Cooper Associates
Consulting Structural Engineers

CA5591.01 - Rev B

September 2018

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Construction Method Statement

Flat 1, 9-10 Regent Square, London, WC1H 8HZ

Proposed extension on the rear of an existing five storey terraced property

INTRODUCTION

It is proposed to carry out a complete refurbishment of the above flat, comprising the demolition of the lower ground external rear wall, part of the lower ground spine wall and the construction of a new rear extension.

COOPER ASSOCIATES

Cooper Associates are a practice of Structural Engineers who have been operating in excess of 20 years. The increasing property values in London means that we have gained considerable experience in designing basement 'retrofits' formed by underpinning existing properties. We have also prepared many Basement Impact Statements and Construction Method Statements as part of planning applications, within the various London Boroughs. The Practise Director is a Chartered Civil and Structural Engineer.

PRE PLANNING WORK

A comprehensive Basement Impact Assessment and a Groundwater Assessment have been carried out by "Ground and Project Consultants Ltd" and by "H Fraser Consulting". Their reports (ref No 40278 and ref No 30277R1) accompany this document.

Cooper Associates carried out a visual inspection of the flat in August 2018. This involved an internal and external inspection of the building to review its current condition and its structural details. The property is structurally in a very good condition.

We saw no structural damage in our investigation.

Similar properties exist on both sides of number 9-10. An apple tree is in the immediate area at the rear. Two trial holes to the rear of the property had no evidence of roots and so we consider it unlikely that the basement excavation will find any significant roots (typically those over 12mm in diameter).

The tree will be protected throughout the works, by erecting a substantial hoarding around it, constructed in accordance with the guidelines given in BS5837:2005 – *Trees in relation to construction.*

- ***Programme for enabling works, construction and restoration***

Enabling works – Site setting up: fencing, lightning and security, waste disposal, services, storage, tree protection, diversion and/or disconnection of existing services

Construction: as per the Construction sequence: Existing rear external wall demolition, Existing spine wall demolition, Steelwork installation, New concrete retaining walls, Construction of the new rear extension

Construction: internal refurbishment, painting, connection of the new services.

- ***Construction Program***

Activity	**Duration (Weeks)**	
Site set-up	1	Jan 19
Demolition/Excavation/ Substructure	6	Jan-Feb 19
Superstructure	8	March-Apr 19
Fit out	8	Apr-May 19
Site Clean Up / Landscaping	3	Jun 19
Total Works	**26**	

- ***Identification of potential risks to land stability (including surrounding structures and infrastructure)***

There will be no potential risks to the development or to the adjacent structures and infrastructures due to land stability. The method of construction avoids any risk of slope instability within the site, as a hit and miss sequence for the excavation and the construction of the retaining walls is proposed. 1000mm long bays will be excavated at one time and the RC walls will be propped horizontally during the construction phase.

- ***Assessment of impact of potential risks on neighbouring properties***

Please note that this list only contains the risks that are considered to be exceptional in this project. Any normal construction hazards/risks expected on construction sites are not included. It is understood and expected that the appointed competent contractor would be aware of the risks/hazards associated with normal construction activities.

Potential Risk	Likelihood	Severity	Suggested Risk Reduction Actions
Damage to the property due to underpinning	Medium	Low	Use a hit and miss sequence and monitoring during the works
Building instability due to the underpinning	Low	High	Use a hit and miss sequence and provide temporary propping
Building instability due to rear/spine wall removal	Low	High	Provide Temporary work support as per CSM
Noise, vibration and dust	Medium	Low	Contractor to apply H&S procedures and good workmanship practice
Damage to external finishes	Low	Low	Contractor to apply H&S procedures and good workmanship practice

- ***Construction Sequence Methodology (CSM)***

It is proposed to carry out a complete refurbishment of the above flat, comprising the demolition of the lower ground external rear wall, part of the lower ground spine wall and the construction of a new rear extension. The new extension will be approximately 3.0m deep, measured from the existing rear wall. Two new concrete retaining walls will be built to retain the existing rear garden and to provide lateral stability to the adjoining property.

The existing superstructure is constructed from suspended concrete slabs and loadbearing brick walls.

