SINCLAIRJOHNSTON

CONSULTING CIVIL AND STRUCTURAL ENGINEERS

STRUCTURAL ENGINEER'S REPORT AND CONSTRUCTION METHOD STATEMENT FOR SUBTERRANEAN DEVELOPMENT AT

51 GLOUCESTER CRESCENT
LONDON
NW1 7EG

Prepared by:

Sian Hill MEng & Ravi Azad MEng (Hons) CEng MICE MIStructE
Sinclair Johnston & Partners Limited
93 Great Suffolk Street
London
SE1 0BX

Tel: **020 7593 1900**
E-mail: **razad@sinclairjohnston.co.uk**

8761 170913 RA

SEPTEMBER 2017
UPDATED DECEMBER 2017

CONTENTS

APPENDICES

1.0 INTRODUCTION

1.1 The following Structural Engineer's Report & Construction Method Statement has been prepared as part of the wider Basement Impact Assessment (BIA) undertaken for the planning application, submitted by UV Architects, for the proposed residential redevelopment at 51 Gloucester Crescent, London NW1 7EG. It is to be read in conjunction with all Architect's and other Consultant's documents submitted with the application.

1.2 The proposals broadly comprise:

- The formation of a new basement under the entirety of the existing ground floor, within the existing building footprint including front and rear light wells.

1.3 This statement has been prepared to address the requirements of Camden's Development Policies and should be read with the Basement Impact Assessment (BIA) Report prepared by Soil Consultants Ltd.

1.4 This statement provides specific details of the excavation, the temporary works, and the construction technique proposed for the development, and investigates the potential impact of the subterranean development on the existing and neighbouring structures.

1.5 The statement is not intended to constitute a structural condition report. Any description of the existing structure is provided based on a non-intrusive, visual inspection without recourse to intrusive investigation or opening up of the existing structure.

1.6 This statement has been prepared by Sian Hill MEng (Engineer) and Ravi Azad MEng CEng MICE MIStructE (Technical Director) at Sinclair Johnston & Partners.

2.0 EXISTING SITE

2.1 The site address is 51 Gloucester Crescent, Camden, London NW1 7EG and is located at approximate National Grid Reference TQ 286 838.

2.2 The site is located within the Camden Town with Primrose Hill Ward and Primrose Hill Conservation Area in the London Borough of Camden.

2.3 The existing site is rectangular in plan shape with approximate dimensions of 10m x 16m. The site is generally level. A map showing the site location is provided in Appendix A of this report.

2.4 The site has a small parking area to the front of the property and a small garden to the rear of the site. The property is situated within a short row of detached dwellings, 51A Gloucester Crescent to the east and 50 Gloucester Crescent to the west.

2.5 The building is set over ground and first floor. The building is of traditional load bearing masonry construction, supporting timber floors and a timber tiled roof. There is a groundbearing slab at ground floor level.

2.6 Access is currently provided to the ground floor of the building off Gloucester Crescent towards the front and via the rear garden to the back. Access to the property is only available via Gloucester Crescent.

2.7 There are no trees present at the front of the site but there is a semi-mature tree in the rear garden of the property to the south-east of the site.

2.8 There are no known below ground tunnels on or close to the site.

2.9 The Environment Agency indicates that the site is located in an area where there is less than a 0.1 per cent (1 in 1000) chance of flooding occurring each year. This is the same as Flood Zone 1, in England.

2.10 As identified in the Camden Flood Risk Management Strategy the site is not in an area at risk of flooding from rivers or the sea. Nor is it in an area that has historically been at risk from surface run-off, groundwater and sewer flooding.

2.11 Site investigation works undertaken (ref. Site Investigation Report) have confirmed that foundations to the building generally consist of corbelled solid brick footings on mass concrete strip foundations.

3.0 GROUND CONDITIONS AND HYDRO-GEOLOGY

3.1 The ground conditions on site consist of made ground to a depth of 4.5m, overlying London Clay, which extends to significant depth.

3.2 Logs from boreholes undertaken at 51 Gloucester Crescent are enclosed, which extend to 6m below current ground level. These logs are considered to be a good representation of the ground conditions across the site.

3.3 Trial pits have been undertaken adjacent to existing walls on site to ascertain the depth, extent and profile of existing wall foundations, and make-up of the underlying ground. Further information can be found in the Site Investigation Report in.

3.4 The following is a summary of the findings of the boreholes, information obtained from desk top study, and information obtained from investigation works undertaken at sites nearby:

i) The existing building is founded on made ground comprising mostly of clays, which contradicts local geological maps for the area, implying the site may have been raised at some stage in the site's history or perhaps the clay was excavated locally. Beneath an initial layer of made ground is London Clay Formation which extends to significant depth.

ii) The site sits 160m south of Grand Union Canal Regent's Canal. The nearest surface water feature is located 140m north of the site.

iii) There are no Zone 2 or Zone 3 floodplains or flood defences within 250m of site.

iv) The site does not sit within an aquifer.

v) The site is not within a Groundwater Source Protection Zone.

vi) Groundwater was encountered within both boreholes. Ground water monitoring was undertaken on 3rd March 2017 & 17th March 2017.

vii) Groundwater monitoring shows the measured groundwater level varies between -2.49m and -2.30m, within the made ground layer which overlies the London Clay.

viii) OS mapping indicates an approximate elevation of 33.0m AOD therefore the groundwater table is likely to be situated at approximately 30.51m – 30.7m AOD. The seasonal range of the groundwater level is expected to be less than 0.2m.

ix) The measured ground water level is approximately 2200mm above the underside of the new basement raft.

x) Ground water flows are expected during excavation. Ground stabilisation works will be required prior to excavation for underpins, to prevent washing in of fines and to minimise the extent of dewatering required.

xi) The proposals will not increase the proportion of hard surfaced-areas and therefore the volume of surface water inflow from surface run-off will not change due to the proposed development.

xii) The development is not considered to impact the surface water regime of the site or adjacent sites.

xiii) The property is in a low probability Radon-affected area.

xiv) It has not been possible to confirm the foundation details to Nos. 51A / 50 Gloucester Crescent or No. 22 Regent's Park Terrace by intrusive investigation due to access restrictions.

xv) Information on the planning section of Camden Council's website shows that No. 51A is founded on a reinforced concrete raft slab. This property does not have a basement. There is no planning history for No. 50 Gloucester Crescent or No. 22 Regent's Park Terrace, and it is therefore assumed that there are no basements under these properties and that they are both founded on shallow spread footings.

4.0 STRUCTURAL PROPOSALS

4.1 Drawings describing the proposed structure are provided in Appendix B. For further information on the proposed scheme, please refer to Architect's information.

4.2 It is proposed to form a new basement under part of the existing ground floor of the property. The basement extends beyond the footprint of the property with front and rear light wells and will be only be accessible internally.

4.3 The new basement sub-structure is to be made up of a reinforced concrete (r.c.) slab, which will act as a raft at basement level, supporting loads from the new basement and loadbearing perimeter walls over in bearing. The existing ground floor perimeter walls will be underpinned with r.c. walls cast in a hit/miss sequence, which will be cast integrally with the basement raft slab.

4.4 The structural make-up of the existing building will remain largely unchanged. The existing ground floor slab over the new basement will be reconstructed as an r.c. slab supported on steel beams bearing onto steel columns built off the r.c. raft slab.

4.5 The basement raft is to be constructed and founded within the London Clay at -4.5m below ground level. The Site Investigation Report classifies the risk of collapsible ground / stability to be low.

4.6 It will be necessary to implement ground stabilisation works in advance of excavation to control water ingress and to limit migration of fines, creating an effectively watertight seal around the site to allow excavation for underpins.

4.7 In this case, low-pressure resin grouting can be used to harden and stabilise soil below the water table, in advance of and during excavation. Specialist geotechnical advice has been sought on this matter and trial excavations using the resin will be undertaken on site in due course. Appendix D gives further information on the resin grouting. The design and implementation of resin grouting works will be specifically addressed within a Basement Construction Plan (BCP).

4.8 Lateral loads due to earth pressures, transient hydrostatic pressures, and surcharge pressures are to be resisted by the walls of the reinforced concrete box around the perimeter of the new basement space, which will act as propped walls in both the temporary and permanent condition. The retaining walls will be propped at ground level, lower ground level and basement level by the r.c. slabs at these levels.

4.9 Heave forces due to the unloading of the clay under the made ground, and forces due to a hypothetical raised ground water level are to be resisted by the reinforced concrete basement raft slab, which spans in two-directions between the walls which form the new basement box structure.

4.10 The stiffness of the below ground structure ensures that ground movements are kept within acceptable defined limits.

4.11 Foul water will be pumped from the new basement level to ground level, to allow the foul waste to be removed from site via gravity as per the existing system, into the existing sewer.

4.12 The existing surface water drainage arrangement will remain, where rainwater will be collected at ground level via rainwater pipes and drained via gravity into the existing sewer as per the existing system. There will be additional surface water collection in the front and rear light wells which will be pumped to the existing surface water drainage system at ground level.

Predicted Structural Damage to Neighbouring Properties:

4.13 An initial prediction of structural damage to neighbouring properties has been undertaken in general accordance with CIRIA publication C580 by Soil Consultants Ltd. Calculations and a summary of their findings are provided in the Ground Movement Analysis Report.

4.14 The assessment by Soil Consultants Ltd has found that the category of damage to Nos. 50 & 51A Gloucester Crescent, and 22 Regent's Park Terrace as classified under Burland et al, anticipated from the proposed construction of the new basement is expected to be no worse than Category 0, Negligible.

4.15 The Contractor will be required to monitor ground movements during the works to check the validity of the ground movement analysis and the performance of the temporary works and working methods. A 'traffic light' system of green, amber, red trigger values will be set with specific Contractor actions set against each trigger values. Indicative ground movement trigger levels to be set are as follows:

Traffic Light	**Trigger Value (mm)**	**Contractor Action**
Green	< 2	No action required.
Amber	2 - 4	Notify the CA and Party Wall Surveyor(s). Increase frequency of monitoring. Implement contingency measures if movement continues.
Red	> 4	Notify the CA and the Party Wall Surveyor(s). Implement measures to cease movement and stop work.

4.16 The monitoring method is to be developed further during detailed design. Monitoring will be undertaken prior to the injection of low pressure resin and before any ground works commence, and will continue through to completion of the basement structure.

5.0 CONSTRUCTION METHODOLOGY

5.1 It is envisaged that ground stabilisation works would be undertaken during excavation for underpin walls for the new basement. The basic permeation resin grouting technique methodology is:

i) Installation of injection lances around the perimeter of the proposed basement to the depth and width specified by the specialist contractor. This will require locally breaking out the existing ground bearing ground floor mass concrete slab locally for access and excavating down to approx. 500mm above assumed ground water level. Lances to extend into the underlying Clay member.

ii) Injection of resin at low pressure, which reacts with the water upon contact, forming a gel which foams and binds the granular components of the soil together, limiting water ingress. Resin to be installed around full perimeter of basement walls, with lances removed once resin has been placed.

iii) Excavate through set / hardened resin to form r.c. walls.

