

Structural Engineers Report For:

Peacocks, 6 Queen Street, Neath , SA11 1DL

Pre-Demolition Survey

Prepared for:

PEACOCKS

REF: 19101



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Document Control

Project	Demolition rear section of Peacocks Neath Pre-Demolition Survey.
Client	Peacocks
Vale Consultancy Ref:	19101

Document Checking:

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1.0 Introduction

- 1.1 Vale Consultancy were appointed by Peacocks, to undertake a pre-demolition survey of the rear section of 6 Queen Street, Neath SA11 1DL.
- 1.2 The site is proposed to have a damaged rear section removed and bricked up and rendered.
- 1.3 The information within this report has been put together via a pre-demolition survey of the existing building and its surrounding area.
- 1.4 This specification is to be read in conjunction with the Employers Requirements, Pre-Construction Health and Safety Information and all other relevant information and drawings.

2.0 The Site

- 2.1 The Site can be located from National Grid Reference SS 75287 97453
- 2.2 The front of the existing building can be accessed via Queen Street and the rear of the building is located inside of a private carpark accessed from Queen street back road.
- 2.3 The existing building is a 3 storey building used as an active Peacocks retail store. On the rear of the building is a single storey annex building. This is freestanding of the structure but post built onto the rear and accessed via the main building. The roof of the main building is flat and with masonry parapets of roughly 1m high with concrete coping stones & internally clad with Asbestos panels.
- 2.4 The section as shown on **Appendix A** in red is to be demolished as well as the side annex building shown in blue, green indicates a void. **Appendix B** shows photographs of these key areas.

3.0 Report Scope

- 3.1 The purpose of this report is to prepare the site adequately prior to the blockwork wall to be constructed at the rear. The works required include the following;
 - Analysis of all sub level services, quote provided
 - Asbestos survey of roof to be completed
 - Removal of all asbestos containing material on roof to a properly licensed receiver
 - Demolition of buildings and structures without damaging supporting sub structure
 - Breaking of all tarmac, bituminous or concrete hard standing areas as required at rear for assessment of current foundations. It is recommended that all suitable clean granular material is to be crushed to conform to a 6F2 Classification in accordance with the specification for Highway Works and to be left on site for re-use.
 - All waste material from the above demolition is required to be removed from site
 - Design of either foundation strip pad or R.C ground beam resting on current pads as to bear load from newly proposed blockwork wall at rear.

3.2 This pre-demolition report is based on visual inspections of the building, internally and externally to determine the forms of construction, areas of the building which are to be retained and parts of the existing structure which need to be removed and replaced. This report highlights any potential issues which may arise during the demolition. The report should be reviewed prior to demolition, and any variation in construction from that concluded from the survey should be assessed by the Demolition Contractor, also referring any issues back to the Project Structural Engineer, if further assessment is deemed to be required.

4.0 General Notes

4.1 Health & Safety and CDM Regulations

- 4.1.1 The contractor is required to conform to the CDM Regulations 2015
- 4.1.2 In accordance with the CDM Regulations, no construction work will be allowed to commence on site until the contractor has prepared an appropriate Construction Phase Plan containing suitable method statements and risk assessments
- 4.1.3 The contractor is responsible for erecting a site hoarding to the perimeter of the site and for maintaining site security
- 4.1.4 All existing services shall be identified, located and isolated/removed prior to commencement of the works
- 4.1.5 All existing services which are to remain shall be clearly identified on site and shall be surveyed by the contractor once exposed.
- 4.1.6 The site is adjacent to a number of existing properties. The contractor is be especially mindful of these adjacent properties and take all necessary precautions to ensure that levels of dust, noise and vibration are not problematic and do not cause a nuisance.

4.2 Temporary supports and Structures

- 4.2.1 The Contractor shall provide all necessary temporary supports during demolition and these shall be designed by the Contractor via a competent specialist temporary works designer.

4.3 Licenses

- 4.3.1 The Contractor is required to obtain all necessary licenses and documentation relating to the demolition, and transfer of materials from site.

4.4 Hoarding

- 4.4.1 A suitable site hoarding shall be provided by the Contractor both during and post-demolition to ensure that site security is maintained. Refer to Employers Requirements.
- 4.4.2 The Contractor is required to maintain safe access around the perimeter of the demolition site for uninterrupted pedestrian and vehicular access.

5.0 Report Limitations

- 5.1 This report assesses structural building elements only, based on a visual inspection. Slab and foundation format should be established following a 2 stage demolition of the super structure
- 5.2 The existing mechanical and electrical installation with the building and wider site are outside the scope of this report. All services should be identified and isolated
- 5.3 An independent Asbestos survey should be undertaken, the finding so which should also be reviewed prior to demolition commencing.
- 5.4 A underground utility services assessment should be undertaken prior to demolition, quote provided.

6.0 Pre-demolition Assessment

- 6.1 As noted above, the survey which has been undertaken was to assess the existing structure and its type of construction, to enable the appointed specialist and competent demolition contractor to develop a method statement for the demolition of the building. Please see **Appendix A** which highlights the existing areas of the building which are to be demolished. **Appendix B** shows photographs of key areas which are to be demolished.
- 6.2 **The Main Building** – From the survey it is likely that the main building has a brickwork inner skin, small cavity and then an outer brickwork skin rendered with a concrete screed. The roof is flat in shape with roof access possible the main roof build up is presumed to be similar of that to the lower building which by inspection has been determined to be formed of a waterproof rubber membrane over plyboard sheets supported by timber joists . The floors throughout the building are wooden boards which would be presumed as to be bearing onto joists. The ground slab is assumed to be bearing onto standard pads foundations below the steel frame columns..
- 6.3 **Rear Annex** – The rear annex are assumed to be constructed with a brickwork inner skin, small cavity similar to that of the main building from external inspection.

7.0 Demolishing Structures – General Requirements

- 7.1 Before starting work, examine all available information, carry out a survey of the underground services and asbestos on roof, the overall site and surrounding carpark at rear, following submit the survey report and method statement to the project manager

covering all relevant matters listed below and in the Health and Safety Executive Guidance Note GS29/1 paragraph 32:

The form, condition and demolition methods of the structures.

The form, location and removal methods of any toxic or hazardous materials.