- ***General Scope***

Materials and debris present within the structures will consist predominantly of bricks. The contractor is to ensure that they are safely removed from the site and that they are disposed of at an appropriate licenced waste site.

When using access towers, the contractor is to ensure that they are adequately braced and in accordance with *BS EN1004* requirements. The first lift may be up to 2.5 m high if necessary for access below the scaffold otherwise the lift heights should, in general, be at 2m intervals. Platforms should be no further than 150 mm from the working edge, and should be provided with guard rails and toe boards as required.

Cooper Associates does not take any responsibility for the demolition sequence should the contractors work vary from what has been described below. It is the contractors' obligation to contact Cooper Associates should there be any doubts over the demolition sequence or the drawings.

- ***Preparation works before the structural demolition.***

All workers employed to work on the demolition should be briefed on the project and be informed of the potential hazards by attending site induction sessions.

All equipment must be tested and properly stored and maintained over the duration of the demolition.

Protective screen covers will be placed, where necessary, to prevent flying pieces. They will be secured to the scaffolding both vertically and horizontally.

At all times during the demolition works, the demolition is to be supervised by a competent supervisor appointed by the main contractor.

The perimeter of the defined demolition zone will be fenced and sign posted to prevent unauthorised access. Access points will be established and only worker(s) who have been inducted with the authority of the Project Manager or supervisor may enter these zones.

It is expected that due to the size of the structural elements, manual handling methods will be used.

- ***Construction sequence***

Engineer to inspect prior to commencing works. Temporary supports to be agreed with engineer on site.

- ***Existing rear external wall demolition***

A) Prop the concrete slab on heavy duty slimshore to an agreed sequence.
B) Prop the rear wall on supporting 203UC needles to the floor above and on heavy duty slimshore props. Pack between needles and the floor. Cast external pad foundation to support the props. The position of the needles to be agreed with Engineers before any demolition.
C) Carefully remove existing external wall.

- ***Existing spine wall demolition***

A) Prop the concrete slabs using heavy duty slimshore.
B) Prop the spine wall on supporting 203UC needles to the floor above. Pack between needles and the floor.
C) Carefully remove the existing spine wall. The position of the needles to be agreed with Engineers before any demolition

- ***Steelwork installation***

A) Install the rear frame steels.
B) Install the spine wall supporting beam and connect it into the rear steel frame.
C) Remove the propping.

- ***New concrete retaining walls***

Following the underpin sequence, based on a 1:3:5:2:4 hit and miss construction sequence, dig and cast the new retaining walls.

Individually, a void for a section of wall will be excavated; a maximum of 1000 wide and reinforcement will be installed. Reinforcing starter bars will be driven into the ground on each side. Shutters will be constructed to retain the wet concrete. Once the concrete is cast, leaving a 50 mm gap between the top of the concrete and the underside of the cleaned brick footing, the gap will be drypacked, but only after a minimum of 24 hours has been allowed for the concrete to cure. The packing will act as structural concrete and will be a good quality 1:3 cements:sand hand damp mix, well rammed into position with a caulker.

48 hours must elapse before any further excavation can be carried out, within two bays of this new footing. A limit of 20% of the building can be undermined at any one time.

- ***Construction of the new rear extension***

A) Build the new rear extension, using scaffolding or access towers as appropriate.

o ***Proposals for monitoring during construction***

Damage or the potential for damage will be monitored on a daily basis by visual inspection of the existing walls of the property.

In the event that any fresh damage is noted or that the neighbours report any damage, structural work on site will be suspended temporarily until the damage is inspected. Once the magnitude and source of the damage has been established, a decision will be made on if to recommence the work or to suspend the work, until further preventative methods are agreed to mitigate any additional movement.

If the cracking is beyond trivial (hairline, caused by obvious reasons) then the decision to start work again will be made in conjunction with the Party Wall Surveyors and the Structural Engineers.

