or pay for re-instatement of any damage caused.

Y. All statutory undertakers and services must be notified of any proposed works required. The contractor must ensure that all notices (e.g. Demolition, Building Regulation inspection) are given as required and that all relevant licenses obtained and relevant notices given (EG scaffolding, skips on the highway etc) and that all safety barriers etc are provided.

Z. If any construction work is commenced prior to the relevant approvals being granted it is undertaken at the client/contractor's own risk.

AA. Do not scale the drawings. If in doubt – ask.

BB. Any discrepancies should be reported to the design engineer immediately, so that clarification can be sought prior to the commencement of works.

CC. All works are to be in accordance with council and building control specifications and standards.

DD. Contractor to locate all utility and drainage locations and co-ordinate safe working procedures before any excavation works take place.

EE. Where applicable existing manhole covers and utility covers are to be adjusted to new proposed levels.

FF. THERMAL BRIDGING
Care shall be taken to limit the occurrence of thermal bridging in the insulation layers caused by gaps within the thermal element, (i.e. around windows and door openings). Reasonable provision shall also be made to ensure the extension is constructed to minimise unwanted air leakage through the new building fabric.

GG. EXISTING STRUCTURE
Existing structure including foundations, beams, walls and lintels carrying new and altered loads are to be exposed and checked for adequacy prior to commencement of work and as required by the Building Control Officer.

HH. BEAMS
Supply and install new structural elements such as new beams, roof structure, floor structure, bearings, and padstones in

accordance with the Structural Engineer's calculations and details. New steel beams to be encased in 15mm Gyproc FireLine board with staggered joints, Gyproc FireCase or painted in Nullifire S or similar intumescent paint to provide 1 hour fire resistance as agreed with Building Control. All fire protection to be installed as detailed by specialist manufacturer.

II. FLAT ROOF RESTRAINT

100m x 50mm C16 grade timber wall plates to be strapped to walls with 1000mm x 30mm x 5mm galvanised mild steel straps at maximum 2.0m centres fixed to internal wall faces.

JJ. STRAPPING FOR PITCHED ROOF

Gable walls should be strapped to roofs at 2m centres. All external walls running parallel to roof rafters to be restrained at roof level using 1000mm x 30mm x 5mm galvanised mild steel horizontal straps or other approved to BSEN 845-1 built into walls at max 2000mm centres and to be taken across minimum 3 rafters and screw fixed. Provide solid noggins between rafters at strap positions. All wall plates to be 100 x 50mm fixed to inner skin of cavity wall using 30mm x 5mm x 1000mm galvanized metal straps or other approved to BSEN 845-1 at maximum 2m centres.

KK. STRAPPING OF FLOORS

Provide lateral restraint where joists run parallel to walls, floors are to be strapped to walls with 1000mm x 30mm x 5mm galvanised mild steel straps or other approved in compliance with BS EN 845-1 at max 2.0m centres, straps to be taken across minimum of 3 joists. Straps to be built into walls. Provide 38mm wide x ¾ depth solid noggins between joists at strap positions.

LL. LINTELS

- For uniformly distributed loads and standard 2 storey domestic loadings only

Lintel widths are to be equal to wall thickness. All lintels over 750mm sized internal door openings to be 65mm deep pre-stressed concrete plank lintels. 150mm deep lintels are to be used for 900mm sized internal door openings. Lintels to have a minimum bearing of 150mm on each end. Any existing lintels carrying additional loads are to be exposed for inspection at commencement of work on site. All pre-stressed concrete lintels to be designed and manufactured in accordance with BS EN 1992-1-1, with a concrete strength of 50 or 40 N/mm² and incorporating steel strands to BS 5896 to support loadings assessed to BS 5977 Part 1.

For other structural openings provide proprietary insulated steel lintels suitable for spans and loadings in compliance with Approved Document A and lintel manufacturer's standard tables. Stop ends, DPC trays and weep holes to be provided above all externally located lintels.

MM. OPENINGS AND RETURNS

An opening or recess greater than 0.1m² shall be at least 550mm from the supported wall (measured internally).

NN. STRIP FOUNDATION

Provide 600mm x 600mm concrete foundation, concrete mix to conform to BS EN 206-1 and BS 8500-2. All foundations to be a minimum of 1000mm below ground level, exact depth to be agreed on site with Building Control Officer to suit site conditions. All constructed in accordance with 2010 Building Regulations A1/2 and BS 8004:2015 Code of Practice for Foundations. Ensure foundations are constructed below invert level of any adjacent drains. Base of foundations supporting internal walls to be min 600mm below ground level. Sulphate resistant cement to be used if required. Please note that should any adverse soil conditions be found or any major tree roots in excavations, the Building Control Officer is to be contacted and the advice of a structural engineer should be sought.