The type and location of adjoining or surrounding premises which may be adversely affected by noise, vibration, dust or removal of structure

The identification and location of services above and below ground

As applicable, party wall notices and relevant approvals

- 7.2 Any work carried out to or which affects new or existing services must be in accordance with the byelaws or regulations of the relevant statutory authority.
- 7.3 Operatives must be suitably skilled and experienced for the type of work and hold or be training to obtain relevant CITB Certificates of Competence.
- 7.4 When demolishing structures against adjoining/adjacent properties leave adequate temporary support and protection at each stage and arrange for inspection by the Supervisor. Leave partly demolished structures in a stable condition with adequate temporary propping to prevent the risk of uncontrolled collapse. Structures should be demolished from the top downwards which will again prevent the risk of any uncontrolled collapse.
- 7.5 Suitable scaffolding structures should be installed to allow easy access to areas at higher level. Prevent overloading of the scaffolding platforms from debris.
- 7.6 Do not demolish and excavate below the existing foundations of the property which are being retained or have not yet been demolished to avoid undermining the existing foundations of the building.
- 7.7 Prevent access of unauthorized persons to partly demolished structures. No person should be working alone on site.
- 7.8 It is expected that the to be demolished superstructure, substructure, building drainage and below ground services will be demolished/grubbed up and any excavations back filled, with engineering material, laid and compacted in controlled layers. Any cohesive arising that remains should be agreed if to be left on site and should be stockpiled and dressed to prevent excessive saturation of the material. Clean building rubble can be crushed, graded and retained on site for back fill where appropriate, and with agreement of the client.

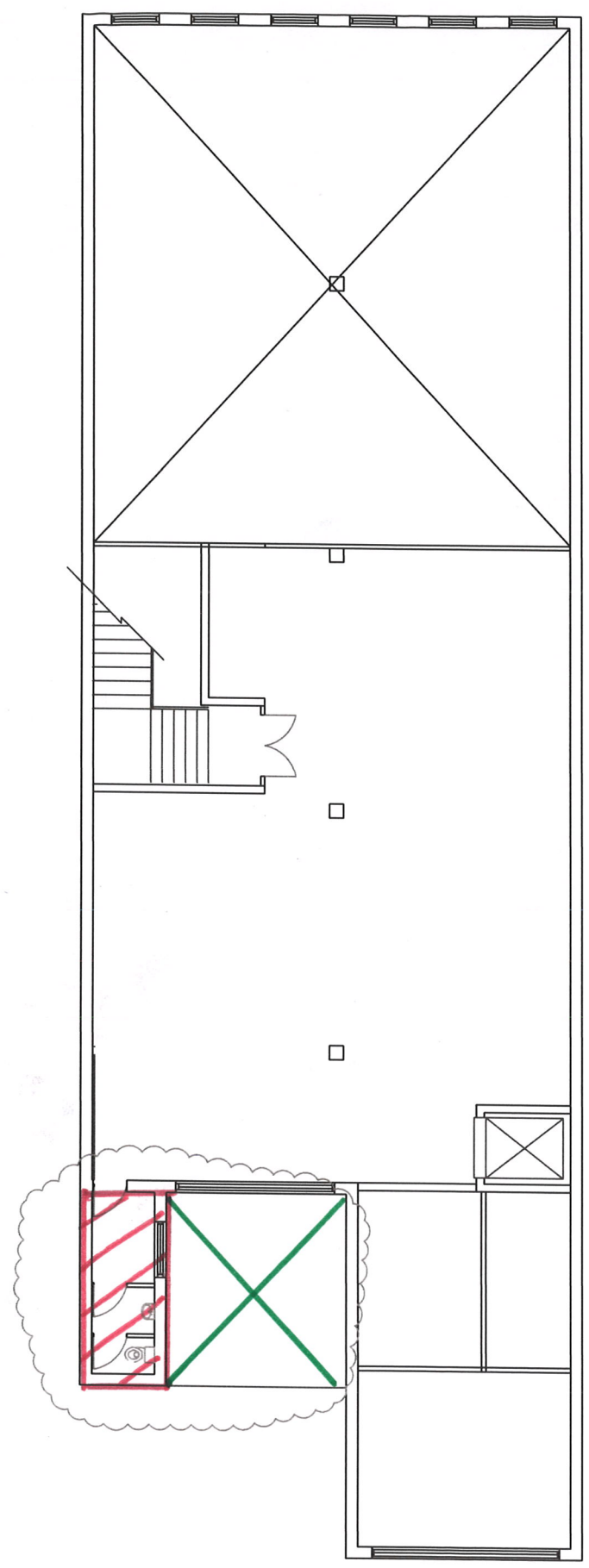
8.0 Conclusion

- 8.1 The building is located on an open site with no particular spatial constraints for access to undertake demolition, the rear private carpark which would need to be un-occupied during demolition which would need to be agreed with the owners/ users and closed off during the demolition process.