- ***Confirmatory and reasoned statement identifying likely damage to nearby properties according to Burland Scale***

When underpinning, it is inevitable that the ground will be unsupported or only partially supported for a short period during the excavation of each pin. This means that the behaviour of the ground will depend on the quality of workmanship and suitability of the methods used. However, provided that the temporary support follows best practice, then extensive past experience has shown that the bulk movements of the ground alongside the basement caused by underpinning for a single storey basement (typical depth 3.5m) should not exceed 5mm in either horizontal or vertical directions.

Due to the limited extent of the proposed new rear extension and the underpinning for the retaining wall (approximately 2.0m long) and due to the shallow excavation involved (approximately 1.5m depth), we consider it reasonable to state that the neighbours' properties will have a risk of only limited damage, corresponding to "very slight damage - category 1" of the Burland Scale (given a normal degree of care by the appointed Contractor). This is defined as "fine cracks which are easily treated during normal decoration" and forms part of the BRT Digest 251.

A Party Wall Award will be in place before the works commence due to three metre notice requirements. This will record any existing damage and will identify any fresh damage, in the event that any did occur.

- ***Confirmatory and reasoned statement with supporting evidence that the structural stability of the building and neighbouring properties will be maintained (by reference to BIA, Ground Movement Assessment and Construction Sequence Methodology), including consideration of cumulative effects***

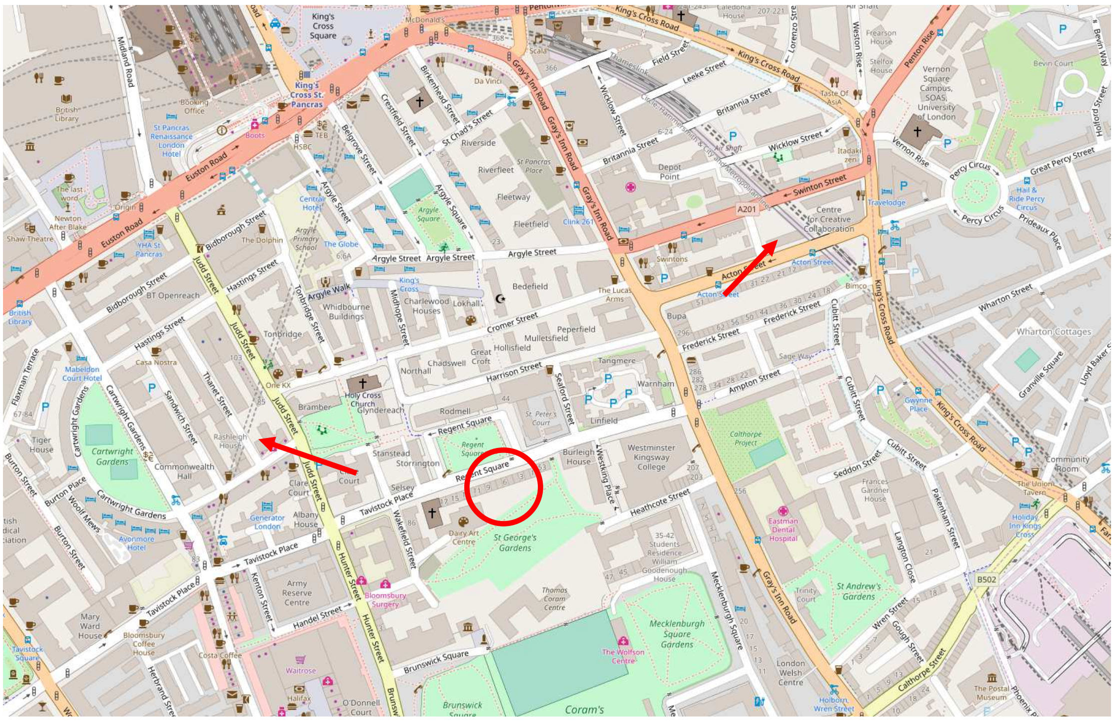
The structural stability of the buildings and neighbouring properties will be maintained. The new proposed concrete retaining walls, excavated and casted with a hit and miss sequence, will provide the required lateral stability to the neighbouring properties. Temporary works will maintain the stability of the building during the construction stages. Moreover the rear steel box frame will guarantee the stability of the property after the rear and spine walls have been removed.