5.2 For the purposes of this report, the basement extension is to be constructed using a bottom-up method of construction, as outlined in the following construction sequence:

i) Install reinforced concrete (r.c.) underpinning under ground floor perimeter walls in a typical 1-3-5-2-4 hit/miss sequence, installing resin as required during excavation. Underpin widths to be limited to 900mm to ensure that the existing walls over can effectively arch over temporary excavations.

ii) Given the granular nature of the existing subsoil, faces of excavations to the depth of underpinning may require temporary propping during underpinning works. Localised trench sheeting and props can be used to form excavations for underpins if required.

iii) Reinstate arisings from excavations for underpins in well-compacted layers once each underpin has been cast. Underpins to be packed up to underside of masonry walls with 3:1 sharp sand / cement well rammed in.

iv) Where existing loadbearing masonry walls are to be supported at ground floor level, needle through existing walls at ground floor level with closely spaced temporary steel needles, and support needles off parallel (deep) steel beams set above existing ground floor level, spanning onto the recently-installed perimeter r.c. underpin walls. Appendix E -Temporary propping layout.

v) Excavate to 500mm below top of underpin wall level. Remove waste through front of property.

vi) Install temporary steel waling beams and flying shore props across the width of the basement r.c. walls to provide a prop to the head of underpin walls.

vii) Excavate down to 500mm above top of basement underpin toe level within the perimeter of the new basement. Excavate through the hardened resin, which will serve to stabilise the gravels and limit water ingress into excavation. Dewater from within basement space as excavation progresses through the water table to basement formation level using pumps. Remove excavation material waste towards front of property.

viii) Fix waling beams and flying shores at low level spanning across width of site between waling beams, to provide temporary propping to the base of the underpin walls.

ix) Excavate to base of underpin toe level and install new basement r.c. raft slab, with reinforcement achieving continuity with the r.c. underpin walls. Remove temporary props and temporary steel shores at basement level.

x) Install new steel columns at basement level built off the basement raft slab.

xi) Install new (permanent) steel beams, packed up tight to the underside of the existing walls over with 3:1 sharp sand / cement dry pack well rammed in (between needles), taking support off the perimeter r.c. underpin walls and new steel columns.

xii) Cast new suspended r.c. slab at ground level on profiled metal decking.

xiii) Remove temporary steel needle beams and temporary flying shores and waling beams once ground floor slab has been installed.

5.3 The undertaking of such works to existing buildings is specialist work and Sinclair Johnston & Partners Ltd will be involved in the selection of an appropriate Contractor, who will need relevant expertise and experience in working on these types of projects.

5.4 The Contractor will be required to demonstrate a positive attitude and commitment toward minimising environmental disturbance to local residents and will be required to be registered with the Considerate Contractors Scheme. Impacts on the local amenity due to construction will be strictly controlled and managed by the Contractor.

5.5 Noise, dust, and vibration will be controlled by employing Best Practical Means (BPM) as prescribed in the following legislative documents and the approved code of practice BS 5228:

- The Control of Pollution Act 1972.
- The Health & Safety at Work Act 1974.
- The Environmental Protection Act 1990.
- Construction (Design and Management) Regulations 1994.
- The Clean Air Act 1993.

5.6 General measures to be adopted by the Contractor to reduce noise, dust and vibration include:

- Drop heights to be minimised during any demolition.
- Use of super-silenced plant where feasible.
- Use of well-maintained modern plant.
- Effective noise and vibration monitoring to be implemented.
- Reducing the need to adopt percussive and vibrating machinery.
- Vehicles not to be left idling.
- All loads entering and leaving site are to be covered.
- Measures to be adopted to prevent site runoff of water or mud.
- Water to be used as a dust suppressant.
- Cutting equipment to use water as suppressant or suitable local exhaust ventilation system.
- Skips to be covered.

5.7 It is not anticipated that cutting of any concrete will be required on site. In any case, demolition of any existing concrete will be undertaken using a 'clean' deconstruction method to reduce noise, dust, and vibration. Concrete elements are to be cut into manageable sections using a stitch drilling method to reduce noise, dust, and vibration.

5.8 Where practical, demolition material is to be taken to recycling plants.

5.9 Working hours will be restricted as required by the Local Authority.

5.10 A Chartered Engineer holding MICE or MIStructE accreditation from Sinclair Johnston & Partners Ltd will have an ongoing role on site to monitor that the works are being carried out generally in accordance with the structural design and specification. This role will typically involve weekly / fortnightly site visits throughout the duration of structural works on site.

6.0 TEMPORARY WORKS

6.1 Please refer to Sinclair Johnston & Partners Ltd structural drawings for information on the outline temporary works required for the construction of the new basement. Appendix E.

6.2 The structural arrangement of the existing building is to be generally retained.

6.3 No vertical temporary supports will be required for installation of the reinforced concrete wall under perimeter walls, with all excavations below existing walls undertaken in short 'hit-miss' segments.

6.4 The existing loadbearing walls extending to basement level will be supported directly off reinforced concrete underpin walls prior to excavation for the basement, thus limiting the length of time that temporary support of the walls is required during the construction works.

6.5 Given the granular nature of the existing subsoil, faces of excavations to the depth of underpinning may require temporary propping during underpinning works. Localised trench sheeting and props can be used to form excavations for underpins if required.

6.6 Given the plan dimensions of the new basement, temporary lateral support in the form of waling beams and flying shores will be of relatively short span lengths.

6.7 The types of temporary works required to construct the permanent structure as described above are common forms of temporary works, which most competent contractors will be familiar with.

6.8 The temporary works are to be designed by a qualified and experienced Temporary Works Co-ordinator in accordance with BS 5975 'Code of Practice for Temporary Works Procedures and the Permissible Stress Design of Falsework.'

6.9 Good workmanship will be required for the works to ensure ground movements due to wall deflection are suitably controlled.

7.0 CONSTRUCTION TRAFFIC MANAGEMENT

7.1 The Contractor will be required to develop a detailed Construction Traffic Management Plan for submission to and agreement with the Local Authority.

7.2 However, the following have been considered at the planning stage to mitigate the impacts on the local highways and highway safety:

- All access to the site will be through the front door only.
- Traffic movements are to be scheduled to avoid periods of heavy traffic such as mornings and evenings.
- All deliveries are to be agreed with the Contractor in advance. Any unscheduled deliveries will be turned away.
- Banksmen are to be provided for all site vehicle movements to ensure pedestrian and highway user safety and to ensure congestion is minimised.
- Vehicles are to be sized so as to be suitable for the local highways.
- As all vehicles arriving to and leaving site will be driving on tarmacked roads and no vehicles will need to access site for pick-up or delivery, it is not envisaged that any muck from site will be tracked onto wheels. Nonetheless, it is anticipated that wheel washing facilities will be put in place by the Contractor on site to ensure that site muck is not tracked onto wheels of vehicles leaving site.

8.0 NEIGHBOURING BUILDINGS AND PARTY WALL MATTERS

8.1 The property is a detached building therefore no Party Walls will require underpinning during the construction of the new basement.

8.2 Excavation is within 6m of neighbouring building boundary walls, and therefore full procedures under the Party Wall etc. 1996 Act will apply.

8.3 The structural scheme adopted has been designed with due regard to maintaining the structural stability and integrity of neighbouring buildings & structures and surrounding land. The structural form of the basement and the method of construction have been developed to ensure that lateral deflections, and associated ground movements, are kept within acceptable limits.

8.4 The design and implementation of resin grouting works will be specifically addressed within a Basement Construction Plan (BCP). This will form the basis of Party Wall Awards, which will be in place prior to the commencement of any excavation works.

9.0 CONCLUSIONS

9.1 The structural proposals and construction methodology for the subterranean development at 51 Gloucester Crescent have been developed with due regard to the existing site constraints, the site specific and local ground conditions, the local amenity and the local highway.

9.2 The ground conditions are well understood and have been investigated by boreholes to 6m below existing ground level.

9.3 The site is located in Flood Risk Zone 1. The site has not previously been, or is likely to be, subject to surface water flooding.

9.4 The proposed works and basement development have been shown to be unlikely to detrimentally affect the surface water regime in the local and wider area. The existing pathway for surface water flows will not be altered by the proposals.

9.5 Anticipated ground movements associated with the works can be limited to acceptable values by a combination of the stiffness of the proposed retaining structure, suitably designed temporary works, and good levels of workmanship.

9.6 The proposals demonstrate that:

- The site geology is capable of supporting the loads and construction techniques to be imposed.
- The subterranean development, and associated construction and temporary works, have been developed so as to have no adverse impact on the structural integrity and natural ability for movement of existing and surrounding structures, utilities, infrastructure and man-made cavities, such as tunnels.
- The permanent and temporary works and the method of construction have been developed so that the development will not initiate slope instability.
- The subterranean development has no adverse impact on drainage, sewage, surface water and ground water flows and levels.
- The proposed temporary works, permanent works and construction method have been developed with due regard to the geology and hydrology.
- The existing structure has been investigated and considered when developing the temporary works, permanent works and construction methodology.
- The report describes the engineering details of the scheme, including proposals for the excavation and construction.
- The proposed subterranean development has no adverse impact on existing trees.

9.7 The proposals described herein are a proven form of construction and are designed to maintain the structural stability and integrity of the existing buildings on and around the site.

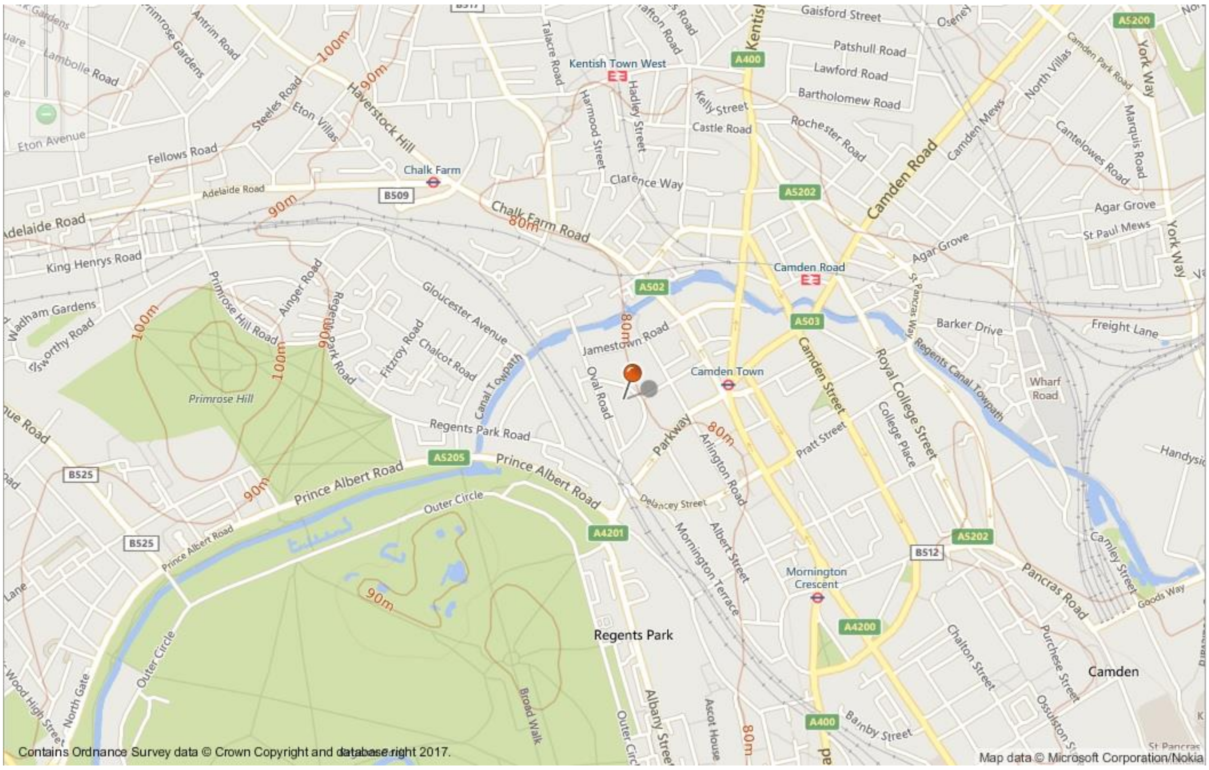
9.8 This report demonstrates that by adopting good construction practices the works can be executed in a safe manner while minimising any impact on the local amenity.