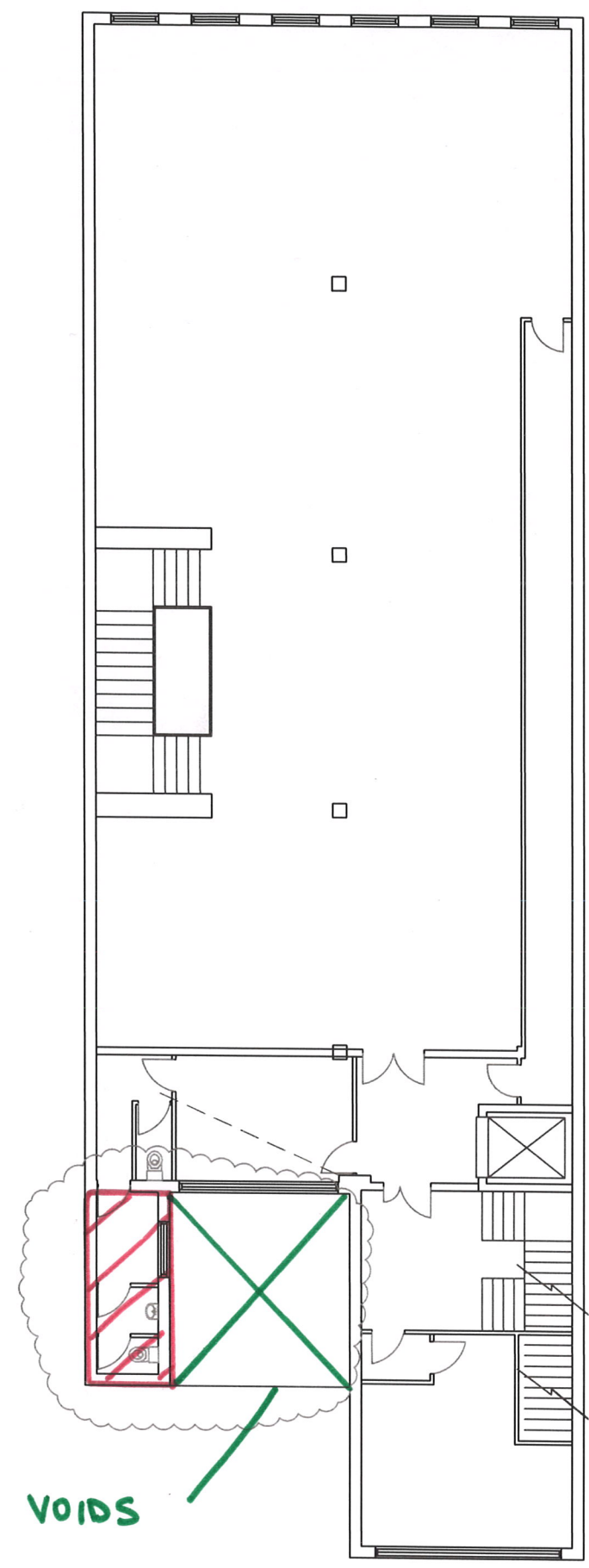
- 8.2 The building superstructure form of construction is a steel frame structure composed of vertical steel columns and I beams spanning laterally in both directions. From visual inspection there is a steel column which is encapsulated in the wall between the proposed demolition section and the retained structure. This would indicate that the rear section is independently stable of the front section which would be retained. We would require the floor/ceiling of the proposed demolished area to be exposed as to allow for the engineer to verify that the steel can be removed without having any adverse effect on the retained structure, and to ascertain the steel members spanning laterally which would become loaded by the new blockwork wall are suitable for the additional load. During initial demolition caution would be advised on carefully removing the steel sections to the rear as to not damage the vertical steels which form the structural component of the retained section.
- 8.3 The building sub substructure cannot be assessed at this stage but it is presumed that pad foundations would be located under the concrete columns. However, during the initial demolition, the form of construction should be confirmed by digging down in the stage 1 demolished area, this would need confirming as possible on the services plan. A structural engineer site visit would be required as to ascertain the structural stability of these foundations post stage 1 demolition. If it is deemed these are sound the wall could be construed as to bear current foundations. If not possible option 2: a R.C ground beam bearing on the 2 concrete pads would be the ideal solution to take vertical loading from the blockwork detailed in **Appendix C**.
- 8.4 The appointed demolition contractor should develop a method statement for the demolition of the building in line with the demolition schedule detailed in **Appendix D**, and reinstatement of any ground following the removal of substructure and below ground foundations. Prior to demolition, the type and location of any materials proposed to be left on site post demolition should be agreed in advance with the Client.
- 8.5 Ideally the roof would be stripped back and maintained in the retained section of the building. Demolition contractor would need to be careful in removal of the section as there is no visible damage on the roof caused by any movement. Following removal assessment of joist buildup would need to be carried out by engineer as to determine as to method of restraining the roof edge to the newly proposed exterior wall.