- ***<u>Confirmatory and reasoned statement with supporting evidence that there will be no adverse effects on drainage or run-off and no damage to the water environment (by reference to ground investigation, BIA and CSM), including consideration of cumulative effects</u>***

There will be no adverse effect on drainage or run-off. Any utilities and other infrastructure immediately adjacent to or through the construction will be exposed, adequately supported and be reinstated (using appropriate specialist subcontractors where necessary) as part of the works. There will be no damage at the water environment as stated in the attached Groundwater Assessment. The new roof and hard standing will be adequately drained to the existing system.

- ***<u>Transport assets</u>***

No underground lines are within the proximity of the site. Red arrows indicate the nearest transport lines.



OpenStreet map

- ***Pictures:***

Existing Party Wall

View of the party wall and the neighbouring property (to the left)

- ***Attachments:***

1. CA5591/01 to CA5591/05 – Structural Drawings
2. CA5591/T01 – Typical Temporary Works Details
3. CA5591/C01 - RC Retaining wall calculations

o ***CA5591.C01***

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.06

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **1500** mm
Stem thickness;	t_{stem} = **200** 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} = **200** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1500** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **0** mm

Retained soil properties

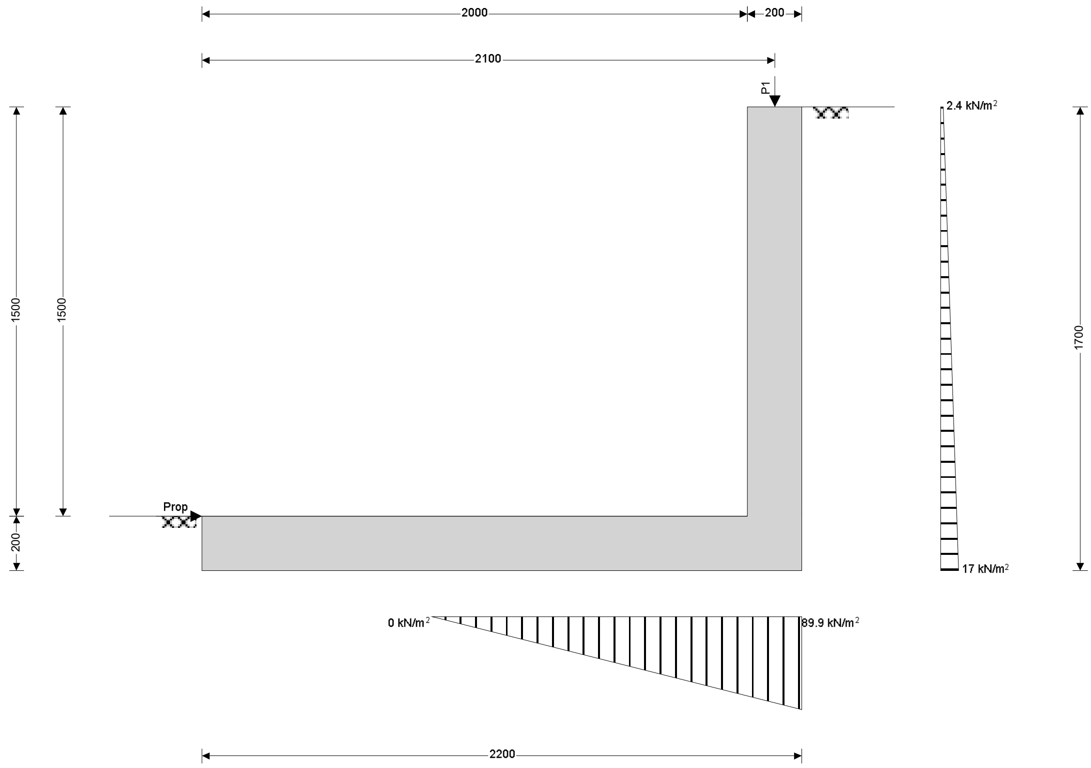
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **5** kN/m^2
Vertical line load at 2100 mm;	P_{G1} = **39** kN/m
;	P_{Q1} = **3.5** kN/m



