CampbellReith
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123 Broadhurst Gardens, London NW6 3BJ

Basement Impact Assessment Audit

For

London Borough of Camden

Project Number: 12727-20
Revision: F1

May 2018

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Document History and Status

Revision	Date	Purpose/Status	File Ref	Author	Check	Review
D1	December 2017	Comment	RMam-12727-20-181217-123 Broadhurst Gardens-D1.doc	A Morocs	R Morley	R Morley
D2	February 2018	Planning	RMam-12727-20-310118-123 Broadhurst Gardens-D2.doc	A Morocs	R Morley	R Morley
F1	May 2018	Final Issue	RMam-12727-20-030518-123 Broadhurst Gardens-F1.doc	A Morocs	R Morley	R Morley

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Author	A Morcos MSC CEng MICE
Project Partner	E M Brown, BSc MSc CGeol FGS
Project Number	12727-20
Project Name	123 Broadhurst Gardens, London, NW63BJ
Planning Reference	2017/4684/P

Structural ◆ Civil ◆ Environmental ◆ Geotechnical ◆ Transportation

Contents

Appendix

1.0 NON-TECHNICAL SUMMARY

1.1. CampbellReith was instructed by London Borough of Camden, (LBC) to carry out an audit on the Basement Impact Assessment submitted as part of the Planning Submission documentation for 123 Broadhurst Gardens, London, NW6 3BJ (planning reference 2017/4684/P). The basement is considered to fall within Category B as defined by the Terms of Reference.

1.2. The Audit reviewed the Basement Impact Assessment for potential impact on land stability and local ground and surface water conditions arising from basement development in accordance with LBC's policies and technical procedures.

1.3. CampbellReith was able to access LBC's Planning Portal and gain access to the latest revision of submitted documentation and reviewed it against an agreed audit check list.

1.4. The individuals concerned in the production of the BIA and other submitted information have suitable qualifications.

1.5. Relevant information screening and scoping as defined and required in the LBC Planning Guidance document 'Basement and Lightwells (CPG4)' has been provided.

1.6. It is proposed to demolish an existing rear garden extension to the five storey dwelling to allow for the construction of two story extension and a 1st floor level terrace to the back of the property, formation of a basement by lowering the existing lower ground floor, formation of two lighwells to the front and one to the back of the property.

1.7. The basement is proposed to be formed by constructing using common construction techniques and methodology.

1.8. The BIA states that most of the properties in the neighbourhood have existing lighwells to the front of the property.

1.9. The BIA has confirmed that the proposed basement will be founded directly within London Clay.

1.10. Further clarification is required as to whether the surface water drainage will be adversely impacted by the proposed development due to the introduction of two front light wells. This to be provided via post planning conditioning.

1.11. It is accepted that the excavation level is unlikely to be below groundwater level or impact on groundwater flows. However further groundwater monitoring is advised to investigate perched water levels.

1.12. Preliminary design calculations for the reinforced concrete retaining structure have been submitted as part of the additional information provided.

1.13. It is accepted that the surrounding slopes to the development site are stable.

1.14. It is accepted that the development will not impact on the wider hydrogeology of the area and is not in an area subject to flooding.

1.15. A schedule of queries for further information is summarised in Appendix 2 of this audit, which have now been closed out, or to be provided via post planning conditioning.

2.0 INTRODUCTION

2.1. CampbellReith was instructed by London Borough of Camden (LBC) 4th of December to carry out a Category B Audit on the Basement Impact Assessment (BIA) submitted as part of the Planning Submission documentation for 123 Broadhurst Gardens, London, NW6 3BJ (planning reference 2017/4684/P).

2.2. The Audit was carried out in accordance with the Terms of Reference set by LBC. It reviewed the Basement Impact Assessment for potential impact on land stability and local ground and surface water conditions arising from basement development.

2.3. A BIA is required for all planning applications with basements in Camden in general accordance with policies and technical procedures contained within

- Guidance for Subterranean Development (GSD). Issue 01. November 2010. Ove Arup & Partners.
- Camden Planning Guidance (CPG) 4: Basements and Lightwells.
- Camden Development Policy (DP) 27: Basements and Lightwells.
- Camden Development Policy (DP) 23: Water.
- Local Plan Policy A5 Basements.

2.4. The BIA should demonstrate that schemes:

a) maintain the structural stability of the building and neighbouring properties;

b) avoid adversely affecting drainage and run off or causing other damage to the water environment;

c) avoid cumulative impacts upon structural stability or the water environment in the local area, and;

evaluate the impacts of the proposed basement considering the issues of hydrology, hydrogeology and land stability via the process described by the GSD and to make recommendations for the detailed design.

2.5. LBC's Audit Instruction described the planning proposal as *"Erection of a two storey rear extension with terrace at 1st floor level following the demolition of the existing rear extension and erection of a rear dormer window, lowering of the existing lower ground floor, the creation of two lightwells to the front and one at the rear with associated landscaping, and the relocation of the side access door on the west elevation. "*

2.6. The Audit Instruction also confirmed 123 Broadhurst Gardens was not, or was a neighbour to listed buildings.

2.7. CampbellReith accessed LBC's Planning Portal on 5th of December 2017 and gained access to the following relevant documents for audit purposes:

- Basement Impact Assessment prepared by PADDOCK GEO ENGINEERING, Mr M Lencher, ref. P17-071BIA, August 2017
- Structural Methodology Statement & Ground Investigation Report (ref. P17-071gi_v2) by Sinclair Johnson & Partners Ltd Consulting Civil and Structural Engineers, August 2017
- Construction Management Plan by Marek Wojciechowski Architects Ltd, 11th of August 2017
- Design and Access Statement by Marek Wojciechowski Architects Ltd, August 2017, ref. 16009
- Tree Report by John Cromar's Arboricultural Company Limited, 1st September 2017, ref. 1-38-4346
- Planning Application Drawings (by Marek Wojciechowski Architects Ltd) consisting of
- E_00 Location Plan Rev.- (11.08.17)
- Existing Plans Section & Elevations consisting of:
    - E_01 Existing Ground Floor Plan Rev.- (11.08.17)
    - E_02 Existing Lower Ground Floor Plan Rev.- (11.08.17)
    - E_07 Existing North Street Elevation Rev.- (May 2017)
    - E_08 Existing North Elevation Rev.- (11.08.17)
    - E_09 Existing South Elevation Rev.- (11.08.17)
    - E_10 Existing West Elevation Rev.- (11.08.17)
    - E_11 Existing Section A-A Rev.- (11.08.17)
    - E_12 Existing Section B-B Rev.- (11.08.17)
- Proposed Plans Sections & Elevations consisting of:
    - P_01 Proposed Ground Floor Plan Rev.- (11.08.17)
    - P_02 Proposed Lower Ground Floor Plan Rev.- (11.08.17)
    - P_08 Proposed North Elevation Rev.- (11.08.17)
    - P_09 Proposed South Elevation Rev.- (11.08.17)

- P_10 Proposed West Elevation rev. – (11.08.17)
- P_11 Proposed Section A-A Rev.- (11.08.17)
- P_12 Proposed Section B-B Rev.- (11.08.17)
- P_15 Proposed Section E-E Rev.- (11.08.17)

- Amended P_01 Proposed Ground Floor Plan rev.A (27.10.17)
- Amended P_02 Proposed Lower-Ground Floor Plan Rev.A (27.10.17)
- Amended P_11 Proposed Section A-A Rev.A (27.10.17)
- Amended P_15 Proposed Section E-E Rev.A (27.10.17)
- Amended P_09 Proposed South Elevation Rev.A (27.10.17)
- Amended P_10 Proposed West Elevation Rev.A (27.10.17)

- Planning Comments and Response:

- Consented Drawings Demolition consisting of:
    - D_01 Demolition Ground Floor Plan Rev.B (04.05.17) consented on 25.07.17
    - D_02 Demolition Lower Ground Floor Plan Rev.- (19.07.16) consented on 06.01.17
    - D_07 Demolition North Elevation Rev.- (19.07-16) consented on 06.01.17
    - D_09 Demolition South Elevation Rev.A (04.05.17) consented on 25.07.17
    - D10 Demolition West Elevation Rev.- (19.07.16) consented on 06.01.17
    - D_11 Demolition Section A-A Rev.- (19.07.16) consented on 06.01.17
    - D_12 Demolition Section B-B Rev.- (19.07.16) consented on 06.01.17
- Consented Drawings_Proposed_Part1(2) consisting of:
    - P_01 Proposed Ground Floor Plan rev.C (04.05.17) consented on 25.07.17
    - P_02 Proposed Lower-Ground Floor Plan Rev.- (21.03.17) consented on 03.05.17
    - P_07 Proposed North Elevation Rev. (19.07.17) consented on 06.01.17
    - P_09 Proposed South Elevation Rev.A (04.05.17) consented on 25.07.17
- Approved Plans Elevation drawings consisting of:

- P_10 Proposed West Elevation Rev.A (21.03.17) consented on 03.05.17
- P_11 Proposed Section A-A Rev.A (12.01.17) consented on 08.03.17
- P_12 Proposed Section B-B Rev.B (12.01.17) consented on 08.03.17

2.8. Following the D1 issue of this audit report the following additional information was received from the applicant which has been included in Appendix 3:

- Clarification email, mw-a, 03/01/18
- 8726 Dec17 Retaining Wall Design.

3.0 BASEMENT IMPACT ASSESSMENT AUDIT CHECK LIST

Item	Yes/No/NA	Comment
Are BIA Author(s) credentials satisfactory?	Yes	
Is data required by Cl.233 of the GSD presented?	Yes	
Does the description of the proposed development include all aspects of temporary and permanent works which might impact upon geology, hydrogeology and hydrology?	Yes	
Are suitable plan/maps included?	Yes	Responded to screening question adequately.
Do the plans/maps show the whole of the relevant area of study and do they show it in sufficient detail?	Yes	
Land Stability Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	A justification statement is generally provided for 'no' answer.
Hydrogeology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	A justification statement is generally provided for 'no' answer.
Hydrology Screening: Have appropriate data sources been consulted? Is justification provided for 'No' answers?	Yes	A justification statement is generally provided for 'no' answer.
Is a conceptual model presented?	Yes	
Land Stability Scoping Provided? Is scoping consistent with screening outcome?	Yes	
Hydrogeology Scoping Provided? Is scoping consistent with screening outcome?	No	All Hydrology Screening answers were No and justification has been provided.
Hydrology Scoping Provided? Is scoping consistent with screening outcome?	No	All Hydrology Screening answers were No and justification has been provided.
Is factual ground investigation data provided?	Yes	Data provided within Structural Methodology & Ground Investigation Report.

Item	Yes/No/NA	Comment
Is monitoring data presented?	Yes	
Is the ground investigation informed by a desk study?	Yes	
Has a site walkover been undertaken?	Yes	
Is the presence/absence of adjacent or nearby basements confirmed?	Yes	
Is a geotechnical interpretation presented?	Yes	
Does the geotechnical interpretation include information on retaining wall design? [	Yes	Soil Shear Strength versus Depth diagram provided within Structural Methodology & Ground Investigation Report document, no retaining wall design included (page 226) Section 10.5.
Are reports on other investigations required by screening and scoping presented?	Yes	Additional investigations have been identified but not as the result of scoping: Ground Movement Assessment, Contamination Risk Screening, UXO Desk Study and Arboricultural Report.
Are the baseline conditions described, based on the GSD?	Yes	
Do the base line conditions consider adjacent or nearby basements?	Yes	Nearby and adjacent basement is mentioned in point 6.0, potential Impact 13.
Is an Impact Assessment provided?	Yes	Basement construction sequence drawing is attached to the Structural Methodology Statement Appendix.
Are estimates of ground movement and structural impact presented?	Yes	Interpretation of modelling has been described in point 5.7.1 and supported with the result data in the Structural Methodology Statement (page 229-230) for 121 Broadhurst Gardens semi-detached property. No estimates of ground movement have been provided to 125-139 Broadhurst Gardens block of flats.
Is the Impact Assessment appropriate to the matters identified by screen and scoping?	Yes	
Has the need for mitigation been considered and are appropriate mitigation methods incorporated in the scheme?	Yes	The need for mitigation has been considered in BIA Section 5.0, but no mitigation methods have been identified as expected damage to party

Item	Yes/No/NA	Comment
		wall has demonstrated Burland Scale Damage Category 1.
Has the need for monitoring during construction been considered?	Yes	The need for monitoring by a Contractor has been identified in Section 5.9 and 5.10 of the 'Structural Methodology Statement'.
Have the residual (after mitigation) impacts been clearly identified?	No	Residual impacts have not been identified
Has the scheme demonstrated that the structural stability of the building and neighbouring properties and infrastructure will be maintained?	Yes	Initial queries existed with respect to structural stability which have been subsequently provided.
Has the scheme avoided adversely affecting drainage and run-off or causing other damage to the water environment?	No	It is unclear if the surface water drainage is adversely impacted.
Has the scheme avoided cumulative impacts upon structural stability or the water environment in the local area?	Yes	
Does report state that damage to surrounding buildings will be no worse than Burland Category 1?	Yes	
Are non-technical summaries provided?	Yes	

4.0 DISCUSSION

4.1. The Basement Impact Assessment (BIA) and Site Investigation Report have been carried out by a firm of engineering consultants, Paddock Geo Engineering Limited (PGE) and the individuals concerned in its production have suitable qualifications as required by CPG4.

4.2. The Structural Design & Construction Statement, which forms part of the wider BIA has similarly been carried out by a firm of engineering consultants, Sinclair Johnson & Partners Ltd. and the authors is a chartered structural engineer.

4.3. The BIA submissions include land Stability, Hydrogeology and Hydrology screening and scoping, relevant site investigations and impact assessment as defined and required in the LBC Planning Guidance document 'Basement and Lightwells (CPG4).

4.4. The LBC Instruction to proceed with the audit identified that the basement proposal neither involved a listed building nor was adjacent to listed building.

4.5. The Design & Access Statement identified that 123 Broadhurst Gardens is located in the South Hampstead Conservation Area and basement conversions are common to the area. The existing property is of five stories and semi-detached and is arranged over lower ground floor, ground, first and second floor and converted loft with grounds to the front and rear. The site is bounded to the other side by block of flats 125-139 Broadhurst Gardens with an existing single story basement and the proposed basement is within three meters at the closest to the property. However, a basement impact assessment has not been discussed for the block of flats. The rear of the property shears private residential gardens.

4.6. The BIA has identified that front and back of the site differ in levels with a change of approximately 2.00 meters. The existing property has a lower ground floor founded at the formation of the rear garden.

4.7. The submitted proposal is a modification to the consented set of development proposal converting the property from two flats to a single family house, with a loft conversion to habitable floor and modification of internal structural walls. An existing rear extension is to be demolished to accommodate a new two storey extension, a first floor level terrace and lightwell within the perimeter of the old rear extension. In addition two front lightwells are proposed.

4.8. The proposed basement consists of a single storey construction formed by lowering an existing lower ground floor area under the entire footprint of the building and underneath the external rear garden stairs by 2.0 metres, and excavating the front portion of the site to the same level to form a lightwell.

4.9. A site specific investigation was conducted comprising two bore holes to a depth of 8mbgl within the front garden and 6.8mbgl in the rear garden, along with five trial pits within the perimeter of the existing lower ground floor on 27th of June 2017.

4.10. The front of the site is consists of 1.5 meter of Made Ground, overlaying the London Clay Formation. The London Clay consists of stiff orange brown silty sandy gravelly Clay to the depth of 2.5-2.6m in bore hole WS1 and 3.7-3.9m in bore hole WS2, overlaying moderately strong dark grey Claystone. The top of the Clay has been possibly identified as the Claygate member given its inclusion of granular material however it has been logged as London Clay. The London Clay Formation has been found at the surface of the rear garden. Therefore, the formation level of the proposed concrete floor basement is in the London Clay.