Appendix A

Site Plan and Areas to be Demolished and plan of Proposal



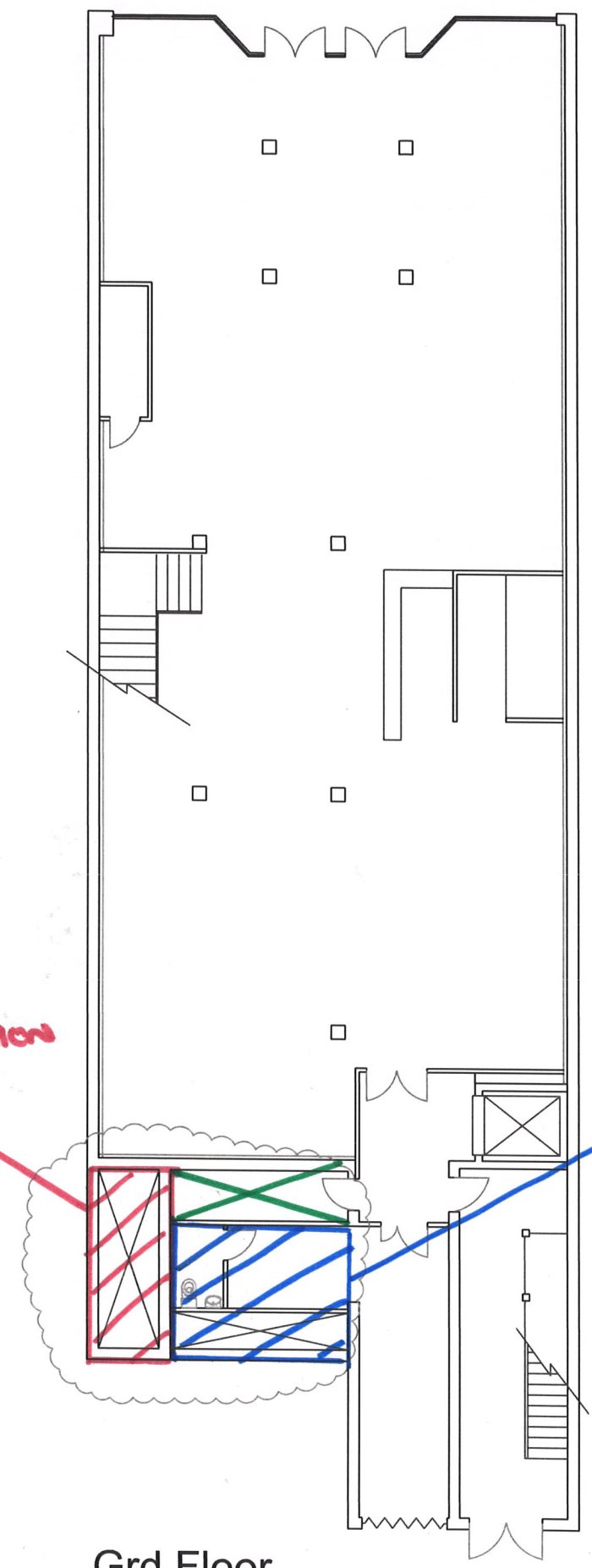
2nd Floor



VOIDS

1st Floor

3 FLOOR DEMOLITION
SECTION

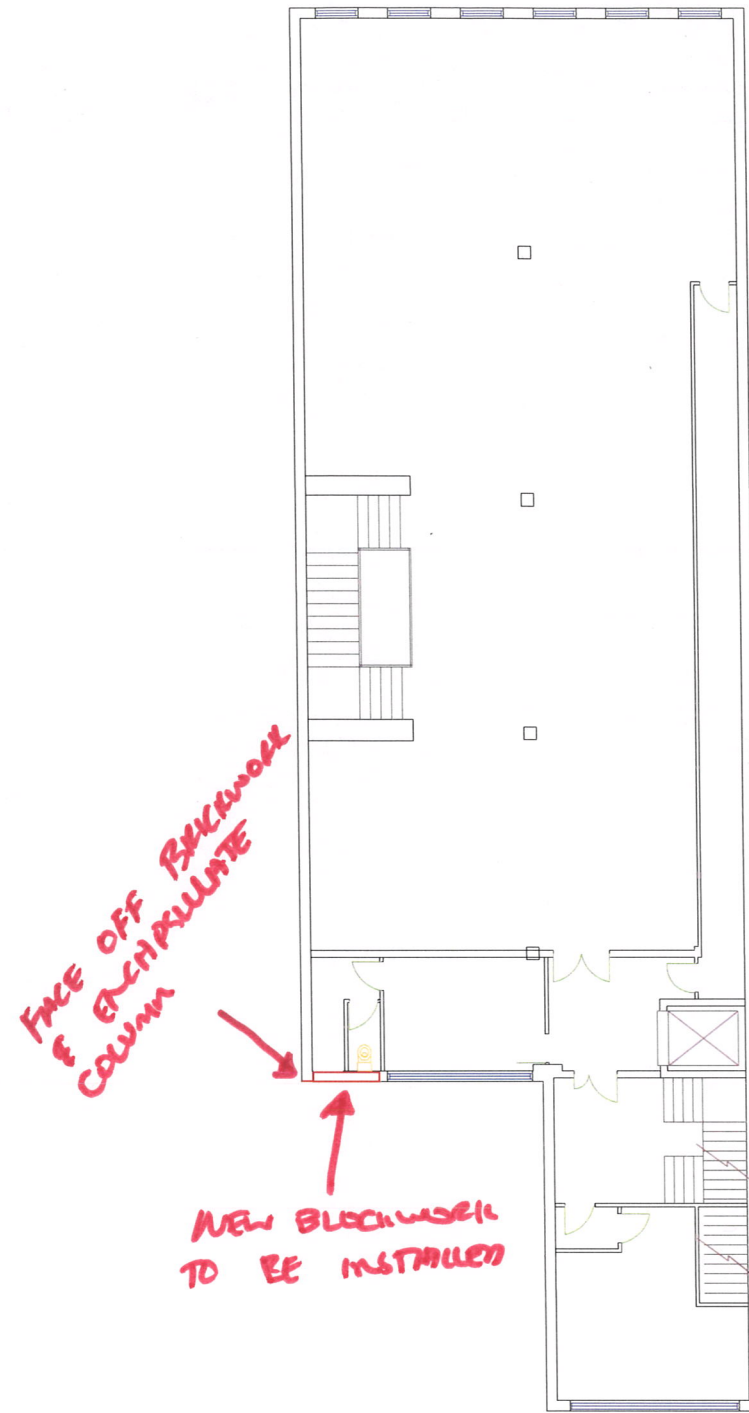


GROUND FL
DEMOLITION

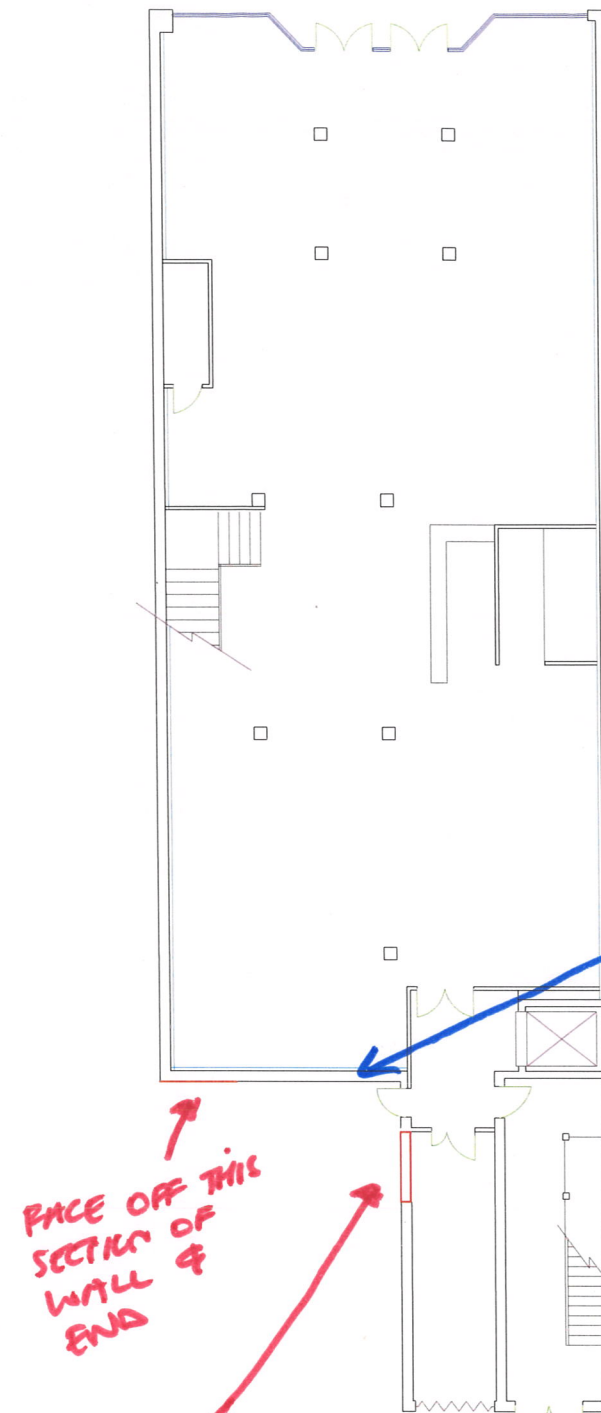
Grd Floor



2nd Floor



1st Floor



0rd Floor

NEW BLOCKWORK TO
 BE INSTALLED IN
 THE WALKTHROUGH
 IN NON STRUCTURAL FILLING
 IN

RE-RENDER WHOLE
 REAR OF THIS
 SECTION INCLUDING
 NEW WALL SECTIONS
 & CURRENT RETAINED
 SECTION, ON ALL FLOORS-

Appendix B

Photographs of Key Areas



Photograph 1 – Rear of Peacocks Neath



Photograph 2 – Left side showing visible crack where building has detached



Photograph 3 – Formation of flat roof



Photograph 4 –Column encapsulated in wall between proposed demolished area and retained area