OO. STEPPED FOUNDATION (strip)

Stepped foundations should overlap by twice the height of the step, by the thickness of the foundations, or 300mm, whichever is the greater. The height of the step should not be greater than the thickness of the foundation.

PP. PIPES PASSING THROUGH TRENCH FOUNDATIONS

The load-bearing capability of foundations must not be affected where services pass through. Services should be either sleeved with flexible joints, pipe to be within 150mm of foundation and connected on each side by of rocker pipes by length of at most 600mm with flexible joints or,

Pipes to pass through a suitably strengthened opening in the foundation, i.e. foundation shuttered and a suitable lintel provided over, sufficient space for movement is required to leave to ensure that the drain is capable of maintaining line and gradient. Opening to be masked with granular backfill (pea shingle) around pipe.

DPC to be provided as required by BCO.

Advice from Building Control to be sought on suitability of pipe running through foundation before construction.

QQ. PIPES PASSING THROUGH WALLS

Walls above pipes passing through substructure walls to be supported on suitable lintel on semi-engineering bricks. Pipe to be provided with a 50mm clearance all round, opening to be masked with granular backfill (pea shingle) around pipe. DPC to be provided as required by BCO.

RR. DPC

Provide horizontal strip polymer (hyload) damp proof course to both internal and external skins minimum 150mm above external ground level. New DPC to be made continuous with existing DPC's and with floor DPM. Vertical DPC to be installed at all reveals where cavity is closed.

SS. WALL TIES

All walls constructed using stainless steel vertical twist type retaining wall ties built in at 750mm ctrs horizontally, 450mm vertically and 225mm ctrs at reveals and corners in staggered rows. Wall ties to be suitable for cavity width and in accordance with BS EN 845

TT. CAVITIES

Provide cavity trays over openings. All cavities to be closed at eaves and around openings using Thermabate or similar non combustible insulated cavity closers. Provide vertical DPCs around openings and abutments. All cavity trays must have 150mm upstands and suitable cavity weep holes (min 2) at max 900mm centres.

UU. EXISTING TO NEW WALL

Cavities in new wall to be made continuous with existing where possible to ensure continuous weather break. If a continuous cavity cannot be achieved, where new walls abuts the existing walls provide a movement joint with vertical DPC. All tied into existing construction with suitable proprietary stainless steel profiles.

VV. MOVEMENT JOINTS

Movement joints to be provided at the following maximum spacing:
Clay brickwork - 12m.
Calcium silicate brick - 7.5-9m.
Lightweight concrete block - density not exceeding 1,500kg/m3 - 6m.
Dense concrete block - density exceeding 1,500kg/m3 – 7.5-9m.
Any masonry in a parapet wall (length to height ratio greater than 3:1) - half the above spacings and 1.5m from corners.
Movement joint widths for clay bricks to be not less than 1.3mm/m i.e. 12m = 16mm and for other masonry not less than 10mm.
Additional movement joints may be required where the aspect ratio of the wall (length :height) is more than 3:1.
Considerations to be given to BS 5628 Code of practice for use of masonry.

WW. CAVITY BARRIERS

30 minute fire resistant cavity barriers to be provided at at tops of walls, gable end walls and vertically at junctions with separating walls & horizontally at separating walls with cavity tray over installed according to manufacturer's details.

XX. UNDERGROUND FOUL DRAINAGE

Underground drainage to consist of 100mm diameter UPVC proprietary pipe work to give a 1:40 fall. Surround pipes in 100mm pea shingle. Provide 600mm suitable cover (900mm under drives). Shallow pipes to be covered with 100mm reinforced concrete slab over compressible material. Provide rodding access at all changes of direction and junctions. All below ground drainage to comply with BS EN 1401-1.

YY. INSPECTION CHAMBERS

Underground quality proprietary UPVC 450mm diameter inspection chambers to be provided at all changes of level, direction, connections and every 45m in straight runs. Inspection chambers to have bolt down double sealed covers in buildings and be adequate for vehicle loads in driveways.