4.11. The site is not indicated to lie within a groundwater Source Protection Zone (SPZ) or within 500m of one. However, a now culverted headwater tributary of the former River Westbourne lays approximately 200 meters west of the site.

4.12. The site has a very low flooding risk from surface water and sewers, reservoirs and fluvial/tidal watercourses.

4.13. The GI identifies suspected perched water has been observed to rise to level 3.8 meters bgl. in one bore hole WS2 at the rear garden of the property. No standing groundwater was observed in the standpits within the boreholes during the observation period. However monitoring was carried out during the summer period and it is advised that additional monitoring should be carried out during wet periods, to monitor the perched water, particularly given the potential presence of the more permeable Claygate member overlaying the London clay due to its ground water bearing capacity.

4.14. The Ground Investigation indicated that the basement will be constructed within a perched water table and recommends water-proofing and tanking of the basement.

4.15. It is accepted that the excavation level is unlikely to be below groundwater level or impact on groundwater flows. However it would be prudent for dewatering to be prepared.

4.16. The hydrogeological screening states: "London Clay is not suitable for SUDS based soakways and the current drainage will be maintained". However it is not clear if the amount of surface water discharge will increase and it should be demonstrated that either surface water discharge is not increased or otherwise the use of SUDs investigated to attenuate surface water discharge.

4.17. The proposed method involves underpinning the existing internal and external walls from inside the existing lower ground floor level, constructing the reinforced concrete retaining walls to form lightwells and for the rear of the basement. The construction method is proposed as hit and miss underpinning with maximum pin length of 900mm, with a raft basement foundation slab.

4.18. Preliminary design calculations for the reinforced concrete retaining walls have been submitted for the back garden and the neighbouring to 121 Broadhurst Gardens property and are attached to the Appendix 3.

4.19. It is accepted that the surface water drainage system will remain unchanged for the rear garden surface area; however further clarification is required in regards to increase in the impermeable surface area that drains to the sewer system at the front garden, as light wells are formed at the front of the property. The change in the surface area that drains to the sewer system needs to be confirmed, with the use of SUDs investigated should any increase be identified. This to be provided via post planning conditioning.

4.20. A need for movement monitoring is identified prior to construction of the underpin walls through to completion of the basement structure; however a monitoring strategy should be confirmed with the Party Wall Surveyor and should be updated with any updates to the structural design, damage impact assessment or temporary works proposals, following agreement of the construction strategy with the contractor.

4.21. A ground movement assessment has been produced by Paddock Geo Engineering Limited (PGE) which follows the method as described in CIRIA580. Vertical and horizontal ground movements have been calculated due to excavation with a damage category to neighbouring properties be no higher than category 1 on the Burland Scale calculated. While the method described in CIRIA760 is not strictly applicable to underpinning or L-shaped retaining walls, it is accepted that this would provide a conservative analogy when applied in this manner.

4.22. It is accepted that there are no slope stability concerns regarding the proposed development and it is not in an area prone to flooding.

4.23. A schedule of queries for further information is summarised in Appendix 2 of this audit, which have now been closed out, or to be provided via post planning conditioning.

5.0 CONCLUSIONS

5.1. The Basement Impact Assessment (BIA) has been carried out by Paddock Geo Engineering Limited (PGE), consultant engineers, by individuals who have suitable qualifications. An external chartered geologist consultant supported the hydrogeological assessment.

5.2. The BIA submissions include Land Stability, Hydrogeology and Hydrology screening and scoping, relevant site investigations and impact assessments as defined and required in the LBC Planning Guidance document 'Basement and Lightwells (CPG4)'.

5.3. The proposed basement consists of a single storey construction formed by lowering an existing lower ground floor area under all footprint of the existing building, and to the rear and front of the property.

5.4. The proposed basement is bordered by, 121 Broadhurst Gardens to the East with a shallow lower ground floor basement, 139 Broadhurst Gardens to the West with an existing single story basement and private gardens to the rear. The BIA states that most of the properties in the neighbourhood have existing lighwells to the front of the property.

5.5. A ground movement assessment has been produced for 121 Broadhurst Gardens that indicates a worst case damage category of 1 (very slight). 139 Broadhurst Gardens has an existing basement within an influence zone and has therefore not been considered within the GMA due to similar basement depth.

5.6. The BIA has confirmed that the proposed basement will be founded within London Clay.

5.7. It is accepted that the groundwater was not encountered during boring or within the standpipe during monitoring. However, monitoring was carried out during the summer period and it is advised that additional monitoring of perched water should be carried out during wet periods.

5.8. Preliminary design calculations for the reinforced concrete retaining structure were submitted.

5.9. Clarification is required in regards to increase in impermeable surface area that is drained to the sewer system as the front garden light wells are formed and the requirement of SUDs needs to be investigated. This to be provided via post planning conditioning.

5.10. It is accepted that the excavation level is unlikely to be below groundwater level or impact on groundwater flows.

5.11. The monitoring strategy should be confirmed with the Party Wall Surveyor and should be updated with any updates to the structural design, damage impact assessment or temporary works proposals, following agreement of the construction strategy with the contractor.

5.12. It is accepted that the surrounding slopes to the development site are stable.

5.13. A schedule of queries for further information is summarised in Appendix 2 of this audit, which have now been closed out, or to be provided via post planning conditioning.

Appendix 1: Residents' Consultation Comments

Residents' Consultation Comments

Surname	Address	Date	Issue raised	Response
Mr. Onn Tammuz	121 Broadhurst Gardens London NW6 3BJ	27/09/2017	1. Structural Risk to semi-detached property has been identified by the property owner concerning 2m deep basement excavations and the need to underpin 16m party wall. 2. Water damage risk to the property as the result of the development.	1. The settlement of the party wall has been modelled in the analytical software which has demonstrated predicted damage category 1 to the neighbouring property on the Burland Scale. Movement monitoring has been suggested during demolishment works and construction of the basement. Sinclair Johnson & Partners Structural Methodology Statement shows proposed hit and miss type of underpinning to the foundation of the party wall. The construction method is acceptable to this type of proposal. 2. Ground Investigation Report by Paddock Geo Engineering has demonstrated, that the excavation level is unlikely to be below groundwater level or impact on groundwater flows.

Appendix 2: Audit Query Tracker

Audit Query Tracker

Query No	Subject	Query	Status	Date closed out
1	Hydrology	Details of existing surface water drainage to demonstrate change in discharge to the sewer system, with SUDs proposed if required.	Not applicable, to be provided via post planning conditioning	-
2	Construction	Preliminary design calculations for the reinforced concrete retaining structure should be submitted to demonstrate feasibility of proposal.	Closed	31/01/18

Appendix 3: Supplementary Supporting Documents

Appendix 3.1 email08 01 18
Appendix 3.2 8726 Dec17 Retaining Wall Design

----- Forwarded by Graham Kite/CRH on 08/01/2018 12:42 -----

From:	"Hope, Obote" <Obote.Hope@camden.gov.uk>
To:	"GrahamKite@campbellreith.com" <GrahamKite@campbellreith.com>
Date:	08/01/2018 11:59
Subject:	FW: 123 Broadhurst Gardens (2017/4684/P) - BIA report

Hi Graham,

Please see the follow up enquiry in regards to retaining wall design for 123 Broadhurst Gardens.