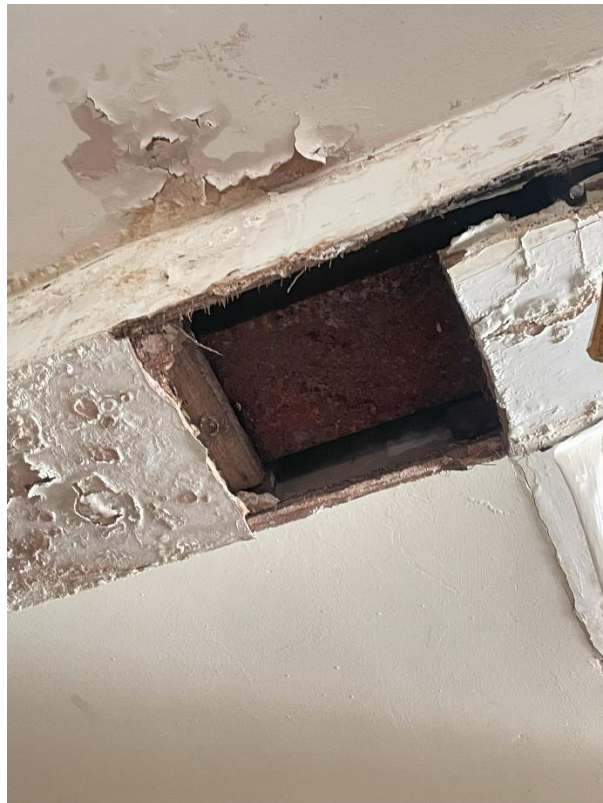
Photograph 5 – Visible crack width at top floor



Photograph 6 –Freestanding lower rear section



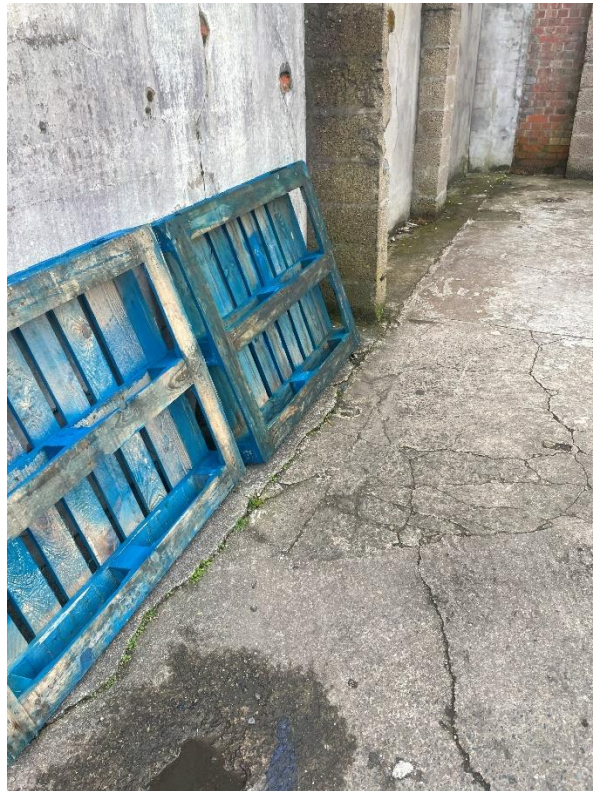
Photograph 7 –Exposed section of steelwork beam spanning laterally



Photograph 8 –Exposed section of steelwork beam spanning longitudinally



Photograph 9 –Lower rear section to be removed



Photograph 10 –Exterior ground showing no sign of dipping indicative of no soil movement issues



Photograph 11 –Location of manhole at rear



Photograph 12 –Asbestos on roof

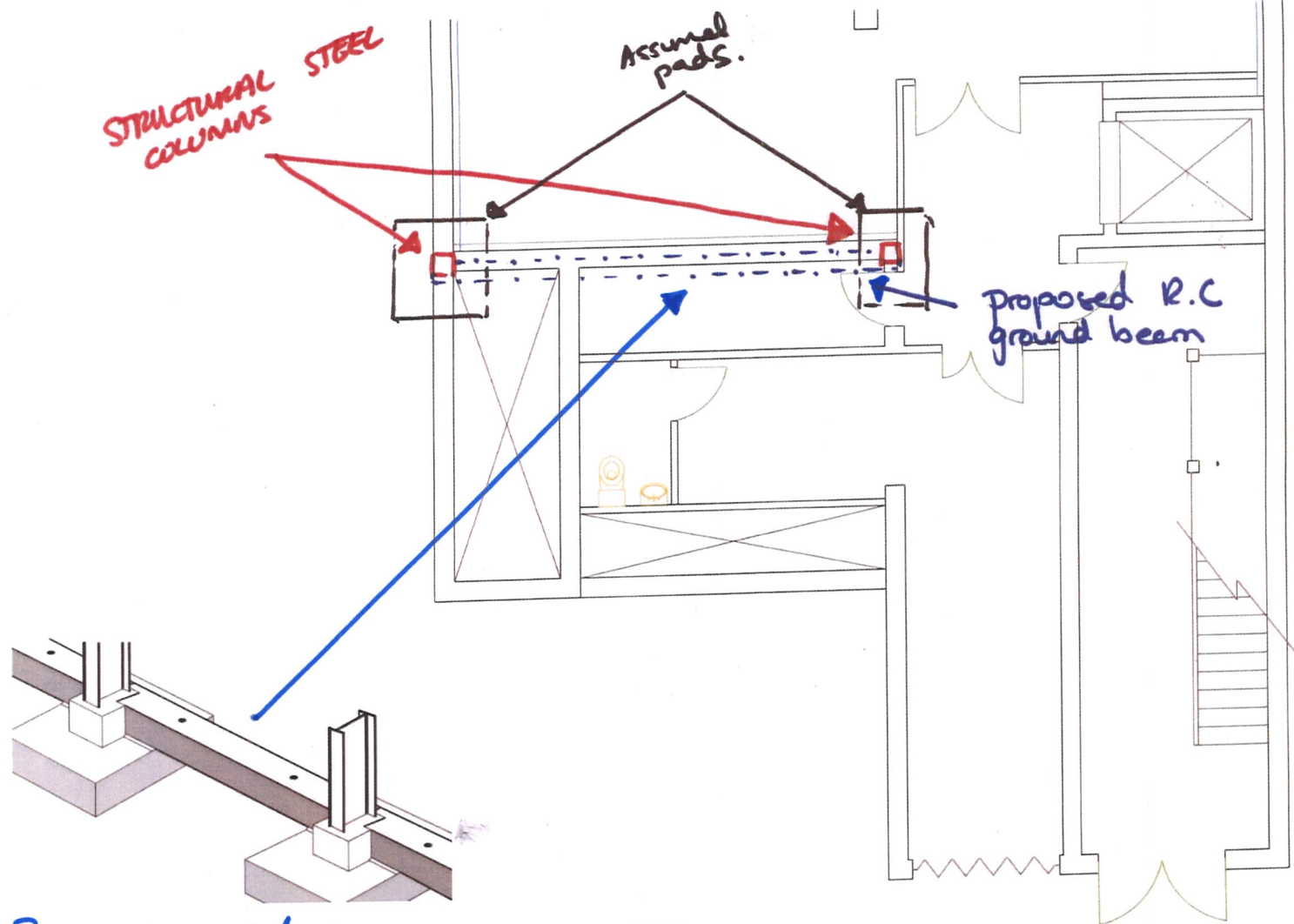
Appendix C

Preliminary R.C Beam design

These are preliminary designs for the blockwork type and R.C ground beam, which may be subject to site analysis of the foundation formation detailed in appendix E.