ZZ. PIPEWORK THROUGH WALLS

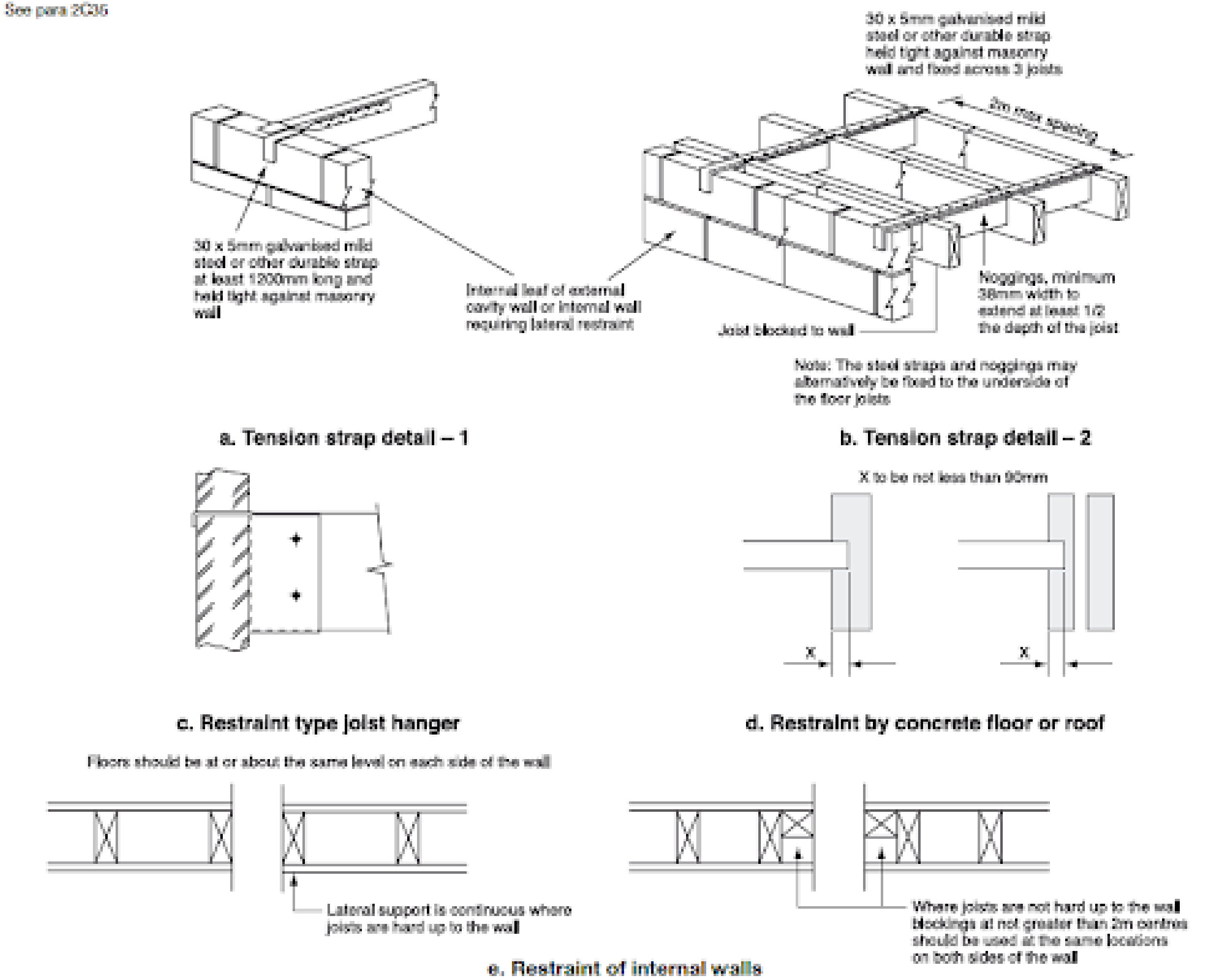
Where new pipework passes through external walls form rocker joints either side wall face of max length 600mm with flexible joints with short length of pipe bedded in wall.
Alternatively provide 75mm deep pre-cast concrete plank lintels over drain to form opening in wall to give 50mm space all round pipe: mask opening both sides with rigid sheet material and compressible sealant to prevent entry of fill or vermin.

AAA. SOIL AND VENT PIPE

Svp to be extended up in 110mm dia UPVC and to terminate min 900mm above any openings within 3m. Provide a long radius bend at foot of SVP.