Kind Regards,

Obote Hope
Planner
Regeneration and Planning
Culture and Environment
London Borough of Camden

Telephone: 020 7974 2555
Web: camden.gov.uk
2nd Floor
5 Pancras Square
London N1C 4AG

From: Megan White [mailto:megan@mw-a.co.uk]
Sent: 03 January 2018 10:38
To: Hope, Obote <Obote.Hope@camden.gov.uk>
Cc: Robert Douge <robert@mw-a.co.uk>; Tori MacCabe <tori@mw-a.co.uk>
Subject: RE: 123 Broadhurst Gardens (2017/4684/P) - BIA report

Good Morning Obote,

In regards to the outstanding Audit Queries from Campbell Reith below please forward the following responses:

Query 1: The surface water drainage system will remain unchanged as the area of impermeable surfaces will remain within the footprint of the previously consented planning application approved last year. Please refer to the plans of the consented scheme showing the consented works: rebuilding of the rear of the house and a paved terrace to the rear. The proposed basement only extends under the existing house footprint and consented rear paved terrace. The analysis made in the BIA remains accurate as the area of impermeable area is not increasing between the previously consented scheme (currently under construction) and the proposed scheme therefore the surface water impact remains unchanged.

Query 2: The design calculations for the reinforced concrete retaining structure are attached.

In the event that Campbell Reith would like to further discuss the queries following the responses above, please direct to:

Sian Hill
Engineer – Sinclair Johnston & Partners
D: 020 7593 1907 E: shill@sinclairjohnston.co.uk
I would greatly appreciate, if you could confirm receipt and that the query responses have been issued to Campbell Reith.
Kind Regards,
Megan White

MAREK WOJCIECHOWSKI ARCHITECTS
LONDON

www.mw-a.co.uk | t:020 7580 9336 | DD: 020 3617 5168
Registered in England Marek Wojciechowski Architects Limited No. 08217864
Registered office: 311 Ballards Lane, North Finchley, London, N12 8LY

From: Sexton, Gavin [mailto:gavin.sexton@camden.gov.uk]
Sent: 19 December 2017 09:30
To: Megan White <megan@mw-a.co.uk>
Cc: Tulloch, Rob <Rob.Tulloch@camden.gov.uk>
Subject: 123 Broadhurst Gardens (2017/4684/P) - BIA report

Dear Megan White

The case officer for the above application (Rob Tulloch) is on leave until 27th Dec so I have been asked to notify you that Campbell Reith's Initial Report on their independent assessment of the Basement Impact Assessment submitted with the application is now available online.

Their report finds (appendix 2) that there are two outstanding issues which must be addressed by revisions/clarifications to the BIA.

I suggest you discuss the results with the author of the BIA and discuss with Rob when he returns.

regards

Gavin Sexton
Principal Planner
Regeneration and Planning
Supporting Communities

London Borough of Camden

Telephone: 020 7974 3231
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8726 Dec17 Retaining Wall Design.pdf

SINCLAIR JOHNSTON
& PARTNERS LIMITED

Date Dec 17	Job No.	Sheet No.
Eng. SH	8726	1
Review RA.		
Project Broadhurst Gardens		

R.C. Retaining Wall design

Floor Loads:

Timber floors DL: 0.7kNm^2

LL: 1.5kNm^2

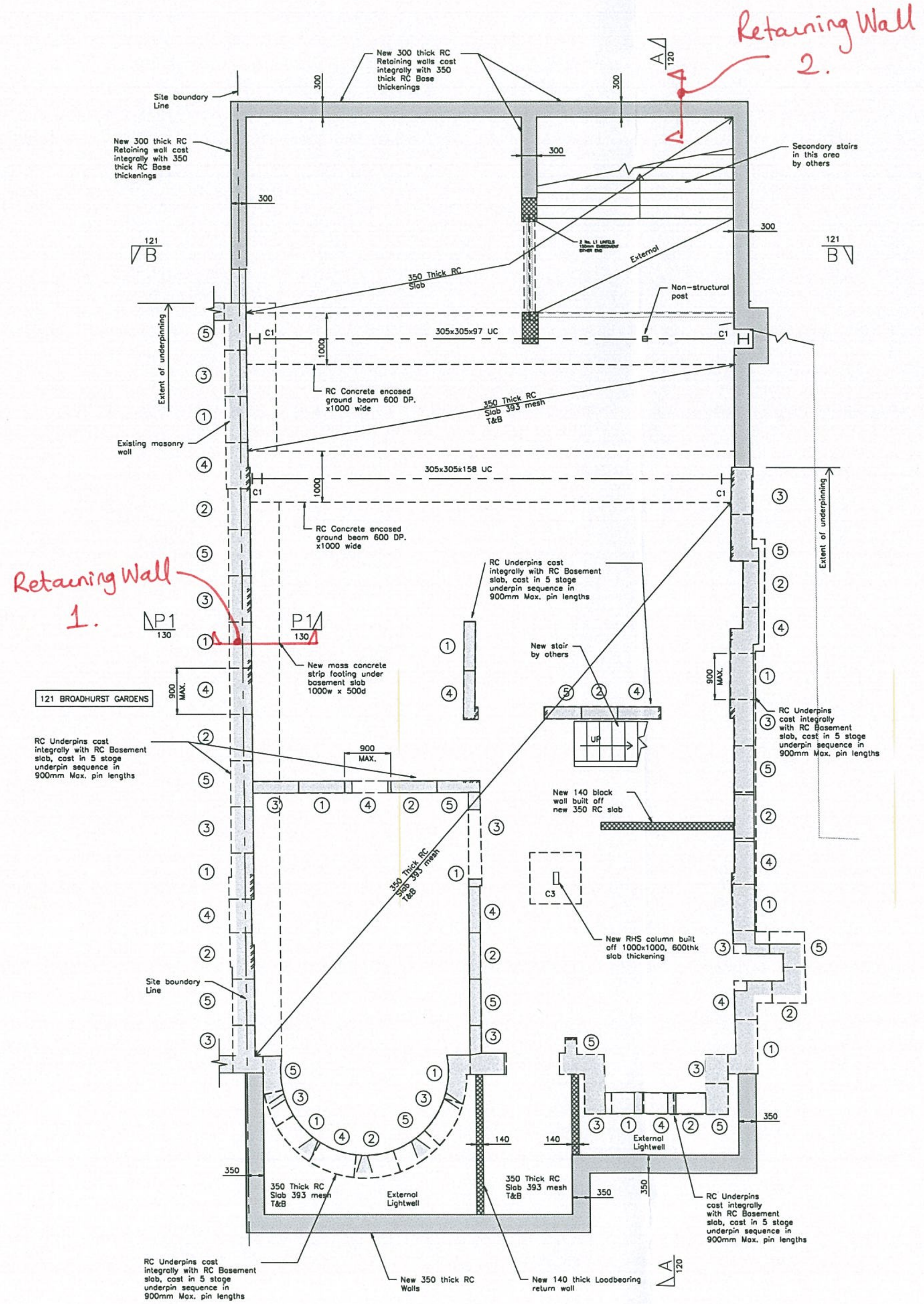
Ground Floor RC Slab: DL: 5.1kNm^2

LL: 1.5kNm^2

Roof (timber joists): DL: 0.9kNm^2

LL: 0.6kNm^2

Solid loadbearing wall: 2.1kNm^2 per 100mm(w)



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:

- New Reinforced concrete.
- Existing masonry.
- New blockwork masonry.
- Structure under.
- Structure demolished.
- New steel beam
- (2) Stage sequence of RC underpinning
- L1. 100 x 215dp PRECAST CONCRETE LINTEL

COLUMN REFERENCE TABLE	
REF.	SIZE
C1	203x203x46 UC
C2	100x100x10 SHS
C3	250x100x10 RHS

Rev	Date	Issued	Amendment
D	28.09.17	SH	Issued for Construction
C	20.09.17	SH	Issued for Tender
B	30.08.17	SH	Issued for Tender
A	11.08.17	DJP	Issued for Planning
–	03.08.17	DJP	Issued for Planning