The blockwork would be tied back to the structural steels using Ancon ties. These would be suitable type is to be specified during phase 2 design.

VALE CONSULTANCY - 19101
SK05 - PEACOCKS NEATH
R.C BEAM PLAN
NTS



Grd Floor

Beam supported on pads $\therefore$ none extensive excavation is required & can be precast & brought to site

 Vale Consultancy CONSULTING CIVIL & STRUCTURAL ENGINEERS 29 Bocam Park Old Field Road, Pencoed CF35 5LJ	<u>Peacocks pre-demolition</u>				Job Ref. 19101	
	Section Design loadings				Sheet no./rev. -	
	Calc. by GK	Date 26/03/2024	Chk'd by JM	Date 26/03/2024	App'd by JM	Date 26/03/2024

Proposed dwellings on former Cynlias School Playing Field

The design has been completed in accordance with British Standards BS6399, BS5628 (Masonry), BS5268 (Timber) & BS 5950 (Steel). All dimensions stated in the following calculations are for design purposes only and not for use for fabrication or construction purposes. All dimensions must be checked by a competent contractor prior to fabrication or construction and any discrepancies found are to be reported to the design engineer.

Design Loadings

1. Roof 35° Pitch (Max)

Dead Loading:	
Tiles	= 0.45 kN/m ²
Insulation, Felt & Battens	= 0.10 kN/m ²
Truss or Purlins/Rafters	= 0.20 kN/m ²
Plasterboard & skim	= 0.15 kN/m ²

	0.90 kN/m ² /cos(35) = 1.10kN/m ² Total Dead Loading
Imposed Loading:	
Snow	= 0.60 kN/m ² Total Live Loading

2. Roof Flat

Dead Loading:	
Waterproof Single Ply Membrane	= 0.10 kN/m ²
Plyboard	= 0.15 kN/m ²
Joists	= 0.15 kN/m ²
Insulation	= 0.05 kN/m ²
Plasterboard & Skim	= 0.15 kN/m ²

	0.60 kN/m ² Total Dead Loading
Imposed Loading:	
Snow	= 0.60 kN/m ² Total Live Loading

3. Walls

Dead Loading:	
Render	= 0.15 kN/m ²
200mm masonry	= 4.30kN/m ²
Plaster	= 0.15 kN/m ²

	4.60 kN/m ² Total Dead Loading
Stud	
Dead Loading:	
Plasterboard	= 0.15 kN/m ²
Render/Render Skin	= 0.20kN/m ²
Stud	= 0.06kN/m ²
OSB	= 0.12 kN/m ²

	0.53 kN/m ² Total Dead Loading

4. Attic / First Floor

Dead Loading:	
Boarding	= 0.15 kN/m ²
Insulation	= 0.05 kN/m ²
Joists	= 0.15 kN/m ²
Plaster and skim Ceiling	= 0.15 kN/m ²

	0.5 kN/m ² Total Dead Loading
Attic Storage Load	= 0.25 kN/m ²
Imposed Loading:	
Residential	= 1.50 kN/m ² Total Live Loading



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Project Peacocks Neath				Job no. 19101	
Calcs for R.C Ground Beam Option				Start page no./Revision 1 1	
Calcs by GK	Calcs date 22/03/2024	Checked by	Checked date	Approved by	Approved date

RC MEMBER ANALYSIS & DESIGN (EN1992-1-1:2004)

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

Tedds calculation version 3.3.10

Design summary

Span 1 moment design reinforcement (including crack requirements)

	Unit	Provided	Required	Utilisation	Result
Zone 1 top (-8.3 kNm)	mm ²	628	309	0.492	PASS (crack width)
Zone 1 bot (1.0 kNm)	mm ²	402	315	0.783	PASS (crack width)
Zone 2 top (0.0 kNm)	mm ²	628	0	0.000	PASS
Zone 2 bot (4.1 kNm)	mm ²	804	217	0.269	PASS (crack width)
Zone 3 top (-8.3 kNm)	mm ²	628	309	0.492	PASS (crack width)
Zone 3 bot (1.0 kNm)	mm ²	402	315	0.783	PASS (crack width)
Span to depth ratio		14.33	40.00	0.358	PASS

Span 1 shear design reinforcement

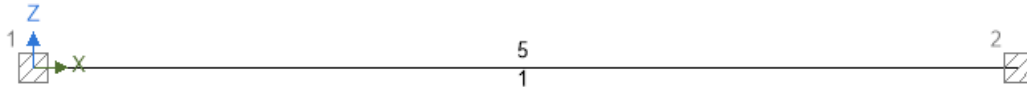
	Unit	Provided	Required	Utilisation	Result
Zone 1 (8.6 kN)	mm ² /m	402	284	0.706	PASS
Zone 2 (5.0 kN)	mm ² /m	402	284	0.706	PASS
Zone 3 (8.6 kN)	mm ² /m	402	284	0.706	PASS

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - Concrete (EC2 normal) - R 300x400



Span	Length (m)	Section	Start Support	End Support
1	5	R 300x400	Fixed	Fixed

R 300x400: Area 1200 cm², Inertia Major 160000 cm⁴, Inertia Minor 90000 cm⁴, Shear area parallel to Minor 1000 cm², Shear area parallel to Major = 1000 cm²

Concrete (EC2 normal): Density 2500 kg/m³, Youngs 34 kN/mm², Shear 14.2 kN/mm², Thermal 0.00001 °C⁻¹

Loading

Self weight included

Wall load live - Loading (kN/m)



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Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wall load live	wall load dead
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50		
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00		
1.0G + 1.0 ψ 2Q (Quasi)	1.00	1.00	0.30		

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Wall load live	UDL	GlobalZ	5.4 kN/m

Results

Total deflection

1.35G + 1.5Q + 1.5RQ (Strength) - Total deflection



1.0G + 1.0Q + 1.0RQ (Service) - Total deflection



1.0G + 1.0 ψ 2Q (Quasi) - Total deflection

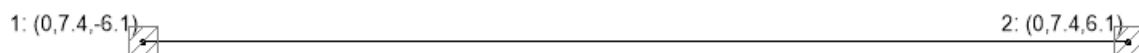


Total base reactions

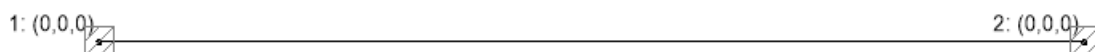
Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	19.9
1.0G + 1.0Q + 1.0RQ (Service)	0	14.7
1.0G + 1.0 ψ 2Q (Quasi)	0	14.7