General arrangement

Calculate retaining wall geometry

Base length; $l_{base} = l_{toe} + t_{stem} =$ **2200** mm

Moist soil height; $h_{moist} = h_{soil} =$ **1500** mm

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

- Distance to vertical component; $x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **2200** mm

Effective height of wall; $h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **1700** mm

- Distance to horizontal component; $x_{sur\_h} = h_{eff} / 2 =$ **850** mm

Area of wall stem; $A_{stem} = h_{stem} \times t_{stem} =$ **0.3** m^2

- Distance to vertical component; $x_{stem} = l_{toe} + t_{stem} / 2 =$ **2100** mm

Area of wall base; $A_{base} = l_{base} \times t_{base} =$ **0.44** m^2

- Distance to vertical component; $x_{base} = l_{base} / 2 =$ **1100** mm

Using Coulomb theory

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

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

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} =$ **7.5** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} =$ **11** kN/m

Line loads; $F_{P\_v} = P_{G1} + P_{Q1} =$ **42.5** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} =$ **61** kN/m

Horizontal forces on wall

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

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times h_{eff}^2 / 2$ = **12.4** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-0.8** kN/m

Total; $F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{pass\_h}$ = **15.6** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = **15.8** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = **12.1** kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-3.4** kNm/m

Line loads; $M_P = (P_{G1} + P_{Q1}) \times p_1$ = **89.3** kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **-7** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist}$ = **106.6** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h}$ = **15.6** kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v}$ = **1748** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = **648** mm

Loaded length of base; $l_{load} = 3 \times (l_{base} - \bar{x})$ = **1356** mm

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

Bearing pressure at heel; $q_{heel} = 2 \times F_{total\_v} / l_{load}$ = **89.9** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.112**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>**RETAINING WALL DESIGN**</u>

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

Tedds calculation version 2.9.06

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

Concrete strength class; C30/37

Characteristic compressive cylinder strength; f_{ck} = **30** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **37** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **38** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **2.9** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.0** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **32837** N/mm^2

Partial factor for concrete - Table 2.1N; γ_C = **1.50**

Compressive strength coefficient - cl.3.1.6(1); α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **17.0** N/mm^2

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

Ultimate strain - Table 3.1; ε_{cu2} = **0.0035**

Shortening strain - Table 3.1; ε_{cu3} = **0.0035**

Effective compression zone height factor; λ = **0.80**

Effective strength factor; η = **1.00**

Bending coefficient k_1; K_1 = **0.40**

Bending coefficient k_2; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

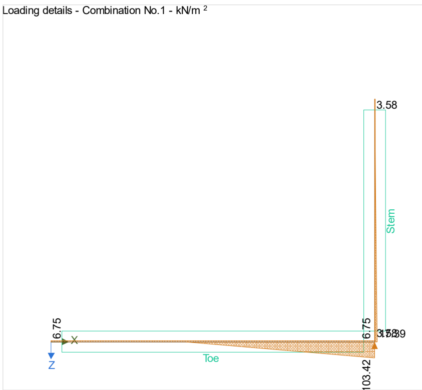
Bending coefficient k_3;	K_3 =**0.40**
Bending coefficient k_4;	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

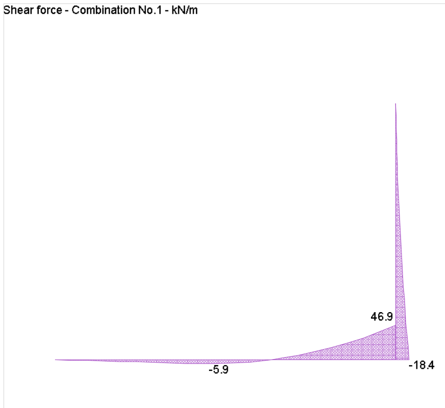
Reinforcement details

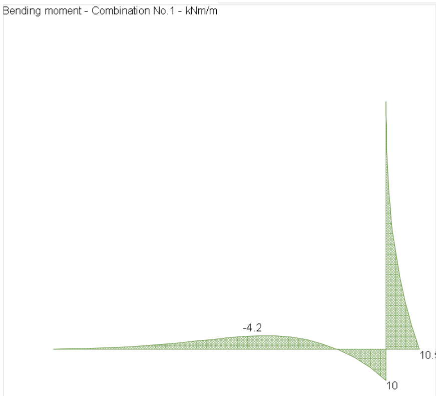
Characteristic yield strength of reinforcement;	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement;	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N;	γ_S = **1.15**
Design yield strength of reinforcement;	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