Status CONSTRUCTION

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123 Broadhurst Gardens
NW6

Proposed
Basement Plan

Drawn D Phillips Scale 1:50 at A1

Project No./Drawing No. 8726/109 Rev D

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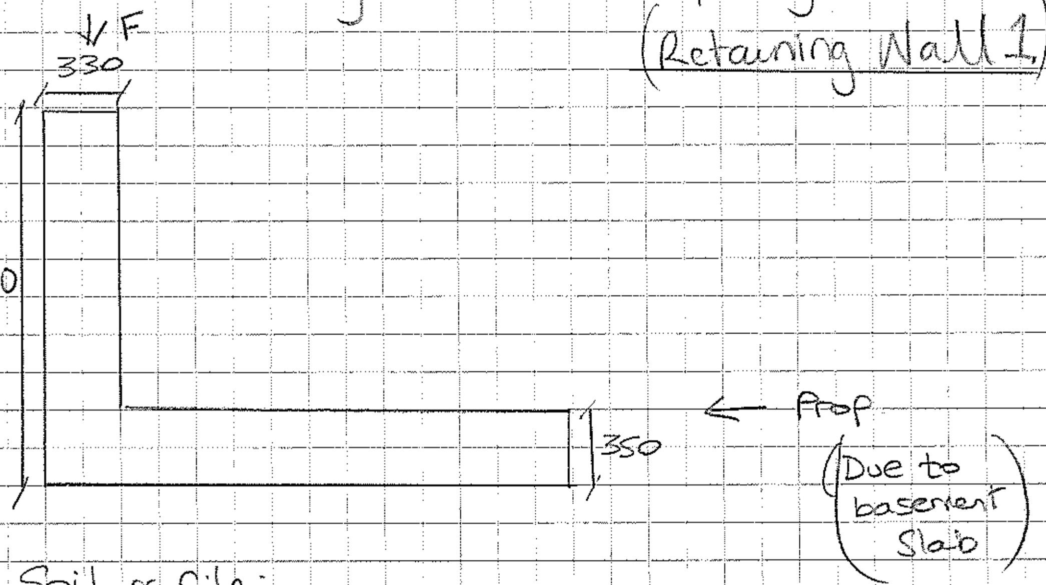
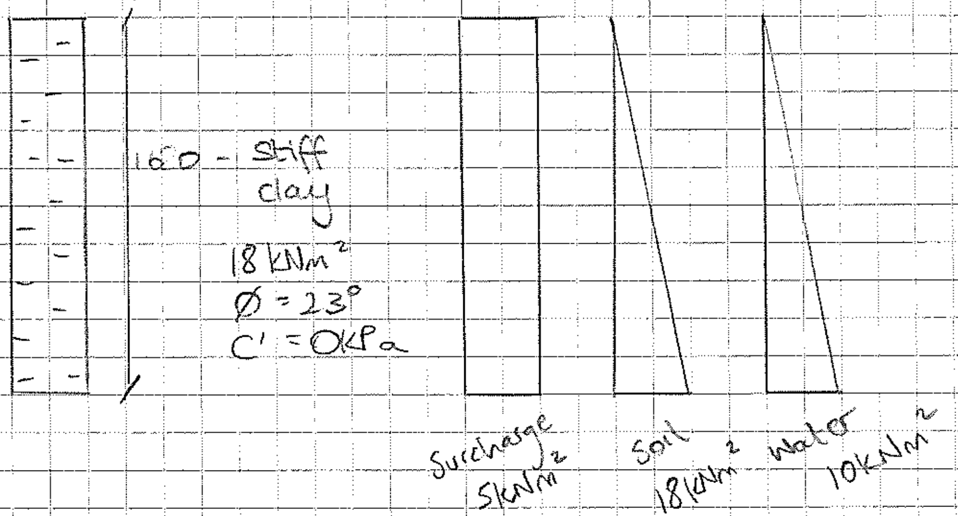
Date Dec 17	Job No.	Sheet No.
Eng.	8726	2
Review		
Project Broadhurst Gardens		

R.C. Retaining Wall under party wall.

(Retaining Wall 1)



Soil profile:



Party Wall Loading (F)

- Wall over: 330Thk solid brickwall 6.3m(h) then above 215thk solid brickwall 5.9m(h):

DL = 2.1 x 3.3 x 6.3m(h) = 44 kNm

2.1kNm² x 2.15(th) x 5.9m(h) = 26kNm

Total DL = 70 kN/m

SJ 2010

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Date Dec 17.	Job No.	Sheet No.
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Project Broadhurst Gardens		

R.C. Retaining Wall under Party Wall

Loading (F):

- Ground Floor timber joists:

  DL = $0.7 kN/m^2 \times 2.1m = 1.5 kN/m$

  LL = $1.5 kN/m^2 \times 2.1m = 3.2 kN/m$

- 1st Floor timber joists (as above)

  DL = 1.5kN/m   LL = 3.2kN/m

- second Floor timber joists (as above)

  DL = 1.5kN/m   LL = 3.2kN/m

- 2nd Floor 100thk wall:

  DL = $2.1 kN/m^2 \times 2.5m(h) \times 2.1m(w) = 11kN$

- 3rd Floor Timber joists.

  DL = $0.7 kN/m^2 \times 2.3m = 1.6 kN/m$

  LL = $1.5 kN/m^2 \times 2.3m = 3.5 kN/m$

Total: DL (SLS) = 78 kN/m

LL (SLS) = 13.1 kN/m

See Tedds output for RC details & spec

SJ 2010

TEKLA Tedds	Project: 123 Brodhurst Gardens			Job no.: 8726	
	Calcs for: Underpin Party Retaining Wall 1.			Start page no./Revision: 1	
	Calcs by: SH	Calcs date: 20/12/2017	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} = **1330** mm
Stem thickness	t_{stem} = **330** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2000** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1330** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1330** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Stiff clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **11** deg

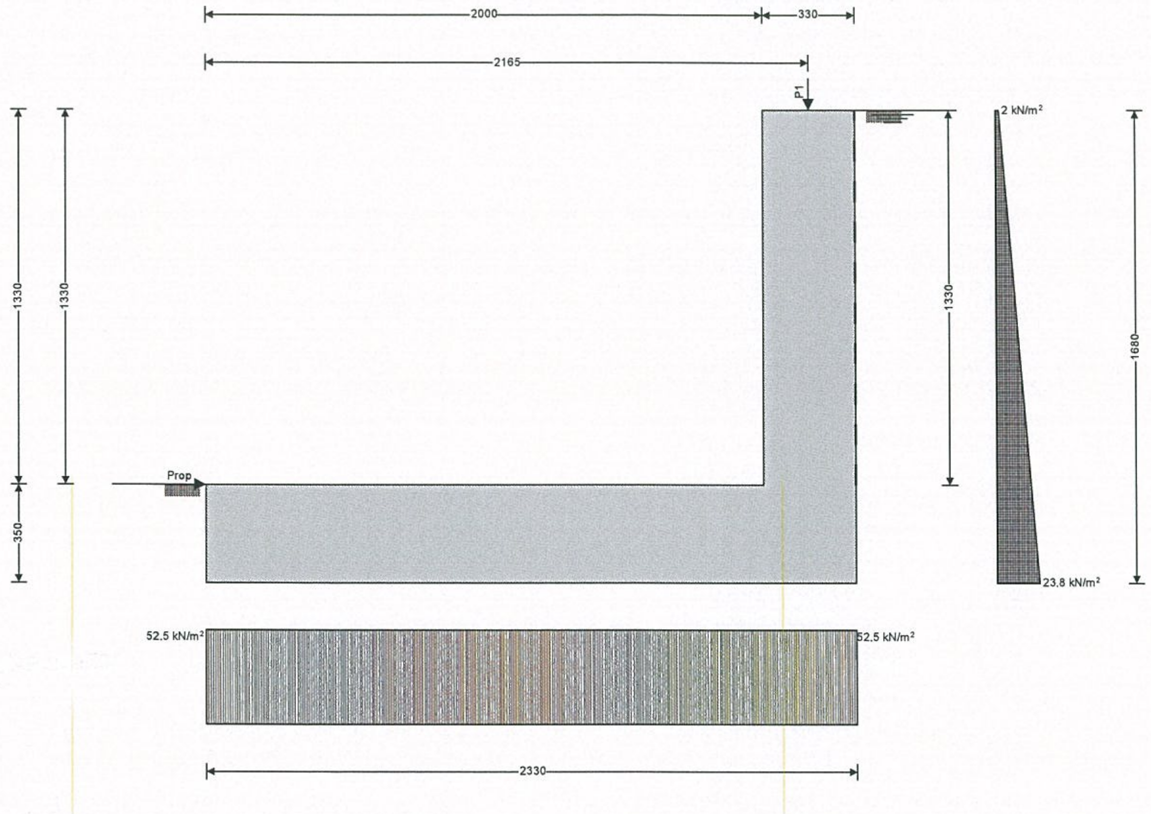
Base soil properties

Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **11** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	Surcharge$_Q$ = **5** kN/m^2
Vertical line load at 2165 mm	P_{G1} = **78** kN/m
	P_{Q1} = **13** kN/m