Diagram 15 Lateral support by floors

See para 2C35



a. Tension strap detail – 1

b. Tension strap detail – 2

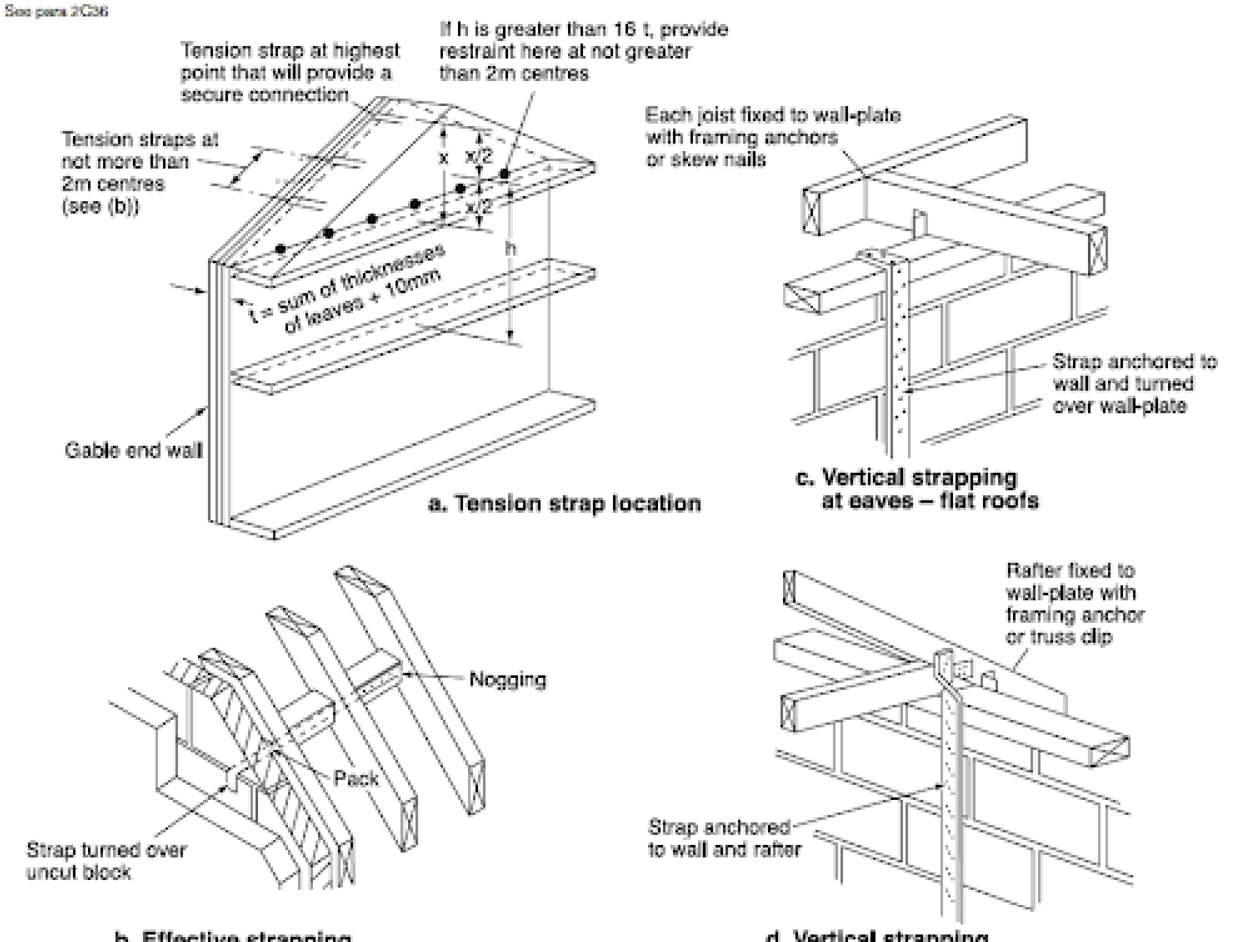
c. Restraint type joist hanger

d. Restraint by concrete floor or roof

e. Restraint of internal walls

Diagram 16 **Lateral support at roof level**

See para 2C36



a. Tension strap location

c. Vertical strapping at eaves – flat roofs

b. Effective strapping at gable wall

d. Vertical strapping at eaves – pitched roofs

Adkins Consultants Terms and Conditions

1. Additional Services
In the event of additional services being required, the fees will be adjusted by prior agreement between us.

2. Diligence and Care
Adkins Consultants will exercise reasonable care, skill and diligence in the performance of the Services.

3. Disbursements and Expenses
Disbursements and expenses are not included in our quoted fees unless stated to the contrary in our fee proposal.