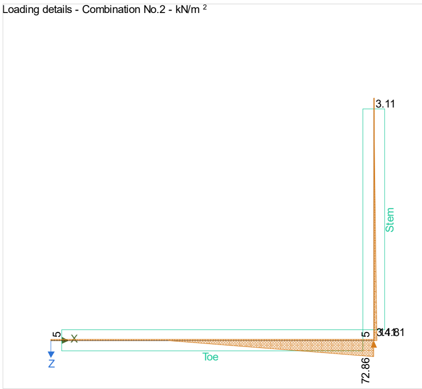
Cover to reinforcement

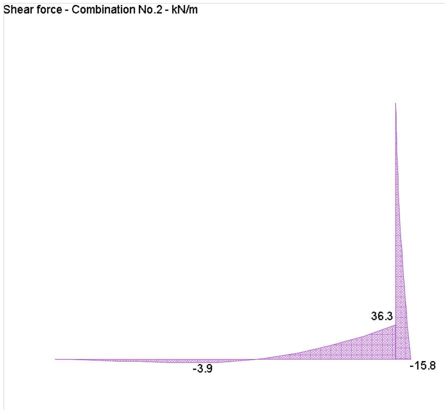
Front face of stem;	c_{sf} = **40** mm
Rear face of stem;	c_{sr} = **50** mm
Top face of base;	c_{bt} = **50** mm
Bottom face of base;	c_{bb} = **75** mm

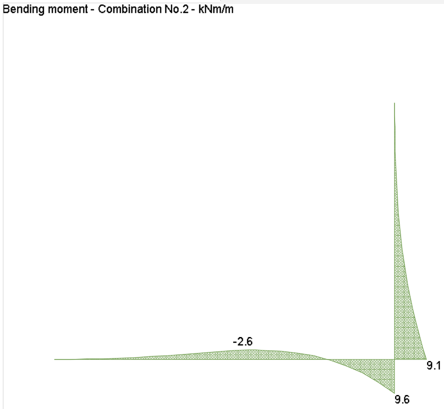












Check stem design at base of stem

Depth of section; $h = \mathbf{200}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{10.5}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{144}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.017}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{137}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{18}$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{177}$ mm²/m

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

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

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{217}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = \mathbf{8000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.384}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

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

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = \mathbf{0.001}$

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

Structural system factor - Table 7.4N; $K_b = \mathbf{0.4}$

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

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b)$ = **16**

Actual span to depth ratio; h_{stem} / d = **10.4**

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 = **0.6**

Serviceability bending moment; M_{sls} = **6.4** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **83.3** N/mm^2

Load duration; Long term

Load duration factor; k_t = **0.4**

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

$A_{c.eff}$ = **60667** mm^2/m

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

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.009**

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **6.091**

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; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **389**
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 = **0.097** mm

w_k / w_{max} = **0.324**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **18.4** kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **2.000**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.542**
N/mm^2

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

$V_{Rd.c}$ = **78.6** kN/m

$V / V_{Rd.c}$ = **0.234**

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); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **200**
mm^2/m

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

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

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm^2/m

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

Check base design at toe

Depth of section; $h = \mathbf{200}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{10}$ kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{119}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.024}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{113}$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = \mathbf{15}$ mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{204}$ mm^2/m

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

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

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{179}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = \mathbf{8000}$ mm^2/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.361}$

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

Library item: Rectangular single output

Crack control - Section 7.3

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

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

Serviceability bending moment; $M_{sls} = \mathbf{7.1}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{110.9}$ N/mm^2

Load duration; Long term

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

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

$A_{c.eff} = \mathbf{61708}$ mm^2/m

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

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.009}$

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

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

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

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{478}$ mm

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

$w_k = \mathbf{0.159}$ mm

$w_k / w_{max} = \mathbf{0.53}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = \mathbf{46.9}$ kN/m