9.9 A Chartered Engineer holding MICE or MIStructE accreditation from Sinclair Johnston & Partners Ltd will have an ongoing role on site to monitor that the works are being carried out generally in accordance with the structural design and specification.

Appendix A - Site Plan

APPENDIX 1

51 GLOUCESTER CRESCENT, NW1 SITE PLAN



Appendix B - Structural Drawings

Appendix C – Structural Calculations

SINCLAIR JOHNSTON & PARTNERS LIMITED

Date	Job No.	Sheet No.
Eng.	8761	1
Review		

Project: 51 Gloucester Crescent

Load Take-down

△ Existing Front, Rear & side wall thickness:

380mm (100mm x 2 cavity wall).

height of building: 7m

7m x 4.5kNm² = 31.5kN/m (DL)

△ Internal walls: 215mm wall: 4.6kNm²
400mm wall: 7.1kNm²

△ New RC wall: 0.4m (thk) x 25kNm³ x 3.3m (h)

= 33kNm. (DL)

△ Existing Roof: 1.5kNm² x 3.4m = 5.1kNm.
(DL + LL)

△ Existing 1st Floor timber floor:

DL: 0.5kNm² LL = 1.5kNm² (Domestic)

4.5m x 2kNm² = 9kNm

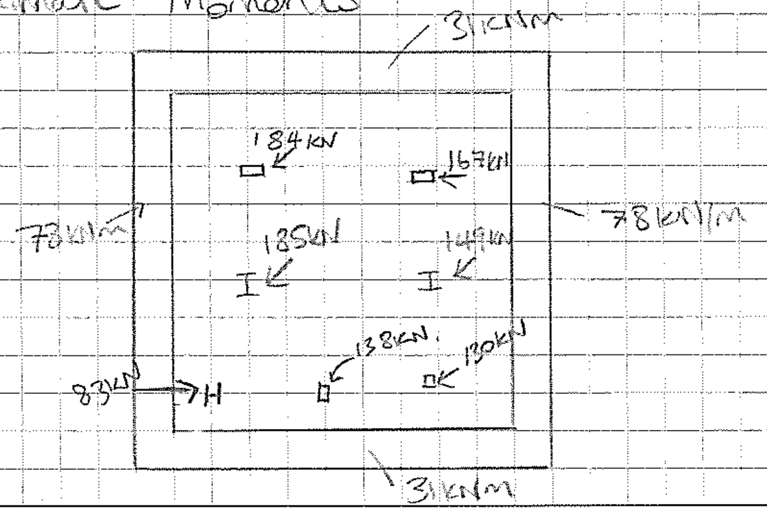
△ Proposed new Ground Floor - 150thk RC slab

on profiled metal decking. DL: (24kNm³ x 0.15) + 1.2 + 0.15 = 5kNm²

DL = 5kNm²

LL = 1.5kNm²

Approximate Moments:



SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date	Job No.	Sheet No.
Eng.	8761	2
Review		
Project 51 Gloucester Crescent.		

Total load: 400 + 800 + 720 + 580

= 3400kN.

Total check

Roof: 1.5kN/m² × 9.7 × 7.2 = 104 kN

Side walls: 4.5kN/m² × 7m × 7.2m = 227 × 2 = 453kN.

front & back: walls 4.5 kN/m² × 5.7m × 9.6
= 246kN × 2 = 492 kN

Internal walls: 215 wall 4.6kN/m² × 5m × 5.7 = 131 kN
400 wall 7.1kN/m² × 5m × 5.7
= 202kN

First floor:
2kN/m² × 9.6 × 7.2 = 138kN

Ground floor slab: = 6.5kN/m² × 9.6 × 7.2 = 450kN

basement walls: 33kN/m × 10 = 330kN × 2 = 660kN

+ 27kN/m × 9.6 = 260 kN × 2 = 520kN.

Total = 3150kN

SJ 2010

SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date	Job No.	Sheet No.
Eng.	8761	3
Review		
Project: 51 Gloucester Crescent		

Steel beam checks under retained masonry walls & new rc slab Beam 1.

Beam under 400thk internal wall:

Length: 3034mm (l)

Wall over. 7.1 kN/m² x 5.7m (h) = 40.5 kN/m (DL)

Slab: 6.5 kN/m² (DL+LL) x 2.2m = 14.3 kN/m

1st Floor: 2 kN/m² (DL+LL) x 3.11m = 6.22 kN/m

Roof 1.5 kN/m² (DL+LL) x 2.2m = 3.3 kN/m

203 x 203 x 86 UC ok:

Max BM: 103 kNm < Capacity 258 kNm

Max SF: 136 kN < Capacity 469 kNm

Max δ: 3.6mm < Capacity 12.1mm

See Tedds Output.

Beam 2 under new suspended RC slab on profiled metal decking:

length: 4078mm

Slab over: 6.5 kN/m² (DL+LL) x 2.3m (max length)

= 15 kN/m.

See Tedds output. 203 x 203 UC 46:

Max BM: 44.5 kNm < Capacity 136 kNm

Max SF 43.6 kN < Capacity 269 kN

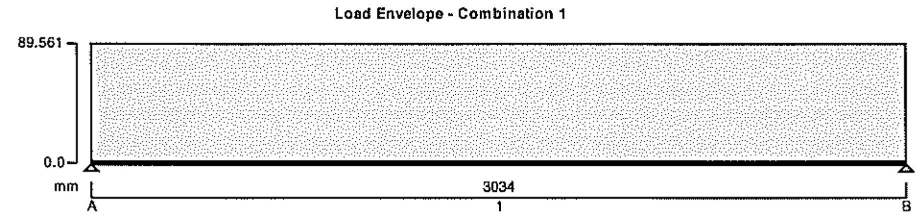
Max δ: 6 mm < Capacity 16.3mm

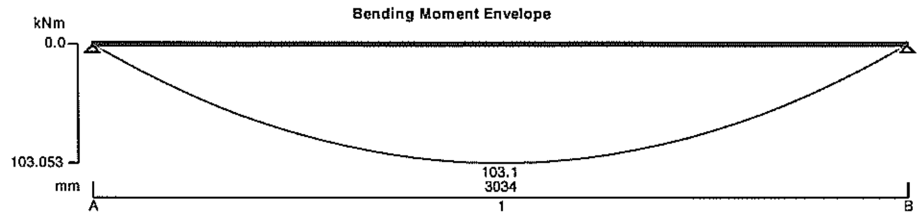
SJ 2010

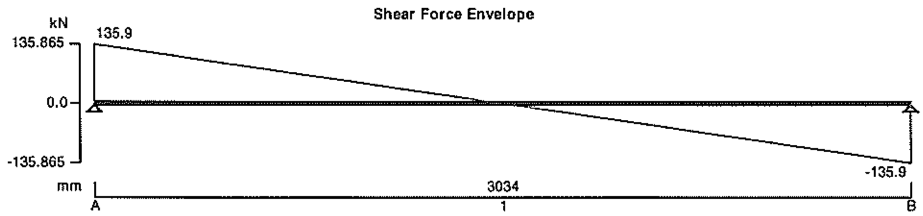
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.13







Support conditions

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

Applied loading

Beam loads	Permanent full UDL 40.5 kN/m
	Permanent self weight of beam × 1
	Permanent full UDL 11 kN/m
	Permanent full UDL 1.7 kN/m
	Variable full UDL 1.7 kN/m
	Variable full UDL 3.3 kN/m
	Variable full UDL 4.7 kN/m
	Permanent full UDL 1.52 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50

	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

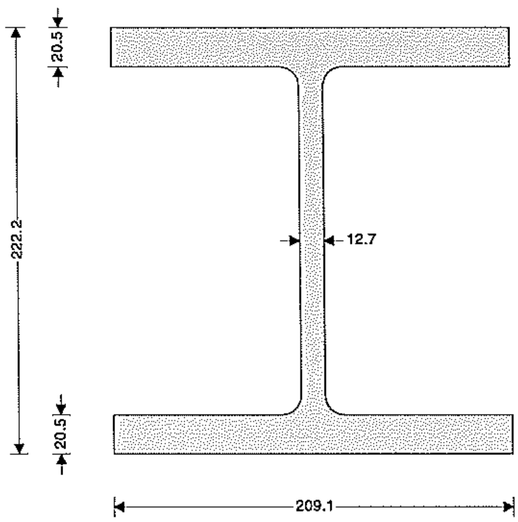
Maximum moment	M_{max} = **103.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **135.9** kN	V_{min} = **-135.9** kN
Deflection	δ_{max} = **3.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **135.9** kN	R_{A_min} = **135.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **84.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **14.7** kN	
Maximum reaction at support B	R_{B_max} = **135.9** kN	R_{B_min} = **135.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **84.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **14.7** kN	

Section details

Section type	**UKC 203x203x86 (Tata Steel Advance)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **20.5** mm
Nominal yield strength	f_y = **265** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 N/mm^2 / f_y]}$ = **0.94**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **160.8** mm

$c / t_w = 13.4 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 4.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = max(abs(V_{max}), abs(V_{min}))$ = **135.9** kN

Shear area - cl 6.2.6(3) $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3069** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **469.6** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **103.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **258.8** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250$ = **12.1** mm

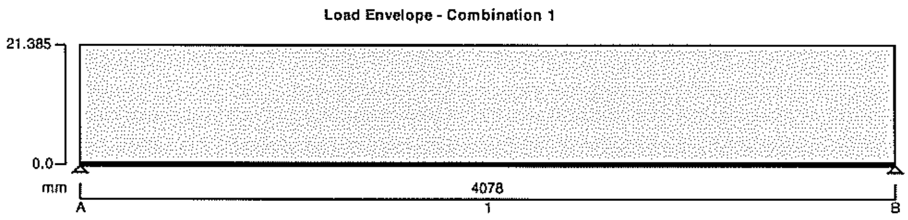
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **3.629** mm

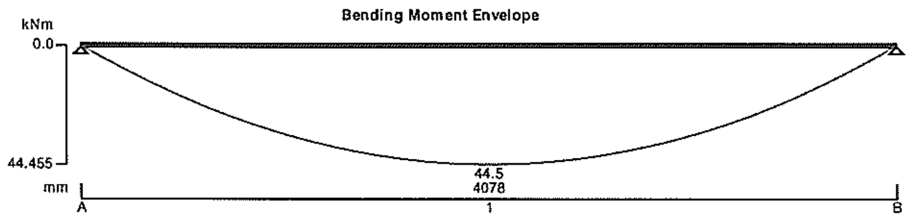
PASS - Maximum deflection does not exceed deflection limit

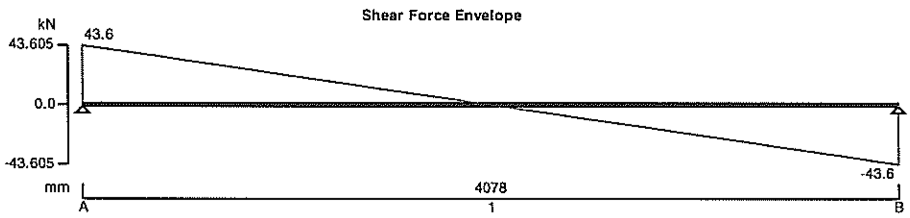
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.13







Support conditions

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

Applied loading

Beam loads	Permanent full UDL 11.5 kN/m
	Variable full UDL 3.5 kN/m
	Permanent self weight of beam × 1