TEKLA Tedds	Project 123 Brodhurst Gardens				Job no. 8726	
	Calcs for Underpin Party Retaining Wall				Start page no./Revision 2	
	Calcs by SH	Calcs date 20/12/2017	Checked by	Checked date	Approved by	Approved date



Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{2330}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{1330}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2330}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{1680}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{840}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.439}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2165}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.816}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1165}$ mm

Using Coulomb theory

Active pressure coefficient

$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.398}$

Passive pressure coefficient

$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{3.094}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{11}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{20.4}$ kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{91}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v} = \mathbf{122.4}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff}$ = **3.3** kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **4.5** kN/m
Water	$F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **13.8** kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-3.3** kN/m
Total	$F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **18.3** kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem}$ = **23.8** kNm/m
Wall base	$M_{base} = F_{base} \times x_{base}$ = **23.8** kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-2.8** kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1$ = **197** kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = **-2.5** kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h}$ = **-7.8** kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P$ = **231.5** kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h}$ = **18.3** kN/m
Distance to reaction	$\bar{x} = l_{base} / 2$ = **1165** mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2$ = **0** mm
Loaded length of base	$l_{load} = l_{base}$ = **2330** mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base}$ = **52.5** kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base}$ = **52.5** kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.904**

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} = **32** N/mm^2
Characteristic compressive cube strength	$f_{ck,cube}$ = **40** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **40** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **3.0** N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.1** N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **33346** N/mm^2
Partial factor for concrete - Table 2.1N	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **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} = **35** mm

Rear face of stem c_{sr} = **75** 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 = **330** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **9.5** kNm/m

Depth to tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2$ = **249** mm

$K = M / (d^2 \times f_{ck})$ = **0.005**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **237** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **31** mm

Area of tension reinforcement required $A_{sr.req} = M / (f_{yd} \times z)$ = **92** mm²/m

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

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **565** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **392** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sr.max} = 0.04 \times h$ = **13200** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.692**

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 $\rho = A_{sr.req} / d$ = **0.000**

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 $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **670.2**

Actual span to depth ratio h_{stem} / d = **5.3**

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} = **5.6** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **42** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **99625** mm²/m

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

Reinforcement ratio $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.006**

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 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **614** 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 = **0.077** mm

w_k / w_{max} = **0.258**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — V = **19.4** kN/m

$C_{Rd.c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2)$ = **1.896**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.002**

$v_{min} = 0.035\text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.517** N/mm^2

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

$V_{Rd.c}$ = **128.7** kN/m

$V / V_{Rd.c}$ = **0.151**

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) — $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **330** mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2) — s_{sx_max} = **400** mm

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

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

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

Check base design at toe

Depth of section — h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **119.8** kNm/m

Depth to tension reinforcement — $d = h - c_{bb} - \phi_{bb} / 2$ = **265** mm

$K = M / (d^2 \times f_{ck})$ = **0.053**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **252** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **33** mm

Area of tension reinforcement required — $A_{bb.req} = M / (f_{yd} \times z)$ = **1095** mm^2/m

Tension reinforcement provided — 20 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **2094** mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **417** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{bb.max} = 0.04 \times h$ = **14000** mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.523**

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} = **87.5** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **166** N/mm^2

Load duration — Long term

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

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{105625}$ mm²/m

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

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.020}$

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 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{426}$ mm

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

$w_k = \mathbf{0.212}$ mm

$w_k / w_{max} = \mathbf{0.708}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{119.8}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.869}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.008}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.506}$ N/mm²

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

$V_{Rd.c} = \mathbf{174.4}$ kN/m

$V / V_{Rd.c} = \mathbf{0.687}$

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) $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{419}$ mm²/m

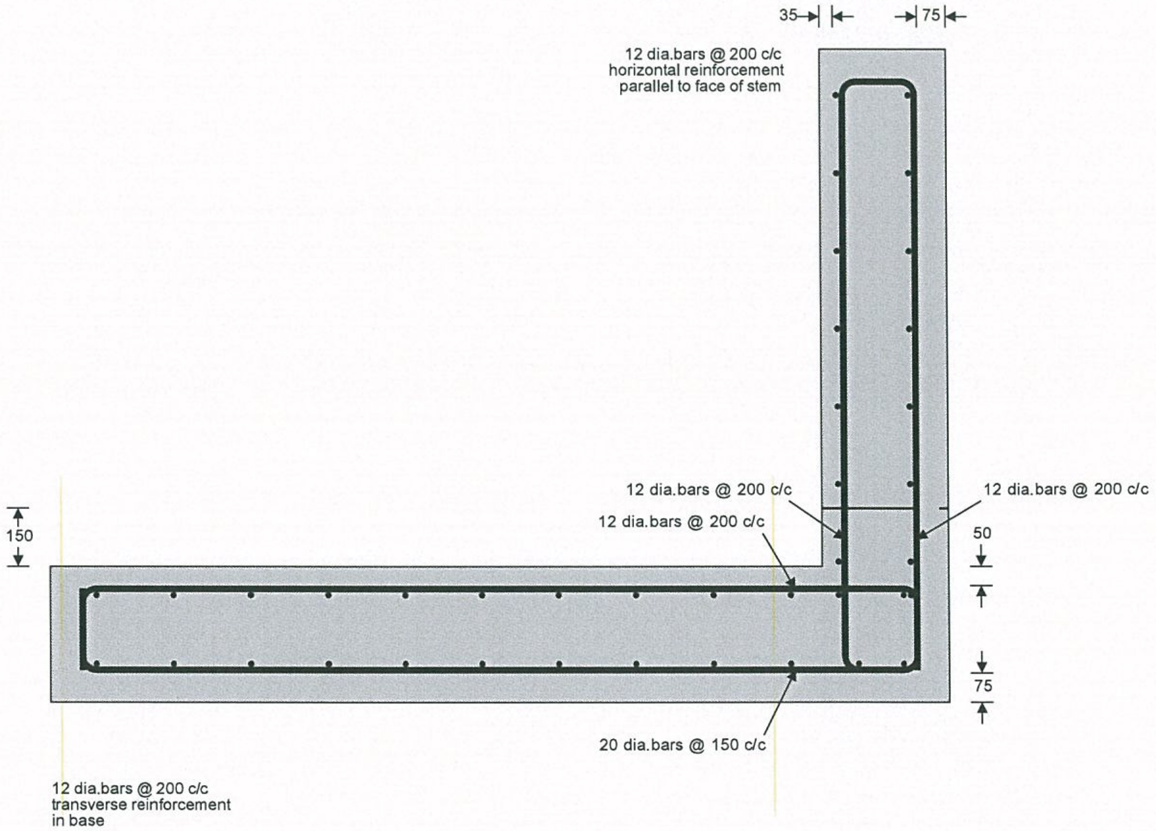
Maximum spacing of reinforcement – cl.9.3.1.1(3) $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided 12 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{565}$ mm²/m

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

TEKLA Tedds	Project 123 Brodhurst Gardens				Job no. 8726	
	Calcs for Underpin Party Retaining Wall				Start page no./Revision 7	
	Calcs by SH	Calcs date 20/12/2017	Checked by	Checked date	Approved by	Approved date