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

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2) = \mathbf{2.000}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.005}$

v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.542** N/mm^2

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

$V_{Rd.c}$ = **69.3** kN/m

V / $V_{Rd.c}$ = **0.678**

PASS - Design shear resistance exceeds design shear force

Check base design at toe

Depth of section; h = **200** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **4.2** kNm/m

Depth to tension reinforcement; d = h - c_{bt} - ϕ_{bt} / 2 = **144** mm

K = M / (d^2 × f_{ck}) = **0.007**

K' = (2 × η × α_{cc}/γ_C)×(1 - λ × (δ - K_1)/(2 × K_2))×(λ × (δ - K_1)/(2 × K_2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5 + 0.5 × (1 - 2 × K / (η × α_{cc} / γ_C))$^{0.5}$, 0.95) × d = **137** mm

Depth of neutral axis; x = 2.5 × (d – z) = **18** mm

Area of tension reinforcement required; $A_{bt.req}$ = M / (f_{yd} × z) = **71** mm^2/m

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

Area of tension reinforcement provided; $A_{bt.prov}$ = π × ϕ_{bt}^2 / (4 × s_{bt}) = **565** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{bt.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **217** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bt.max}$ = 0.04 × h = **8000** mm^2/m

max($A_{bt.req}$, $A_{bt.min}$) / $A_{bt.prov}$ = **0.384**

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

Library item: Rectangular single output

Crack control - Section 7.3

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

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **0** kNm/m

Tensile stress in reinforcement; σ_s = M_{sls} / ($A_{bt.prov}$ × z) = **0** 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 × (h - d), (h - x) / 3, h / 2)

$A_{c.eff}$ = **60667** mm^2/m

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

Reinforcement ratio; $\rho_{p.eff}$ = $A_{bt.prov}$ / $A_{c.eff}$ = **0.009**

Modular ratio; α_e = E_s / E_{cm} = **6.091**

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 × c_{bt} + k_1 × k_2 × k_4 × ϕ_{bt} / $\rho_{p.eff}$ = **389** mm

Maximum crack width - exp.7.8; w_k = $s_{r.max}$ × max(σ_s – k_t × ($f_{ct.eff}$ / $\rho_{p.eff}$) × (1 + α_e × $\rho_{p.eff}$), 0.6 × σ_s) / E_s

w_k = **0** mm

$w_k / w_{max} = \mathbf{0}$

PASS - Maximum crack width is less than limiting crack width

Secondary transverse reinforcement to base - Section 9.3

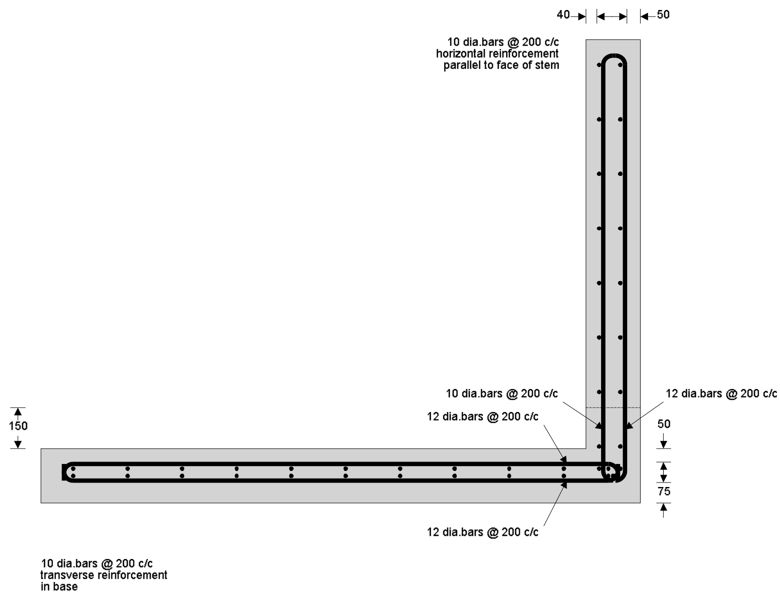
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{113}$ mm^2/m

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; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{393}$ mm^2/m

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



Reinforcement details