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment M_{max} = **44.5** kNm M_{min} = **0** kNm

Maximum shear V_{max} = **43.6** kN V_{min} = **-43.6** kN

Deflection δ_{max} = **5.8** mm δ_{min} = **0** mm

Maximum reaction at support A R_{A_max} = **43.6** kN R_{A_min} = **43.6** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **24.4** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **7.1** kN

Maximum reaction at support B R_{B_max} = **43.6** kN R_{B_min} = **43.6** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **24.4** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **7.1** kN

Section details

Section type **UC 203x203x46 (BS4-1)**

Steel grade **S275**

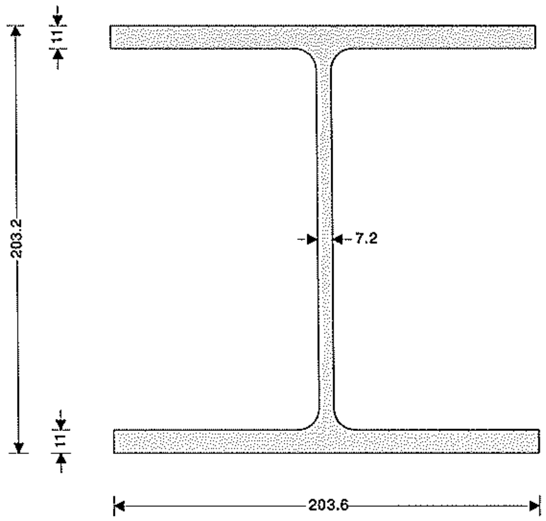
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = max(t_f, t_w)$ = **11.0** mm

Nominal yield strength f_y = **275** N/mm²

Nominal ultimate tensile strength f_u = **410** N/mm²

Modulus of elasticity E = **210000** N/mm²



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d = \mathbf{160.8}$ mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{181.2}$ mm

Shear area factor $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = max(abs(V_{max}), abs(V_{min})) = \mathbf{43.6}$ kN

Shear area - cl 6.2.6(3) $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1698}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{269.5}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min})) = \mathbf{44.5}$ kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{136.8}$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 250 = \mathbf{16.3}$ mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{5.801}$ mm

PASS - Maximum deflection does not exceed deflection limit

SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date	Job No.	Sheet No.
Eng.	8761	4
Review		
Project: 51 Gloucester Crescent		

Total bearing pressure on new raft slab:

Total moments acting on slab: 3150kN.

Weight of slab: 25kNm³ × 94m² × 0.4m = 940kN (DL)

LL: 1.5 × 94m² = 141 kN

+ Masonry walls at basement level:

4.5m (L) × 4.5kNm² × 3.86m(h) = 21.87kN

Total: 3150 + 940 + 22 kN + 141 = 4253kN

Total Area: 94.06m²

Bearing pressure. 4253/94.06 = 45kNm²
< 140kNm²: ok

Worst case span between columns: 4.08m

2-way spanning slab - check for uplift.

Permanent action: DL: G_k = 40kNm²

Variable action: LL Q_k = 5.1kNm²

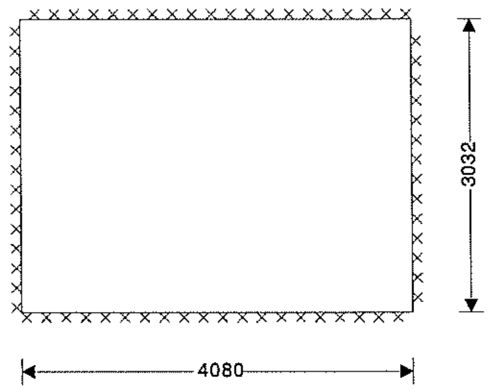
See Tedds output - 400thk slab ok.

SJ 2010

RC SLAB DESIGN

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

Tedds calculation version 1.0.11



Slab definition

Slab reference name	**Basement**
Type of slab	**Two way spanning with restrained edges**
Overall slab depth	$h = \mathbf{400}$ mm
Shorter effective span of panel	$l_x = \mathbf{3032}$ mm
Longer effective span of panel	$l_y = \mathbf{4080}$ 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	$G_k = \mathbf{40.0}$ kN/m^2
Characteristic variable action	$Q_k = \mathbf{5.1}$ kN/m^2
Partial factor for permanent action	$\gamma_G = \mathbf{1.35}$
Partial factor for variable action	$\gamma_Q = \mathbf{1.50}$
Quasi-permanent value of variable action	$\psi_2 = \mathbf{0.30}$
Design ultimate load	$q = \gamma_G \times G_k + \gamma_Q \times Q_k = \mathbf{61.7}$ kN/m^2
Quasi-permanent load	$q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k = \mathbf{41.5}$ kN/m^2

Concrete properties

Concrete strength class	C32/40
Characteristic cylinder strength	$f_{ck} = \mathbf{32}$ N/mm^2
Partial factor (Table 2.1N)	$\gamma_C = \mathbf{1.50}$
Compressive strength factor (cl. 3.1.6)	$\alpha_{cc} = \mathbf{0.85}$
Design compressive strength (cl. 3.1.6)	$f_{cd} = \mathbf{18.1}$ N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{3.0}$ N/mm^2
Maximum aggregate size	$d_g = \mathbf{20}$ mm

Reinforcement properties

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

Concrete cover to reinforcement

Nominal cover to outer top reinforcement c_{nom_t} = **50** mm
Nominal cover to outer bottom reinforcement c_{nom_b} = **75** 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} = **32** mm
Min. btm cover requirement with regard to bond c_{min,b_b} = **32** mm
Reinforcement fabrication **Not subject to QA system**
Cover allowance for deviation Δc_{dev} = **10** mm
Min. required nominal cover to top reinft $c_{nom\_t\_min}$ = **42.0** mm
Min. required nominal cover to bottom reinft $c_{nom\_b\_min}$ = **42.0** 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} = **0.0359**
Design bending moment $M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2$ = **20.4** kNm/m
Reinforcement provided 32 mm dia. bars at 200 mm centres
Area provided A_{sx_p} = 4021 mm²/m
Effective depth to tension reinforcement $d_{x\_p} = h - c_{nom\_b} - \phi_{y\_p} - \phi_{x\_p} / 2$ = **277.0** mm
K factor $K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck})$ = **0.008**
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 $z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + (1 - 3.53 \times K)^{0.5}))$ = **263.1** mm
Area of reinforcement required for bending $A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z)$ = **178** mm²/m
Minimum area of reinforcement required $A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p})$ = **436** mm²/m
Area of reinforcement required $A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min})$ = **436** mm²/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q$ = **13.0** N/mm²
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 $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2$ = **13.6** kNm/m
Reinforcement provided 32 mm dia. bars at 200 mm centres
Area provided A_{sy_p} = 4021 mm²/m
Effective depth to tension reinforcement $d_{y\_p} = h - c_{nom\_b} - \phi_{y\_p} / 2$ = **309.0** mm
K factor $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck})$ = **0.004**
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 $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + (1 - 3.53 \times K)^{0.5}))$ = **293.5** mm

Area of reinforcement required for bending $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z) =$ **107** mm^2/m

Minimum area of reinforcement required $A_{sy\_p\_min} = max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p}) =$ **486** mm^2/m

Area of reinforcement required $A_{sy\_p\_req} = max(A_{sy\_p\_m}, A_{sy\_p\_min}) =$ **486** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q =$ **7.8** 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 $\beta_{sx\_n} =$ **0.0478**

Design bending moment $M_{x\_n} = \beta_{sx\_n} \times q \times l_x^2 =$ **27.1** kNm/m

Reinforcement provided 32 mm dia. bars at 200 mm centres

Area provided $A_{sx\_n} = 4021$ mm^2/m

Effective depth to tension reinforcement $d_{x\_n} = h - c_{nom\_t} - \phi_{x\_n} / 2 =$ **334.0** mm

K factor $K = M_{x\_n} / (b \times d_{x\_n}^2 \times f_{ck}) =$ **0.008**

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 $z = min(0.95 \times d_{x\_n}, d_{x\_n}/2 \times (1 + (1 - 3.53 \times K)^{0.5})) =$ **317.3** mm

Area of reinforcement required for bending $A_{sx\_n\_m} = M_{x\_n} / (f_{yd} \times z) =$ **196** mm^2/m

Minimum area of reinforcement required $A_{sx\_n\_min} = max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_n}, 0.0013 \times b \times d_{x\_n}) =$ **525** mm^2/m

Area of reinforcement required $A_{sx\_n\_req} = max(A_{sx\_n\_m}, A_{sx\_n\_min}) =$ **525** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sx\_n} = (f_{yk} / \gamma_S) \times min((A_{sx\_n\_m}/A_{sx\_n}), 1.0) \times q_{SLS} / q =$ **14.3** 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 $\beta_{sy\_n} =$ **0.0320**

Design bending moment $M_{y\_n} = \beta_{sy\_n} \times q \times l_x^2 =$ **18.1** kNm/m

Reinforcement provided 32 mm dia. bars at 200 mm centres

Area provided $A_{sy\_n} = 4021$ mm^2/m

Effective depth to tension reinforcement $d_{y\_n} = h - c_{nom\_t} - \phi_{x\_n} - \phi_{y\_n} / 2 =$ **302.0** mm

K factor $K = M_{y\_n} / (b \times d_{y\_n}^2 \times f_{ck}) =$ **0.006**

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 $z = min(0.95 \times d_{y\_n}, d_{y\_n}/2 \times (1 + (1 - 3.53 \times K)^{0.5})) =$ **286.9** mm

Area of reinforcement required for bending $A_{sy\_n\_m} = M_{y\_n} / (f_{yd} \times z) =$ **145** mm^2/m

Minimum area of reinforcement required $A_{sy\_n\_min} = max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_n}, 0.0013 \times b \times d_{y\_n}) =$ **475** mm^2/m

Area of reinforcement required $A_{sy\_n\_req} = max(A_{sy\_n\_m}, A_{sy\_n\_min}) =$ **475** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy\_n} = (f_{yk} / \gamma_S) \times \min((A_{sy\_n\_m}/A_{sy\_n}), 1.0) \times q_{SLS} / q$ = **10.6** N/mm²

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 $V_{x\_n} = q \times l_x / 2$ = **93.5** kN/m

Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200 \text{ mm} / d_{x\_n})^{0.5})$ = **1.774**

Reinforcement ratio $\rho_l = \min(0.02, A_{sx\_n} / (b \times d_{x\_n}))$ = **0.0120**

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} = **156.2** kN/m

Shear resistance (Exp. 6.2a) $V_{Rd,c\_x\_n} = \max(V_{Rd,c\_min}, (0.18 \text{ N/mm}^2 / \gamma_C) \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{x\_n})$

V_{Rd,c_x_n} = **239.8** kN/m

PASS - Shear capacity is adequate

Shear capacity check at long span continuous support

Shear force $V_{y\_n} = q \times l_x / 2$ = **93.5** kN/m

Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200 \text{ mm} / d_{y\_n})^{0.5})$ = **1.814**

Reinforcement ratio $\rho_l = \min(0.02, A_{sy\_n} / (b \times d_{y\_n}))$ = **0.0133**

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} = **146.1** kN/m

Shear resistance (Exp. 6.2a) $V_{Rd,c\_y\_n} = \max(V_{Rd,c\_min}, (0.18 \text{ N/mm}^2 / \gamma_C) \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{y\_n})$

V_{Rd,c_y_n} = **229.3** 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_δ = **1.5**