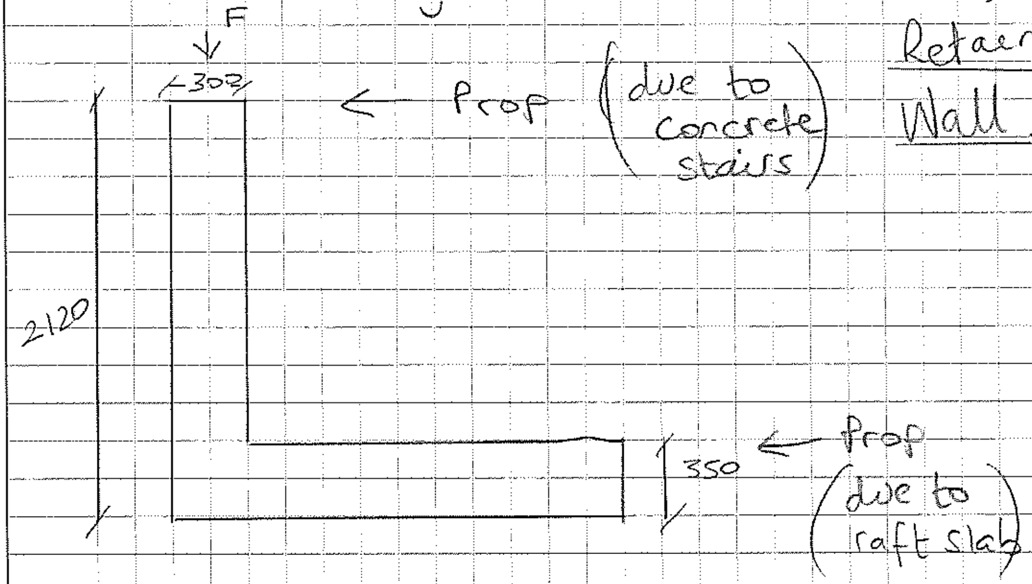


SINCLAIR JOHNSTON
& PARTNERS LIMITED

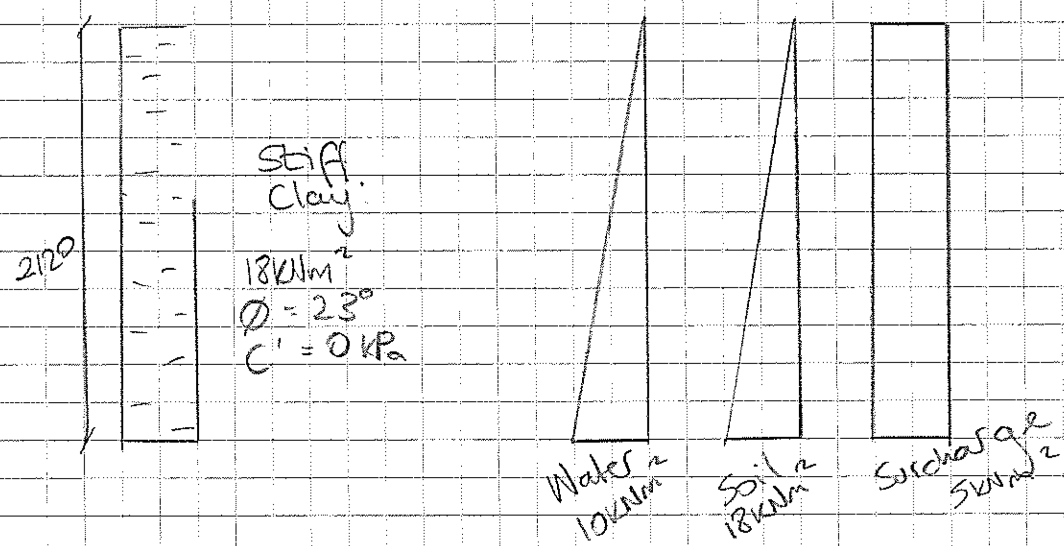
Date Dec 17	Job No.	Sheet No.
Eng.	8726	4
Review		
Project Broadhurst Gardens		

R.C. Retaining Wall (Rear extension)

Retaining Wall 2.



Soil profile:



Retaining Wall loading (F)

- Concrete stairs : length: 3.2m Waist: 280mm

$DL = 25 kN/m^3 \times 0.28m = 7 kN/m^2 \times 1.6m = 11.2 kN/m$

$LL = 1.5 kN/m^2 \times 1.6m = 2.4 kN/m$.

See Tedds output for reinforcement details

TEKLA Tedds	Project 123 Brodhurst Gardens				Job no. 8726	
	Calcs for Rear Retaining Wall 2.				Start page no./Revision 1	
	Calcs by SH	Calcs date 20/12/2017	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	Propped cantilever
Stem height	h_{stem} = **1770** mm
Prop height	h_{prop} = **1770** 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} = **2000** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1770** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1770** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Stiff clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **11** deg

Base soil properties

Soil type	Firm clay
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **11** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2150 mm	P_{G1} = **11.2** kN/m
	P_{Q1} = **2.4** kN/m

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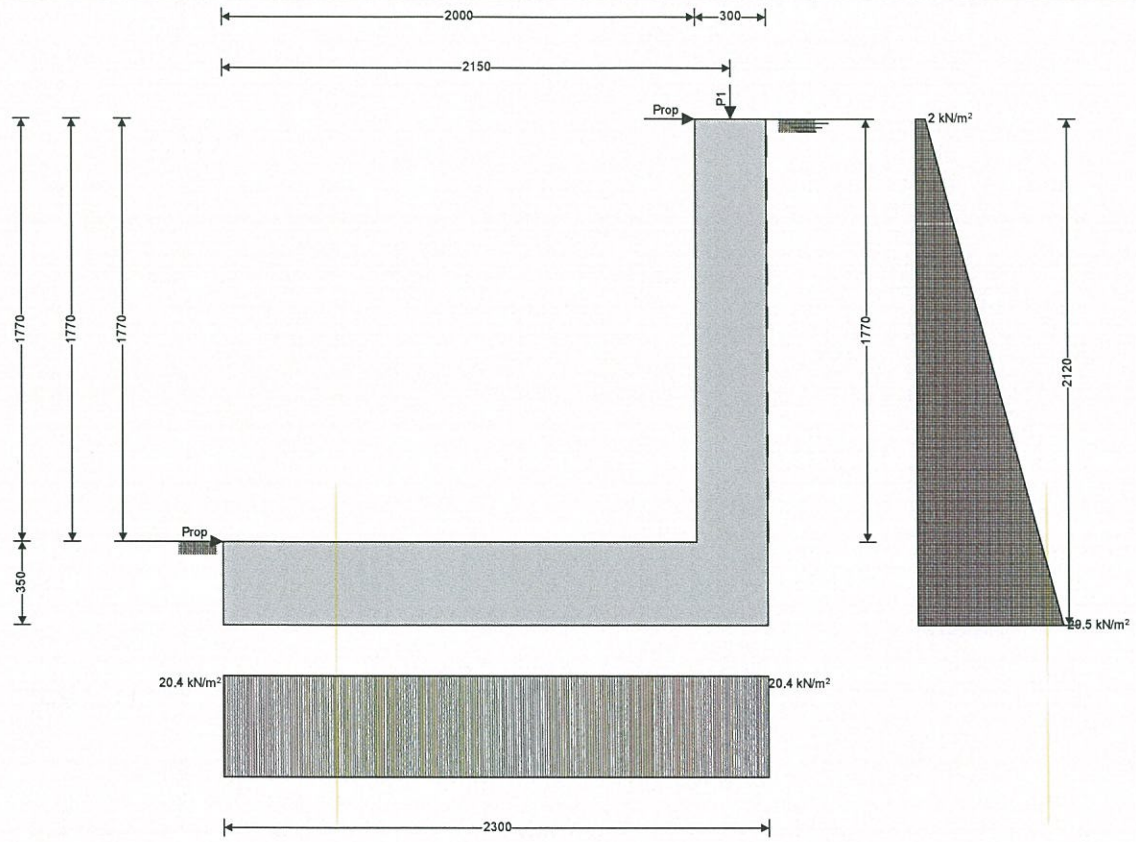