4. Document Formats
The proposed fees allow for a digital copy usually in PDF format.
Hard copies of documents may be subject to a fee dependent on the type of document. Our printing costs are available upon request.

5. Copyright
Adkins Consultants retain copyright in any document and works they produce; in all cases the copyright will remain vested in Adkins Consultants. The Client, subject to payment of fees and disbursements due under the Agreement, will have a license to copy and use all such documents for any purpose related to the project in question. They will not have a license to use these documents for any other project and no liability will be held by Adkins Consultants.

6. Value Added Tax
All quotations are subject to the addition of VAT at the current rate.

7. Liability
7.1 None of our employees, partners or consultants individually has a contract with you or owes you a duty of care or personal responsibility. You agree that you will not bring any claim against any such individuals personally in connection with our services."

7.2 The liability of Adkins Consultants Ltd to the client, (or any third party claiming through the client) shall in no circumstance (except in case of death or personal injury) exceed the lesser of:
(a) The amount that can be recovered from our professional indemnity insurance
(b) We will have proportionate liability and this will be limited to 25% (twenty five percent) of any loss.

(c) The diminution in value of the property concerned.

7.3 Adkins Consultants shall have no liability to the Client (or to any third party claiming through the Client) whether in contract or in tort (including but not limited to negligence) or for breach of statutory duty or otherwise for any claim arising in connection with:-
(a) pollution, contamination, terrorism, asbestos or any related risk or
(b) designs or reports prepared by other professionals and specialist sub-contractors/suppliers.
7.4 The limitation period, before which any claim must commence, is six years.
7.5 If you suffer loss as a result of our breach of contract or negligence, our liability shall be limited to a just and equitable proportion of your loss having regard to the extent of responsibility of any other party. Our liability shall not increase by reason of a shortfall in recovery from any other party.

8 Payments

Payment shall be received in full before Adkins Consultants will release any work, save for clients with an account with Adkins Consultants.

8.1 Where such an account is available our invoices will be issued monthly for the work completed in that month or upon completion of the work, whichever occurs soonest. Payment shall become due on submission of the invoice and the final date of payment shall be 7 days after the invoice date. We reserve the right to charge interest at the statutory rate on any overdue amounts. Please note that all invoices not settled within our payment terms will be referred to our debt recovery agents and will be subject to a surcharge of 15% plus VAT in lieu of our recovery charges.

8.2 If you do not have an account with Adkins Consultants Ltd, then
payment is required prior to the release of information, unless agreed otherwise in the quotation, which would be unusual.

8.3 Please note that all invoices not settled within our payment terms will be referred to our debt recovery agents an d will be subject to a surcharge of 15% plus VAT in lieu of our recovery charges.

9. CDM Regulations

Under the current CDM Regulations 2015 it is held to be our responsibility to inform you that the aforementioned regulations may be applicable to your project.
We have attached an information sheet for your benefit and should you have any queries in connection with this, please contact us.

10. Changes in Terms and Conditions

Terms and conditions are liable to change without notice. Amended versions will supersede any printed or electronic versions held in the clients' possession. You can find an up-to-date copy of this Terms and Conditions Statement on our website.

11. Terms and Conditions further information

Unless otherwise agreed in writing by the Company these Conditions will override any terms and conditions stipulated or referred to by the Customer in his order or pre-contract negotiations.

12. Cancellations

Adkins Consultants will only accept cancellations at the discretion of Adkins Consultants unless they are within the cooling-off period of 14 days. Acceptance of the cancellation will only be binding on the company if it is sent in writing. Any cost or expenses incurred by the company up to the date of cancellation and all loss and damage resulting from the cancellation will be paid by the customer to the company, including within the cooling-off period if Adkins Consultants have started works during this time, which is a likely course of events given Adkins Consultant's desire to dispatch customer's instructions at a speedy rate .

13. Exclusions

In dispute of any assertion that anything is excluded, all warranties, conditions and other terms, implied, statutory or otherwise, are expressly excluded excepting so far as they are contained in these conditions or otherwise expressly agreed by Adkins Consultants Ltd in writing. If any legislation makes it unlawful to exclude any term from the Contract this clause will naturally not apply to such.

14. Resolutions of Disputes

14.1 The parties will endeavour to resolve any dispute amicably. Each of them shall in good faith consider any proposal by the other that a dispute be referred to mediation.
14.2 Disputes shall be finally resolved by the English Courts.

15.Governing Law

The Agreement shall be solely within the jurisdiction of and governed by English Courts.