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

(Exp. 7.16) $ratio_{lim\_x\_bas}$ = **50.21**

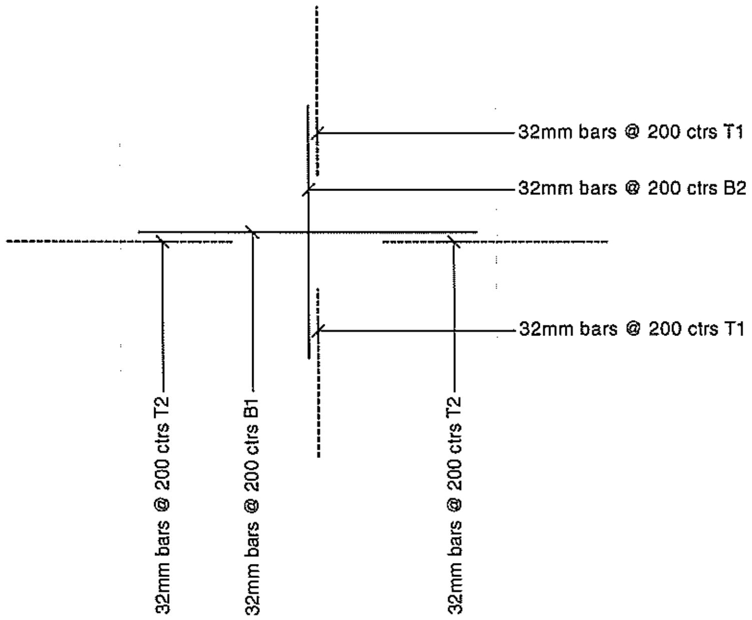
Mod span-to-depth ratio limit $ratio_{lim\_x} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2/f_{yk}) \times (A_{sx\_p}/A_{sx\_p\_m})) \times ratio_{lim\_x\_bas})$ = **60.00**

Actual span-to-eff. depth ratio $ratio_{act\_x} = l_x / d_{x\_p}$ = **10.95**

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

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.



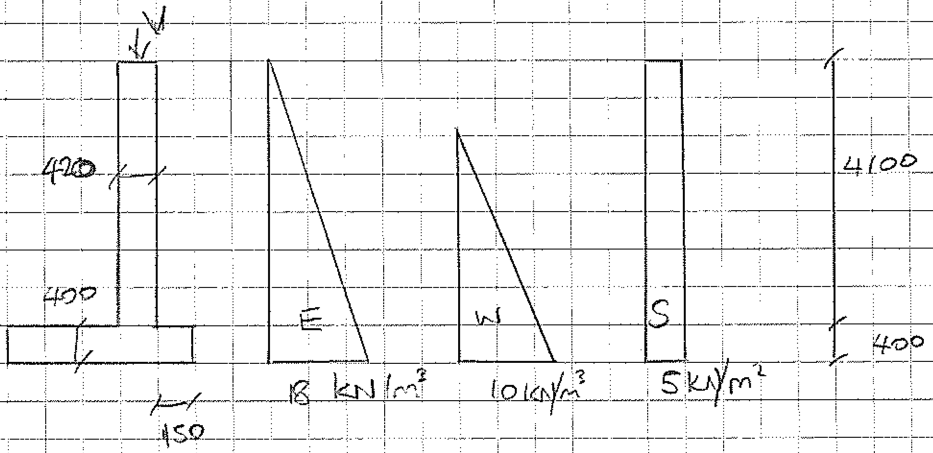
32mm bars @ 200 ctrs T1
32mm bars @ 200 ctrs B2
32mm bars @ 200 ctrs T1
32mm bars @ 200 ctrs T2
32mm bars @ 200 ctrs B1
32mm bars @ 200 ctrs T2

SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date	Job No.	Sheet No.
Eng.	8761	5
Review		

Project
51 Gloucester Crescent

New Basement underpin wall. (sides)
Element A.
Retained height: 4.1m, propped at base.
Load on wall:



V = wall weight (solid brick)
= 78kN/m. (SLS)

→ see Tedds output overleaf.

(420thk r.c. wall & 400thk base OK)

SJ 2010

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.6.05

Retaining wall details

Stem type	Cantilever		
Stem height	h_{stem} = **4100** mm		
Stem thickness	t_{stem} = **420** mm		
Angle to rear face of stem	α = **90** deg		
Stem density	γ_{stem} = **25** kN/m^3		
Toe length	l_{toe} = **2500** mm		
Heel length	l_{heel} = **150** mm		
Base thickness	t_{base} = **400** mm		
Base density	γ_{base} = **25** kN/m^3		
Height of retained soil	h_{ret} = **4100** mm	Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm		
Height of water	h_{water} = **4100** mm		
Water density	γ_w = **9.8** kN/m^3		

Retained soil properties

Soil type	Medium dense well graded sand	
Moist density	γ_{mr} = **18** kN/m^3	
Saturated density	γ_{sr} = **23** kN/m^3	
Characteristic effective shear resistance angle		$\phi'_{r.k}$ = **23** deg
Characteristic wall friction angle		$\delta_{r.k}$ = **0** deg

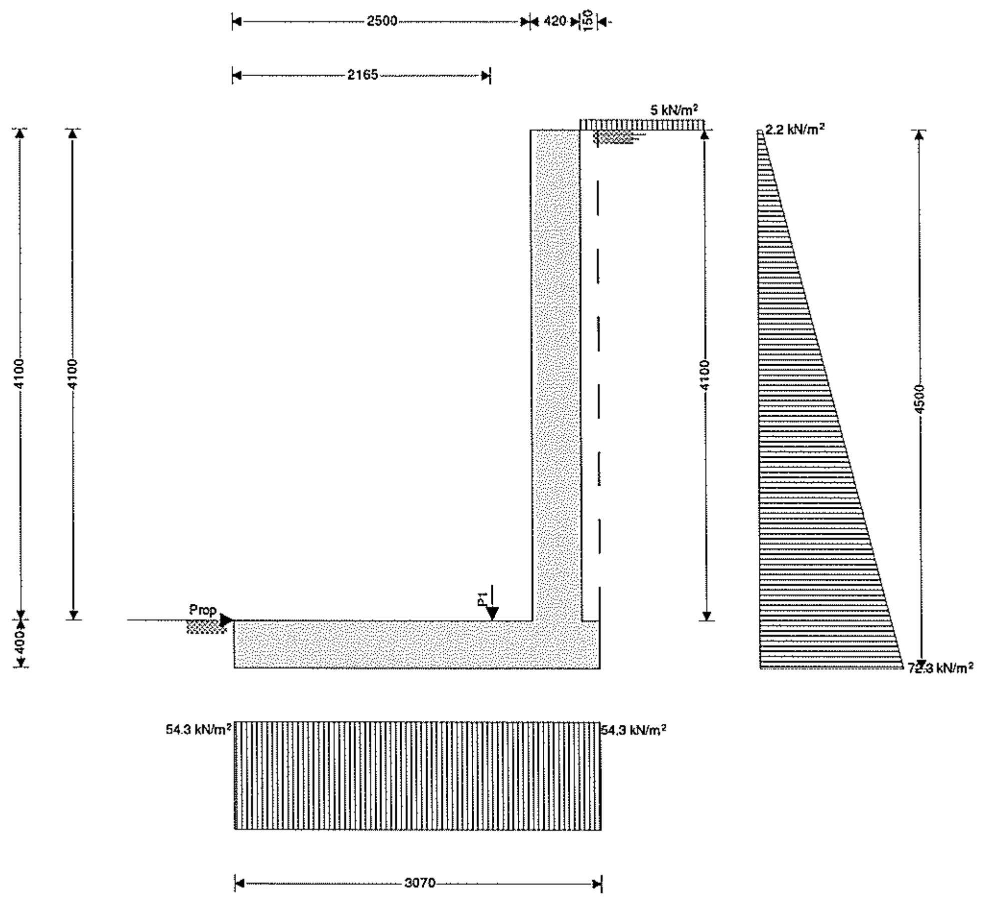
Base soil properties

Soil type	Medium dense well graded sand	
Soil density	γ_b = **18** kN/m^3	
Characteristic effective shear resistance angle		$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle		$\delta_{b.k}$ = **18** deg
Characteristic base friction angle		$\delta_{bb.k}$ = **21** deg
Presumed bearing capacity	$P_{bearing}$ = **140** kN/m^2	

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2165 mm	P_{G1} = **78** kN/m

Project				Job no.	
51 Gloucester Crescent				8761	
Calcs for: Element A				Start page no./Revision: 2	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
SH	08/05/2017				



Calculate retaining wall geometry

Base length	l_{base} = **3070** mm		
Saturated soil height	h_{sat} = **4100** mm		
Moist soil height	h_{moist} = **0** mm		
Length of surcharge load	l_{sur} = **150** mm		
Vertical distance	x_{sur_v} = **2995** mm		
Effective height of wall	h_{eff} = **4500** mm		
Horizontal distance	x_{sur_h} = **2250** mm		
Area of wall stem	A_{stem} = **1.722** m²	Vertical distance	x_{stem} = **2710** mm
Area of wall base	A_{base} = **1.228** m²	Vertical distance	x_{base} = **1535** mm
Area of saturated soil	A_{sat} = **0.615** m²	Vertical distance	x_{sat_v} = **2995** mm
		Horizontal distance	x_{sat_h} = **1500** mm
Area of water	A_{water} = **0.615** m²	Vertical distance	x_{water_v} = **2995** mm
		Horizontal distance	x_{water_h} = **1500** mm

Using Coulomb theory

Active pressure coefficient K_A = **0.438** Passive pressure coefficient K_P = **5.609**

Bearing pressure check

Vertical forces on wall

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} + F_{P\_v}$ = **166.6** kN/m

Horizontal forces on wall

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **160** kN/m

Moments on wall

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **118.3** kNm/m

Check bearing pressure

Propping force F_{prop_base} = **160** kN/m

Bearing pressure at toe q_{toe} = **54.3** kN/m^2 Bearing pressure at heel q_{heel} = **54.3** kN/m^2

Factor of safety FoS_{bp} = **2.579**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.05

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

Concrete strength class C32/40

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

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

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

Reinforcement details

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

Design yield strength f_{yd} = **435** N/mm^2 Partial factor γ_S = **1.15**

Cover to reinforcement

Front face of stem c_{sf} = **40** mm Rear face of stem c_{sr} = **50** mm

Top face of base c_{bt} = **50** mm Bottom face of base c_{bb} = **75** mm

Check stem design at base of stem

Depth of section h = **420** mm

Rectangular section in flexure - Section 6.1

Design bending moment M = **269.3** kNm/m K = **0.065** K' = **0.207**

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{sr.req}$ = **1833** mm^2/m

Tens.reinforcement provided 20 dia.bars @ 100 c/c Tens.reinforcement provided $A_{sr.prov}$ = **3142** mm^2/m

Min.area of reinforcement $A_{sr.min}$ = **566** mm^2/m Max.area of reinforcement $A_{sr.max}$ = **16800** mm^2/m

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

Deflection control - Section 7.4

Limiting span to depth ratio 12.7 Actual span to depth ratio 11.4

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

Crack control - Section 7.3

Limiting crack width w_{max} = **0.3** mm Maximum crack width w_k = **0.181** mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = **190.3** kN/m Design shear resistance $V_{Rd.c}$ = **228.8** kN/m

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Min.area of reinforcement $A_{sx.req}$ = **785** mm^2/m Max.spacing of reinforcement s_{sx_max} = **400** mm

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

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

Check base design at toe

Depth of section h = 400 mm

Rectangular section in flexure - Section 6.1

Design bending moment M = 160.8 kNm/m K = 0.050 K' = 0.207

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bb.req}$ = 1228 mm²/m

Tens.reinforcement provided 16 dia.bars @ 100 c/c Tens.reinforcement provided $A_{bb.prov}$ = 2011 mm²/m