Reactions

Self Weight - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Permanent - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



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Imposed - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



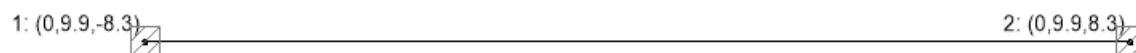
Wall load live - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



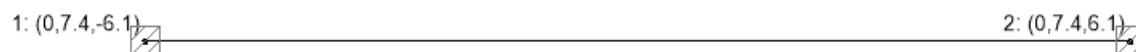
wall load dead - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



1.35G + 1.5Q + 1.5RQ (Strength) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



1.0G + 1.0Q + 1.0RQ (Service) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

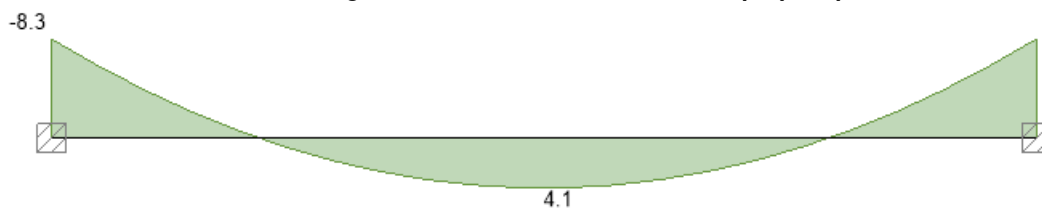


1.0G + 1.0ψ₂Q (Quasi) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

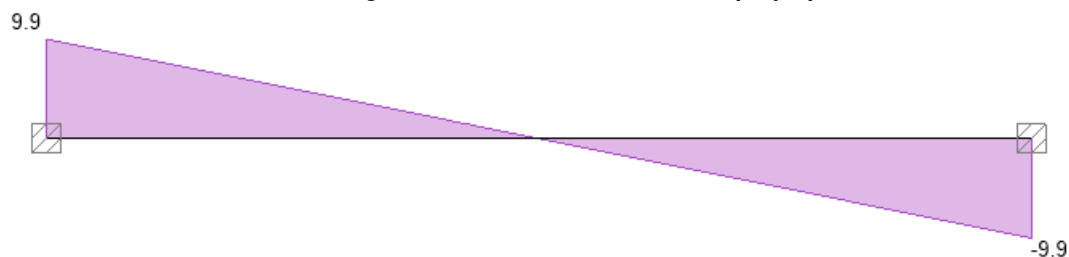


Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



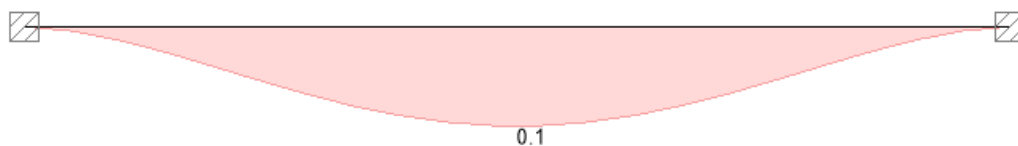


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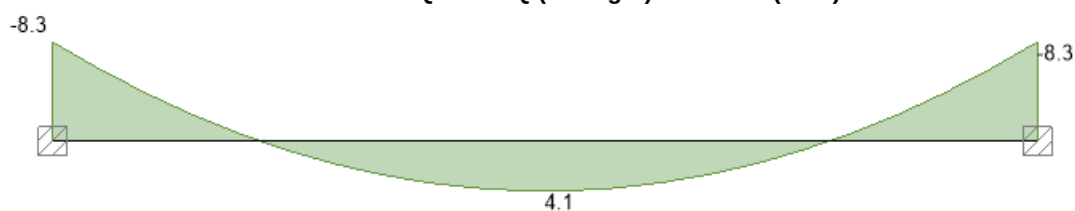
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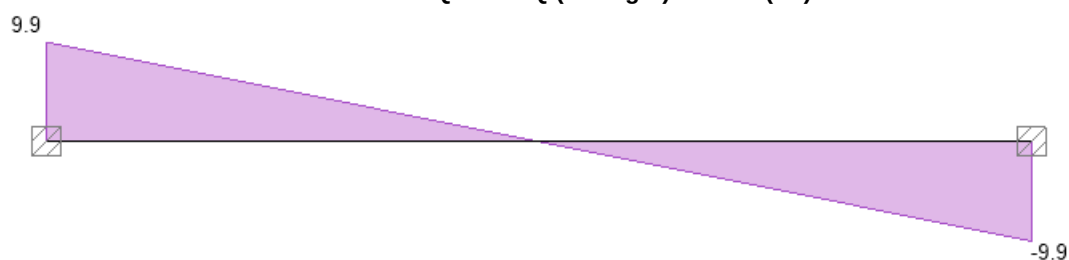
Service combinations - Deflection envelope (mm)



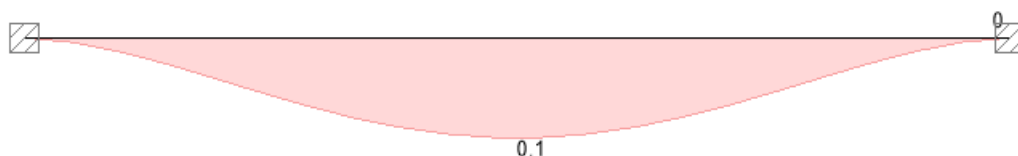
1.35G + 1.5Q + 1.5RQ (Strength) - Moment (kNm)



1.35G + 1.5Q + 1.5RQ (Strength) - Shear (kN)



1.0G + 1.0Q + 1.0RQ (Service) - Deflection (mm)



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

Concrete strength class	C35/45
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = 1.0
Characteristic compressive cylinder strength	$f_{ck} = 35 \text{ N/mm}^2$
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 43 \text{ N/mm}^2$
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 3.2 \text{ N/mm}^2$
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} \times \text{AAF} = 34077 \text{ N/mm}^2$
Ultimate strain - Table 3.1	$\epsilon_{cu2} = 0.0035$
Shortening strain - Table 3.1	$\epsilon_{cu3} = 0.0035$
Effective compression zone height factor	$\lambda = 0.80$
Effective strength factor	$\eta = 1.00$
Coefficient k_1	$k_1 = 0.40$
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \epsilon_{cu2}) = 1.00$
Coefficient k_3	$k_3 = 0.40$