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{2300}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{1770}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2300}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2120}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1060}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.531}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2150}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.805}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1150}$ mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.398}$
Passive pressure coefficient	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{3.094}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{13.3}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{20.1}$ kN/m

Line loads $F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{13.6}$ kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{water\_v} + F_{P\_v} = \mathbf{47}$ kN/m

Horizontal forces on wall

Surcharge load $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} = \mathbf{4.1}$ kN/m

Saturated retained soil $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{7.2}$ kN/m

Water $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{22}$ kN/m

Base soil $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-3.3}$ kN/m

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{30}$ kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem} = \mathbf{28.5}$ kNm/m

Wall base $M_{base} = F_{base} \times x_{base} = \mathbf{23.1}$ kNm/m

Surcharge load $M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-4.4}$ kNm/m

Line loads $M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{29.2}$ kNm/m

Saturated retained soil $M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-5.1}$ kNm/m

Water $M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-15.6}$ kNm/m

Total $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} + M_{sur} + M_P = \mathbf{55.9}$ kNm/m

Check bearing pressure

Propping force to stem $F_{prop\_stem} = \min((F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}), F_{total\_h}) = \mathbf{-0.9}$ kN/m

Propping force to base $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{30.9}$ kN/m

Moment from propping force $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{-1.8}$ kNm/m

Distance to reaction $\bar{x} = l_{base} / 2 = \mathbf{1150}$ mm

Eccentricity of reaction $e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm

Loaded length of base $l_{load} = l_{base} = \mathbf{2300}$ mm

Bearing pressure at toe $q_{toe} = F_{total\_v} / l_{base} = \mathbf{20.4}$ kN/m^2

Bearing pressure at heel $q_{heel} = F_{total\_v} / l_{base} = \mathbf{20.4}$ kN/m^2

Factor of safety $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{4.894}$

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 $h_{agg} = \mathbf{20}$ 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} = **35** mm

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

Top face of base — c_{bt} = **50** mm

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

Check stem design at 826 mm

Depth of section — h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **3.5** kNm/m

Depth to tension reinforcement — $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **249** mm

$K = M / (d^2 \times f_{ck})$ = **0.002**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **237** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **31** mm

Area of tension reinforcement required — $A_{sfM.req} = M / (f_{yd} \times z)$ = **34** mm^2/m

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

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

Minimum area of reinforcement - exp.9.1N — $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **392** mm^2/m

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

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov}$ = **0.692**

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

Deflection control - Section 7.4

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

Required tension reinforcement ratio — $\rho = A_{sfM.req} / d$ = **0.000**

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

Structural system factor - Table 7.4N — K_b = **1**

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

Limiting span to depth ratio - exp.7.16.a — $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}]$ = **7414.8**

Actual span to depth ratio — h_{prop} / d = **7.1**

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} = **2.3** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **17** N/mm^2

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **89625** mm^2/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm} =$ **3.0** N/mm^2

Reinforcement ratio $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff} =$ **0.006**

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 $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff} =$ **442** mm

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

$w_k =$ **0.023** mm

$w_k / w_{max} =$ **0.075**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **7.6** kNm/m

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

$K = M / (d^2 \times f_{ck}) =$ **0.005**

$K' =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d =$ **208** mm

Depth of neutral axis $x = 2.5 \times (d - z) =$ **27** mm

Area of tension reinforcement required $A_{sr.req} = M / (f_{yd} \times z) =$ **85** mm^2/m

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

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

Minimum area of reinforcement - exp.9.1N $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **344** mm^2/m

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

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} =$ **0.609**

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 $\rho = A_{sr.req} / d =$ **0.000**

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

Structural system factor - Table 7.4N $K_b =$ **1**

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 $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}] =$ **1574.1**

Actual span to depth ratio $h_{prop} / d =$ **8.1**

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 $\psi_2 =$ **0.3**

Serviceability bending moment $M_{sls} =$ **5** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **42.8** N/mm^2

Load duration Long term

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

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{90875}$ mm²/m

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

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

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 $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{583}$ mm

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

$w_k = \mathbf{0.075}$ mm

$w_k / w_{max} = \mathbf{0.25}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{25.3}$ 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 $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.541}$ N/mm²

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

$V_{Rd.c} = \mathbf{118.6}$ kN/m

$V / V_{Rd.c} = \mathbf{0.213}$

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

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

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{7.5}$ 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 $\rho_l = \min(A_{sr1.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.541}$ N/mm²

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

$V_{Rd.c} = \mathbf{118.6}$ kN/m

$V / V_{Rd.c} = \mathbf{0.063}$

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) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{300}$ mm²/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 $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{393}$ mm²/m

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

Check base design at toe

Depth of section $h = \mathbf{350}$ mm

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Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **31.9** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **269** mm
	$K = M / (d^2 \times f_{ck})$ = **0.014**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **256** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **34** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **287** mm²/m
Tension reinforcement provided	12 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **754** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **423** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h$ = **14000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.561**

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} = **23.4** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **121.3** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **105458** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.007**
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	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **540** 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 = **0.197** mm
	w_k / w_{max} = **0.655**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **31.9** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.862**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.003**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.503** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **135.3** kN/m
	$V / V_{Rd.c}$ = **0.235**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

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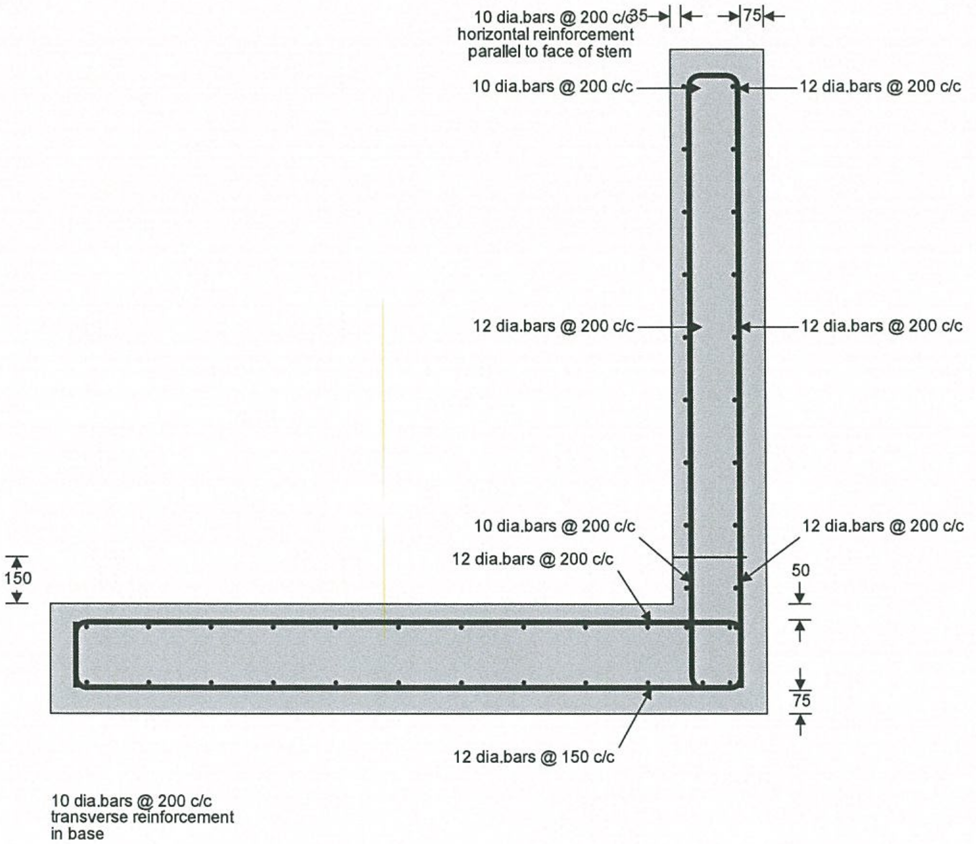
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{151}$ mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{393}$ mm²/m

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



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