Min.area of reinforcement $A_{bb.min}$ = 498 mm²/m Max.area of reinforcement $A_{bb.max}$ = 16000 mm²/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = 0.3 mm Maximum crack width w_k = 0.232 mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = 71.5 kN/m Design shear resistance $V_{Rd.c}$ = 186.2 kN/m

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure - Section 6.1

Design bending moment M = 0.8 kNm/m K = 0.000 K' = 0.207

K' > K - No compression reinforcement is required

Tens.reinforcement required $A_{bt.req}$ = 6 mm²/m

Tens.reinforcement provided 16 dia.bars @ 200 c/c Tens.reinforcement provided $A_{bt.prov}$ = 1005 mm²/m

Min.area of reinforcement $A_{bt.min}$ = 538 mm²/m Max.area of reinforcement $A_{bt.max}$ = 16000 mm²/m

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

Crack control - Section 7.3

Limiting crack width w_{max} = 0.3 mm Maximum crack width w_k = 0.003 mm

PASS - Maximum crack width is less than limiting crack width**Rectangular section in shear - Section 6.2**

Design shear force V = 11.2 kN/m Design shear resistance $V_{Rd.c}$ = 158.7 kN/m

PASS - Design shear resistance exceeds design shear force

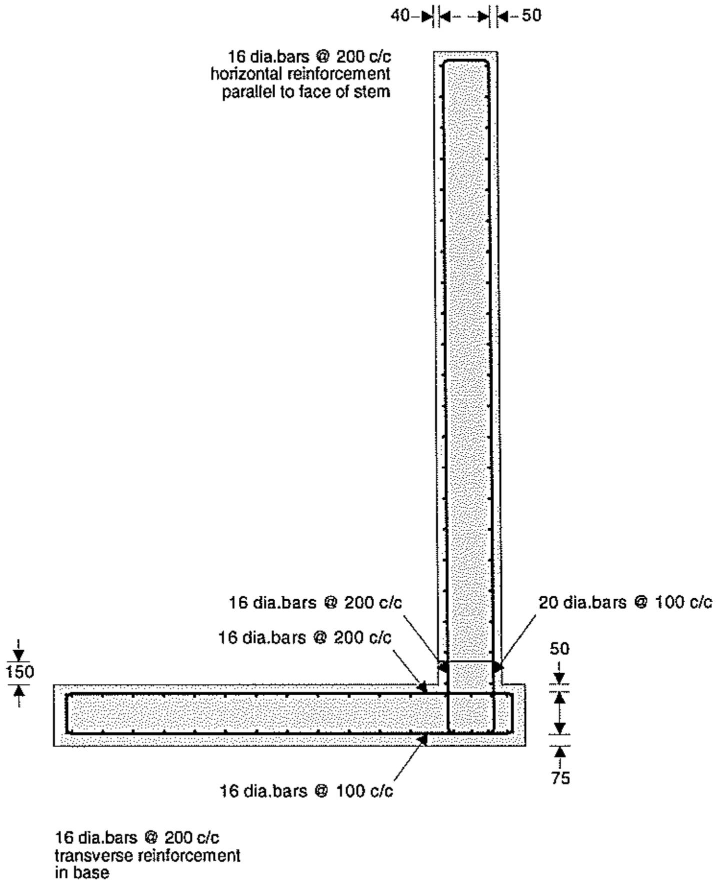
Secondary transverse reinforcement to base - Section 9.3

Min.area of reinforcement $A_{bx.req}$ = 402 mm²/m Max.spacing of reinforcement s_{bx_max} = 450 mm

Trans.reinforcement provided 16 dia.bars @ 200 c/c Trans.reinforcement provided $A_{bx.prov}$ = 1005 mm²/m

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

TEKLA Tedds	Project: 51 Gloucester Crescent			Job no.: 8761	
	Calcs for: Element A			Start page no./Revision: 5	
	Calcs by: SH	Calcs date: 08/05/2017	Checked by	Checked date	Approved by / Approved date



SINCLAIR JOHNSTON
& PARTNERS LIMITED

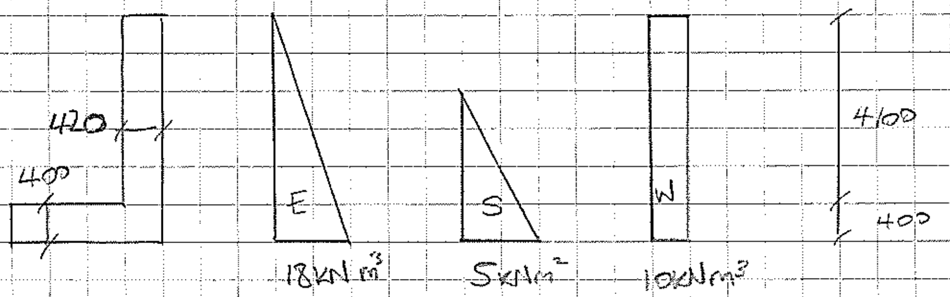
Date	Job No.	Sheet No.
Eng.	8761	6
Review		

Project: 51 Gloucester Crescent

New Basement front & rear walls - Element B

Retained height: 4.1m, propped at base, no vertical load

Load on wall:



→ See Todd's output over leaf.

(420Thk wall & 400 thick rc base - ok)

SJ 2010

TEKLA Tedds	Project: 51 Gloucester Crescent				Job no.: 8761	
	Calcs for: Element B				Start page no./Revision: 1	
	Calcs by: SH	Calcs date: 08/05/2017	Checked by	Checked date	Approved by	Approved date

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.6.05

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **4100** mm
Stem thickness	t_{stem} = **420** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **4500** mm
Heel length	l_{heel} = **150** mm
Base thickness	t_{base} = **400** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **4100** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **4100** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

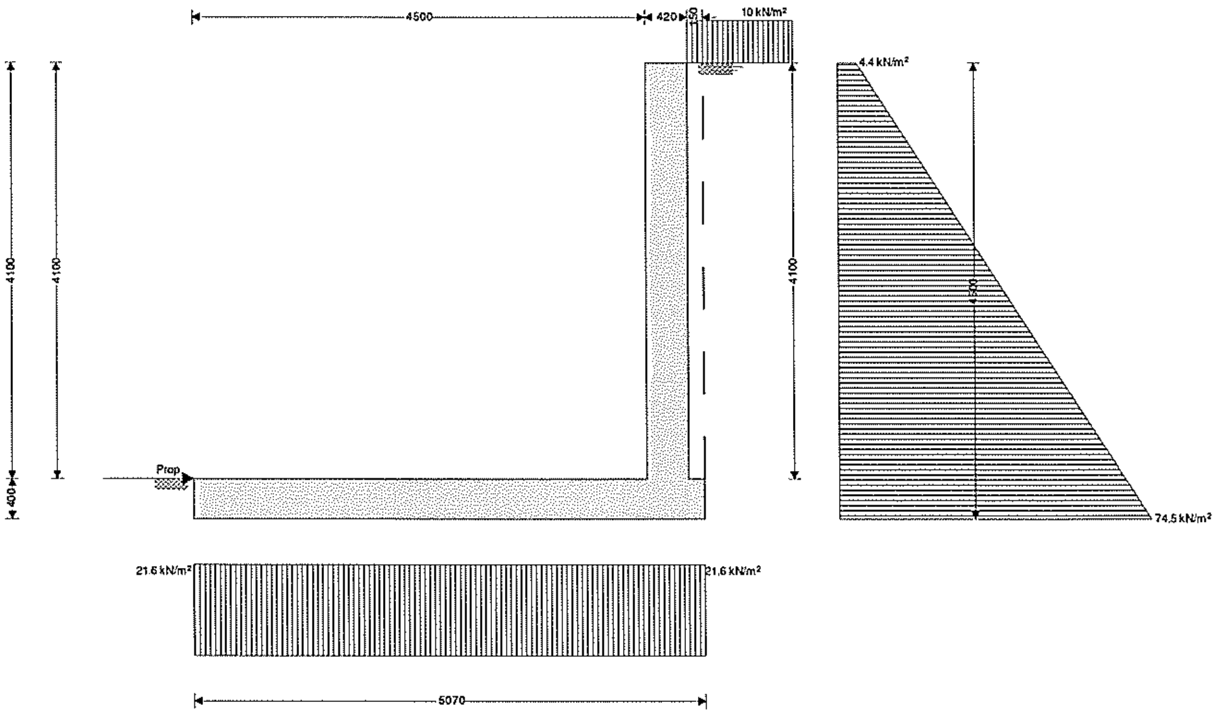
Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **23** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **0** deg

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **21** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **18** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **21** deg
Presumed bearing capacity	$P_{bearing}$ = **140** kN/m^2

Loading details

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



Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel}$ = **5070** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover}$ = **4100** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water}$ = **0** mm
Length of surcharge load	$l_{sur} = l_{heel}$ = **150** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **4995** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **4500** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **2250** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **1.722** m²
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **4710** mm
Area of wall base	$A_{base} = l_{base} \times t_{base}$ = **2.028** m²
- Distance to vertical component	$x_{base} = l_{base} / 2$ = **2535** mm
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel}$ = **0.615** m²
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat}$ = **4995** mm
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3$ = **1500** mm
Area of water	$A_{water} = h_{sat} \times l_{heel}$ = **0.615** m²
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat}$ = **4995** mm
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3$ = **1500** mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2)$ = **0.438**
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2)$ = **3.482**

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem}$ = **43** kN/m

Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{50.7}$ kN/m
Surcharge load	$F_{sur\_v} = Surcharge_Q \times l_{heel} = \mathbf{1.5}$ kN/m
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr}' - \gamma_w') = \mathbf{8.1}$ kN/m
Water	$F_{water\_v} = A_{water} \times \gamma_w' = \mathbf{6}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{water\_v} + F_{sur\_v} = \mathbf{109.4}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_Q \times h_{eff} = \mathbf{19.7}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{58.5}$ kN/m
Water	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{99.3}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-4.8}$ kN/m
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{172.8}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{202.8}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{128.5}$ kNm/m
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-36.9}$ kNm/m
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{-47.2}$ kNm/m
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{-118.9}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} = \mathbf{128.3}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{172.8}$ kN/m
Distance to reaction	$\bar{x} = l_{base} / 2 = \mathbf{2535}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{5070}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} = \mathbf{21.6}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} = \mathbf{21.6}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{6.488}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.6.05

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

Concrete strength class	C32/40
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{32}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{40}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 $= \mathbf{40}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{3.0}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{33346}$ 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	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1}$ N/mm^2

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

Reinforcement details

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

Modulus of elasticity of reinforcement — E_s = **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N — γ_S = **1.15**

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

Cover to reinforcement

Front face of stem — c_{sf} = **40** mm

Rear face of stem — c_{sr} = **50** mm

Top face of base — c_{bt} = **50** mm

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

Check stem design at base of stem

Depth of section — h = **420** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **297** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **360** mm

$K = M / (d^2 \times f_{ck})$ = **0.072**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **336** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **61** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **2035** mm^2/m

Tension reinforcement provided — 20 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **3142** mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **566** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **16800** mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.648**

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

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.006**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d$ = **0.006**

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2$ = **0.000**

Structural system factor - Table 7.4N — K_b = **0.4**

Reinforcement factor - exp.7.17 — $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **11.7**

Actual span to depth ratio — h_{stem} / d = **11.4**

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

Crack control - Section 7.3

Limiting crack width — w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.3**

Serviceability bending moment — M_{sls} = **190.1** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **180.3** N/mm^2

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{119664}$ mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{3.0}$ N/mm²

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.026}$

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{300}$ mm

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

$w_k = \mathbf{0.19}$ mm

$w_k / w_{max} = \mathbf{0.634}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{203.8}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.745}$

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.009}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.457}$ N/mm²

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

$V_{Rd.c} = \mathbf{228.8}$ kN/m

$V / V_{Rd.c} = \mathbf{0.891}$

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) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{785}$ mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) — $s_{sx\_max} = \mathbf{400}$ mm

Transverse reinforcement provided — 20 dia.bars @ 200 c/c

Area of transverse reinforcement provided — $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{1571}$ mm²/m

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

Check base design at toe

Depth of section — $h = \mathbf{400}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{158.7}$ kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{315}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.050}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{299}$ mm

Depth of neutral axis — $x = 2.5 \times (d - z) = \mathbf{39}$ mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{1220}$ mm²/m

Tension reinforcement provided — 20 dia.bars @ 100 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{3142}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{495}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h = \mathbf{16000}$ mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.388}$

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.3**
Serviceability bending moment	M_{sls} = **117.2** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **124.7** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **120208** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.026**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **385** 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 = **0.144** mm
	w_k / w_{max} = **0.48**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **70.5** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.797**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.010**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.477** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **215.4** kN/m
	$V / V_{Rd.c}$ = **0.327**

PASS - Design shear resistance exceeds design shear force

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **1.4** kNm/m
Depth to tension reinforcement	$d = h - c_{bt} - \phi_{bt} / 2$ = **340** mm
	$K = M / (d^2 \times f_{ck})$ = **0.000**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **323** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **43** mm
Area of tension reinforcement required	$A_{bt.req} = M / (f_{yd} \times z)$ = **10** mm^2/m
Tension reinforcement provided	20 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1571** mm^2/m
Minimum area of reinforcement - exp.9.1N	$A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **535** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bt.max} = 0.04 \times h$ = **16000** mm^2/m
	$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.34**

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

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.3**
Serviceability bending moment	M_{sls} = **1** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **1.9** N/mm^2
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **119167** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.013**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **5.998**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **428** 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 = **0.002** mm
	w_k / w_{max} = **0.008**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = 19 kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.767**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bt.prov} / d, 0.02)$ = **0.005**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.465** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **176.9** kN/m
	$V / V_{Rd.c}$ = **0.107**

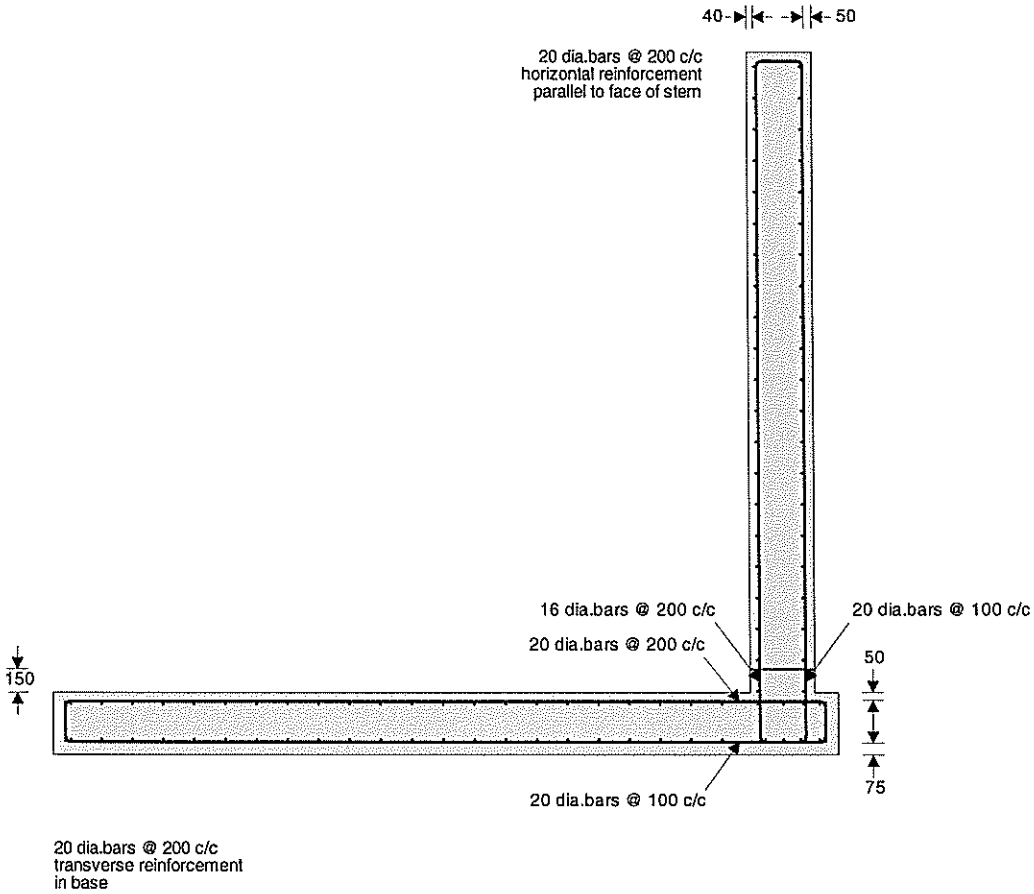
PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **628** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	20 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **1571** mm^2/m

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

Project: 51 Gloucester Crescent				Job no.: 8761	
Calcs for: Element B				Start page no./Revision: 8	
Calcs by: SH	Calcs date: 08/05/2017	Checked by:	Checked date:	Approved by:	Approved date:



Appendix D – Resin Grouting Product Information

Stabila

Product Information

P200 Soil Injection Resin

Aqua-reactive PU Injection Resin

Stabila P200 SOIL is a solvent free, one component aqua-reactive polyurethane injection resin. It is sufficiently fast reacting to cut off active or gushing water flows, even under pressure. Stabila P200 SOIL Accelerator is added to Stabila P200 SOIL to vary the speed of reaction for the application in hand.
When P200 SOIL comes into contact with water, it reacts to produce a rapidly expanding closed cell foam. This rapid expansion acts to close off flow paths and hence arrest movement of free water. The foam is moderately flexible, hydrophobic and chemically resistant. It is harmless to the environment and resists biological attack

Uses

Injecting into cracks and minor open fissures in concrete and masonry structures where water leaks are to be controlled.

Injection into open, granular soils where specialised stabilisation is required.

Advantages

An extremely efficient injection resin for leak sealing use where flowing water is encountered.

Exhibits excellent penetration into voids and porous substrates.

Application

Stabila P200 SOIL injection is by single component pump through injection packers or ports as appropriate.

Speed of reaction is dependent on percentage of accelerator used and water temperature. Cold water will increase reaction times.

Where cold water, or relatively fast water flows occur, higher accelerator dosage is necessary. There is no advantage to be gained by increasing the accelerator dose beyond the recommended maximum.

Stabila P200 SOIL may be used to control fast flowing water or water under pressure, but where such conditions are encountered, please contact our Technical Department before use.

Package & Storage

Stabila P200 SOIL is supplied in 25kg drums. P200 SOIL accelerator is supplied in 2.5kg plastic containers. Store in original containers in a dry area, protect from heat and sunlight. Once opened, use as soon as possible.

Health & Safety

Avoid contact with eyes and skin. Follow advice in separate Health & Safety data sheet.

Technical data

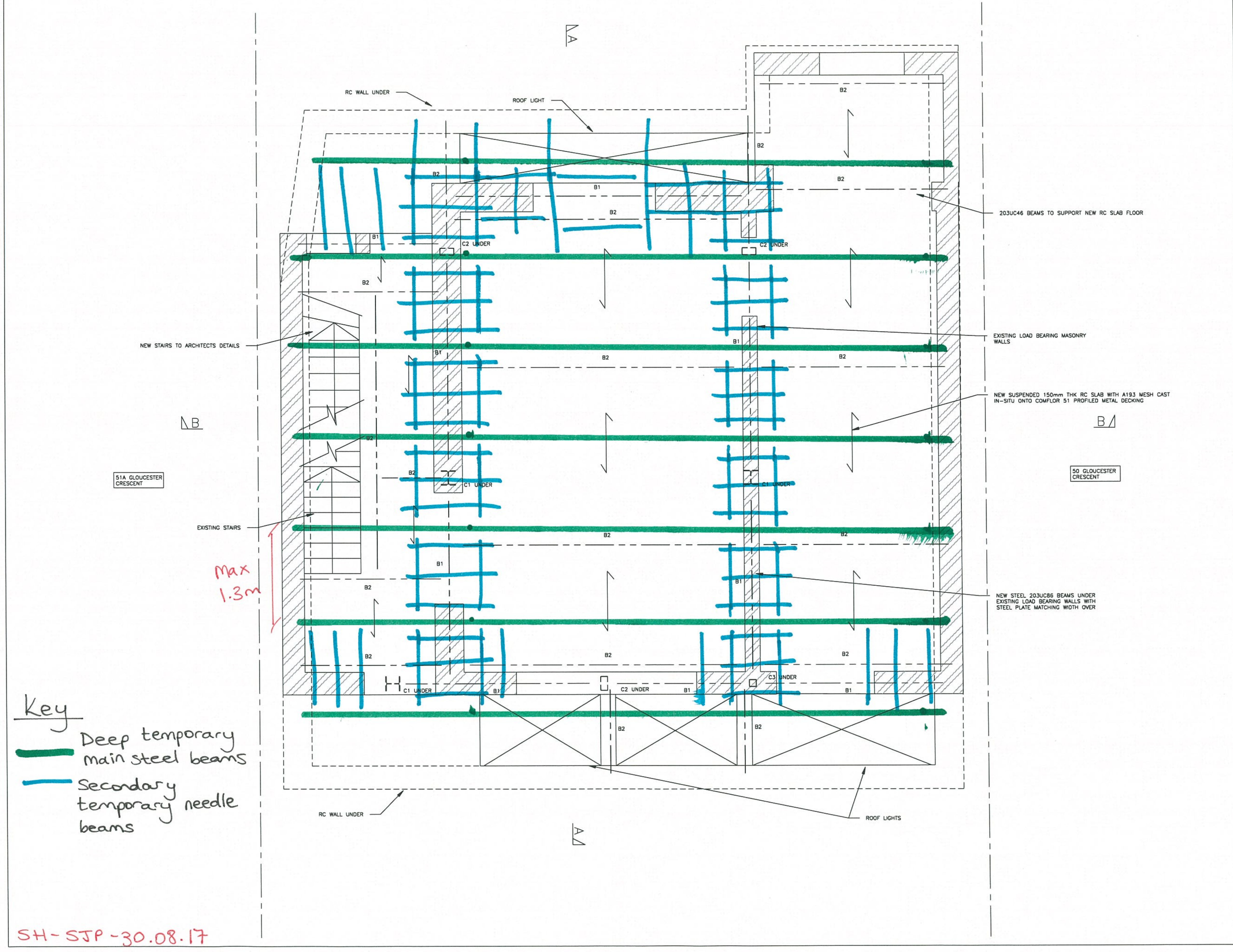
	P200 Soil	P200 Accelerator
Form	Liquid	Liquid
Viscosity (25°C)	190 mPas	9 mPas
Colour	Brown	Pale Yellow
Specific gravity (20°C)	1.10	1.04
Mixing ratio	1% to 10% Accelerator by weight	
Application temp	Not less than 5°C	

Reaction times (15°C)

% Accel dosage	Induction time	Gel time
3% (by weight)	50 sec	8 min 20 secs
6% “	29 sec	2 min 35 secs
9% “	28 sec	2 min 05 secs

Stabila UK. Oxon. OX27 7SR
tel 01869 346010 fax 01869 345455

Appendix E – Temporary Propping Design



NOTES:

1. All structural engineering drawings are to be read with the specification and with all relevant Architect's and Service Engineer's drawings and specifications.
2. Do not scale from this drawing in either paper or digital form. Use written dimensions only. To check drawing has been printed to intended scale this bar should be 50mm long @ A1 or 25mm long @ A3:
3. All dimensions are in millimetres and levels in metres.

KEY:

DENOTES EXISTING SOLID BRICKWORK

DENOTES SPAN OF EXISTING RC SLAB

DENOTES STRUCTURE UNDER

BEAM SCHEDULE	
B1	203x203UC86
B2	203x203UC46

THIS DRAWING IS NOT FOR CONSTRUCTION

Rev	Date	Issued	Amendment
C	18.05.17	SH	For planning
B	09.05.17	SH	For planning
A	05.05.17	SH	For planning
–	24.03.17	SH	For comment

Status PLANNING

SINCLAIRJOHNSTON
Consulting Civil & Structural Engineers

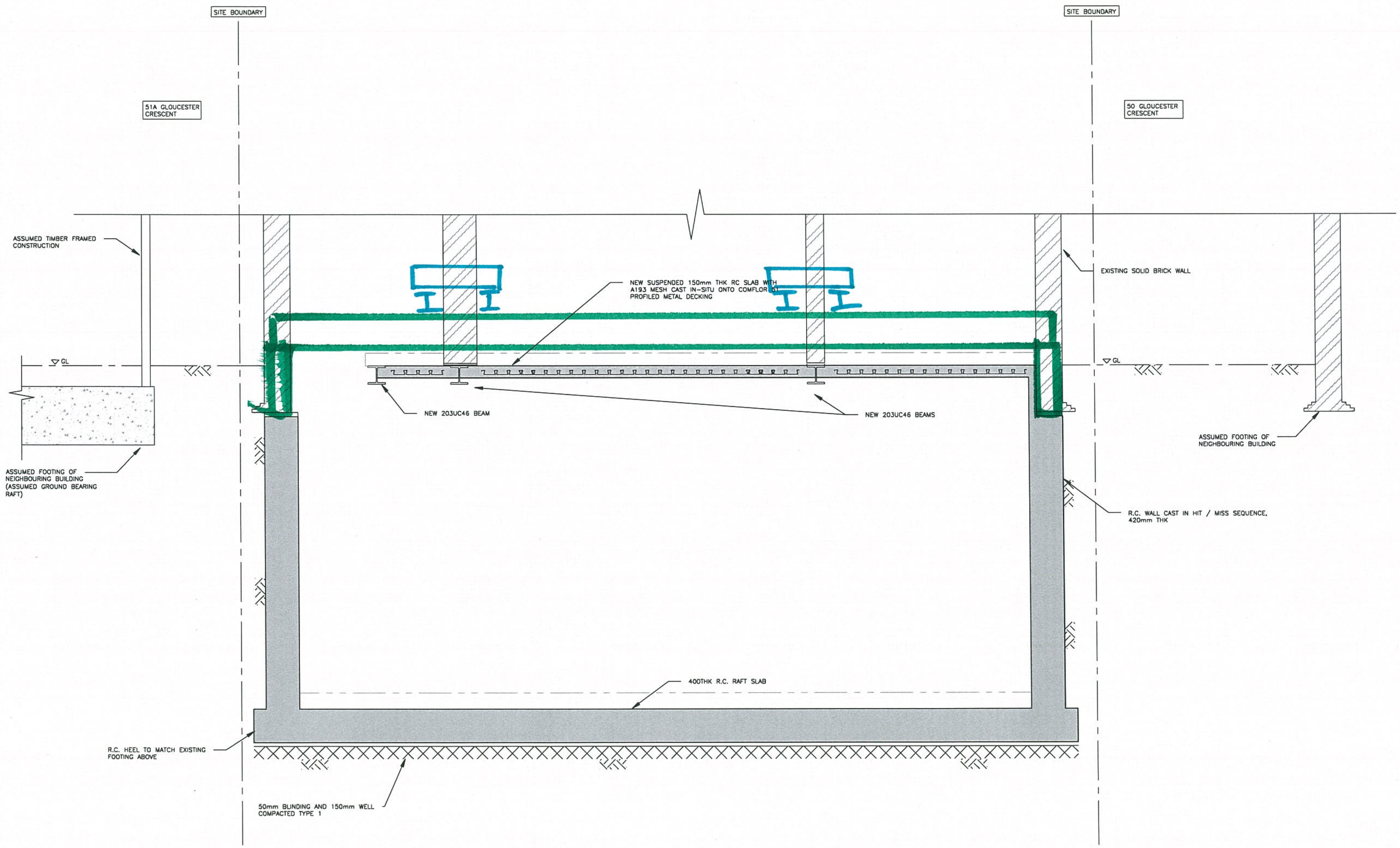
93 Great Suffolk Street
London SE1 0BX
T: 020 7593 1900
F: 020 7593 1910
www.sinclairjohnston.co.uk

51 Gloucester Crescent
NW1

Proposed
Ground Floor

Drawn S Hill Scale 1:25 at A1

Project No./Drawing No. 8761/010T Rev C



NOTES:

1. All structural engineering drawings are to be read with the specification and with all relevant Architect's and Service Engineer's drawings and specifications.
2. Do not scale from this drawing in either paper or digital form. Use written dimensions only. To check drawing has been printed to intended scale this bar should be 50mm long @ A1 or 25mm long @ A3:
3. All dimensions are in millimetres and levels in metres.

KEY:

DENOTES EXISTING SOLID BRICKWORK

THIS DRAWING IS NOT FOR CONSTRUCTION

Rev	Date	Issued	Amendment
C	18.05.17	SH	For planning
B	09.05.17	SH	For planning
A	05.05.17	SH	For planning
–	24.03.17	SH	For comment

Status: PLANNING

SINCLAIR JOHNSTON
Consulting Civil & Structural Engineers

93 Great Suffolk Street
London SE1 0BX
T: 020 7593 1900
F: 020 7593 1910
www.sinclairjohnston.co.uk

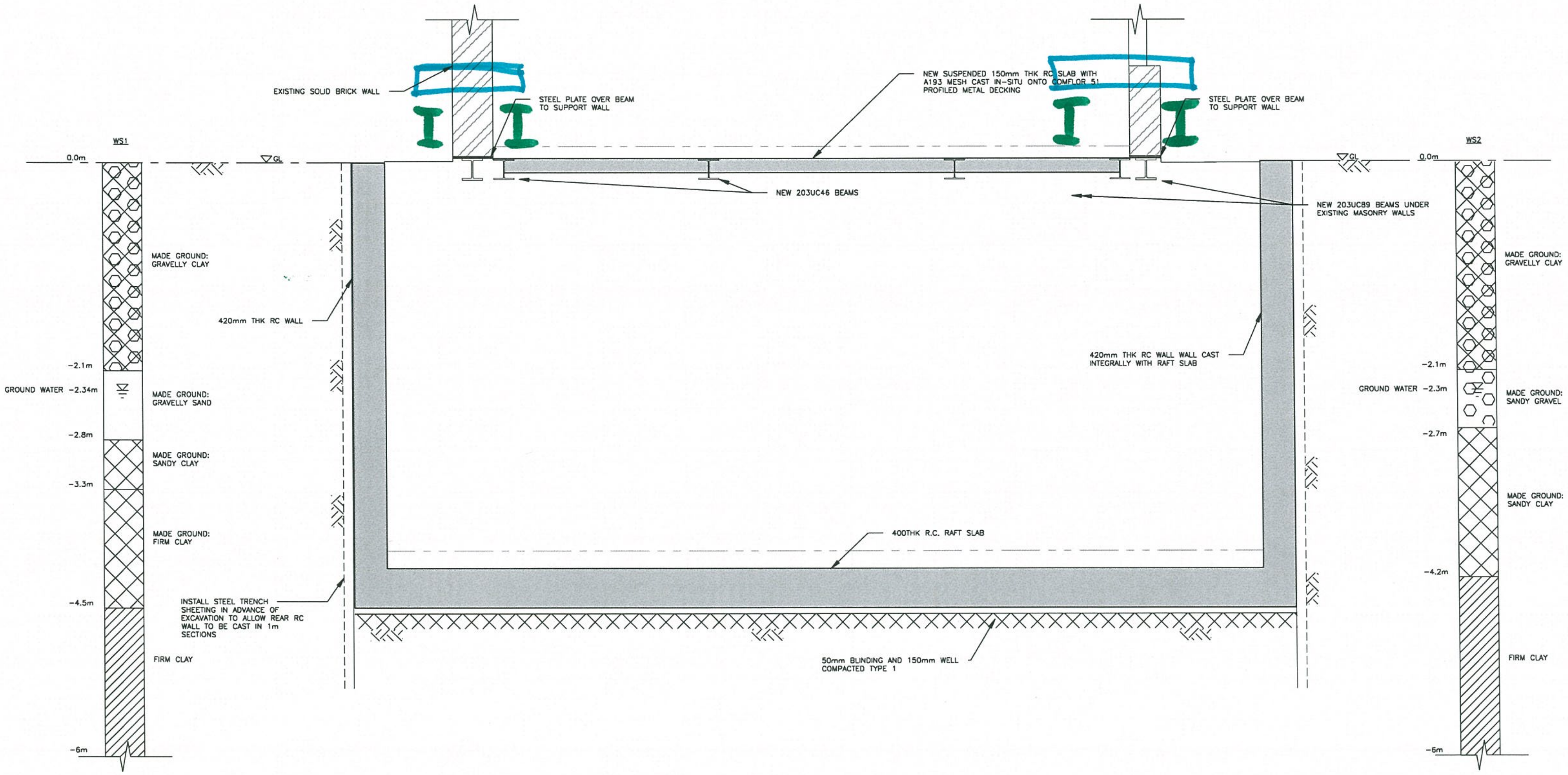
51 Gloucester Crescent
NW1

Proposed
Section B–B

Drawn: S Hill | Scale: 1:25 at A1

Project No./Drawing No.: 8761/031T | Rev: C

SH- SJP - 30.08.17



NOTES:

1. All structural engineering drawings are to be read with the specification and with all relevant Architect's and Service Engineer's drawings and specifications.
2. Do not scale from this drawing in either paper or digital form. Use written dimensions only. To check drawing has been printed to intended scale this bar should be 50mm long @ A1 or 25mm long @ A3:
3. All dimensions are in millimetres and levels in metres.

KEY:

DENOTES EXISTING SOLID BRICKWORK

THIS DRAWING IS NOT FOR CONSTRUCTION

Rev	Date	Issued	Amendment
C	18.05.17	SH	For planning
B	09.05.17	SH	For planning
A	05.05.17	SH	For planning
–	24.03.17	SH	For comment

Status PLANNING

SINCLAIR JOHNSTON
Consulting Civil & Structural Engineers

93 Great Suffolk Street
London SE1 0BX
T: 020 7593 1900
F: 020 7593 1910
www.sinclairjohnston.co.uk

51 Gloucester Crescent
NW1

Proposed
Section A–A

Drawn S Hill Scale 1:25 at A1

Project No./Drawing No. 8761/030T Rev C

SH – SJP – 30.08.17

Sinclair Johnston
and Partners Limited

93 Great Suffolk Street
London
SE1 0BX

t. 020 7593 1900
f. 020 7593 1910
email@sinclairjohnston.co.uk
www.sinclairjohnston.co.uk