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

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

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

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

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

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

Maximum aggregate size $h_{agg} = \mathbf{20 \text{ mm}}$

Density of reinforced concrete $\rho = \mathbf{2500 \text{ kg/m}^3}$

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

Reinforcement details

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

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

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

Nominal cover to reinforcement

Nominal cover to top reinforcement $c_{nom_t} = \mathbf{35 \text{ mm}}$

Nominal cover to bottom reinforcement $c_{nom_b} = \mathbf{35 \text{ mm}}$

Nominal cover to side reinforcement $c_{nom_s} = \mathbf{35 \text{ mm}}$

Fire resistance

Standard fire resistance period $R = \mathbf{60 \text{ min}}$

Number of sides exposed to fire $\mathbf{3}$

Minimum width of beam - EN1992-1-2 Table 5.5 $b_{min} = \mathbf{120 \text{ mm}}$

Beam - Span 1

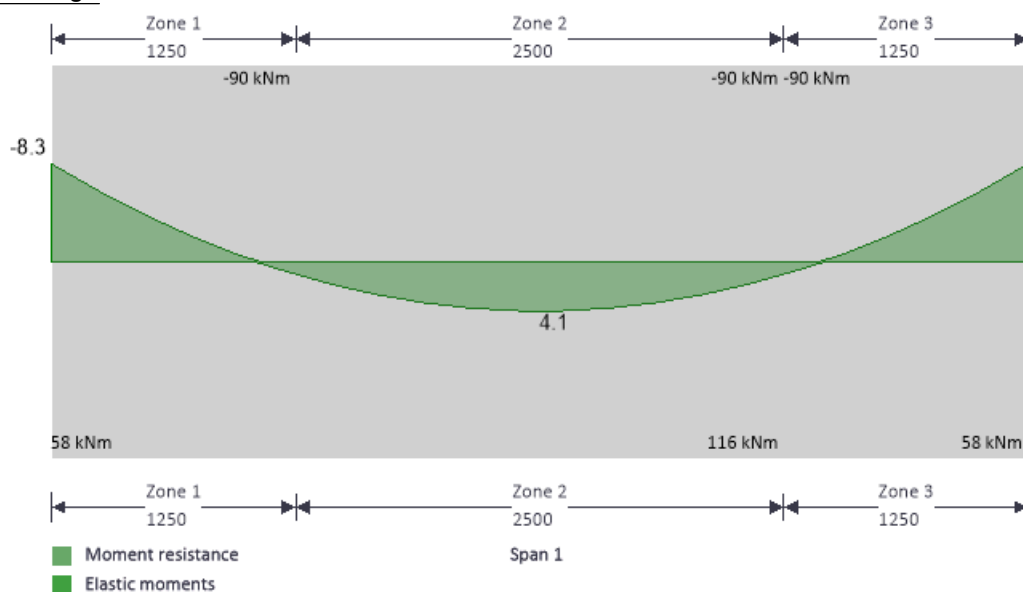
Rectangular section details

Section width $b = \mathbf{300 \text{ mm}}$

Section depth $h = \mathbf{400 \text{ mm}}$

PASS - Minimum dimensions for fire resistance met

Moment design

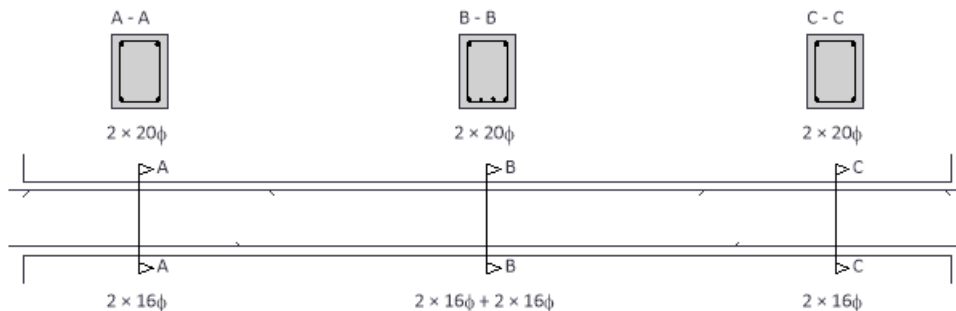




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Zone 1 (0 mm - 1250 mm) Positive moment - section 6.1

Design bending moment	$M = \text{abs}(M_{m1_s1_z1_max_red}) = 1.0 \text{ kNm}$
Effective depth of tension reinforcement	$d = 349 \text{ mm}$
Redistribution ratio	$\delta = \min(M_{pos_red_z1} / M_{pos_z1}, 1) = 1.000$
	$K = M / (b \times d^2 \times f_{ck}) = 0.001$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$
	$K' > K$ - No compression reinforcement is required
Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d) = 332 \text{ mm}$
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = 44 \text{ mm}$
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = 7 \text{ mm}^2$
Tension reinforcement provided	$2 \times 16\phi$
Area of tension reinforcement provided	$A_{s,prov} = 402 \text{ mm}^2$
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = 175 \text{ mm}^2$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = 4800 \text{ mm}^2$

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

Crack control - Section 7.3

Maximum crack width	$w_k = 0.30 \text{ mm}$
Design value modulus of elasticity reinf - 3.2.7(4)	$E_s = 200000 \text{ N/mm}^2$
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = 3.2 \text{ N/mm}^2$
Stress distribution coefficient	$k_c = 0.4$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = 1.00$
Actual tension bar spacing	$S_{bar} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z1_b_L1} - 1) + \phi_{m1_s1_z1_b_L1}) = 198 \text{ mm}$
Maximum stress permitted - Table 7.3N	$\sigma_s = 242 \text{ N/mm}^2$
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = 5.87$
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = 198 \text{ mm}$
Area of concrete in the tensile zone	$A_{ct} = b \times y = 59282 \text{ mm}^2$
Minimum area of reinforcement required - exp.7.1	$A_{s,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = 315 \text{ mm}^2$
	<i>PASS - Area of tension reinforcement provided exceeds minimum required for crack control</i>
Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z1_pos_quasi})) = 0.8 \text{ kNm}$
Permanent load ratio	$R_{PL} = M_{QP} / M = 0.74$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = 6 \text{ N/mm}^2$
Maximum bar spacing - Tables 7.3N	$S_{bar,max} = 300 \text{ mm}$

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PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 1 (0 mm - 1250 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1_s1_max_red}), \text{abs}(M_{m1_s1_z1_min_red})) = \mathbf{8.3 \text{ kNm}}$
Effective depth of tension reinforcement	$d = \mathbf{347 \text{ mm}}$
Redistribution ratio	$\delta = \min(M_{neg_red_z1} / M_{neg_z1}, 1) = \mathbf{1.000}$
	$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.007}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$
	<i>$K' > K$ - No compression reinforcement is required</i>
Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d) = \mathbf{330 \text{ mm}}$
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = \mathbf{43 \text{ mm}}$
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{58 \text{ mm}^2}$
Tension reinforcement provided	$2 \times 20\phi$
Area of tension reinforcement provided	$A_{s,prov} = \mathbf{628 \text{ mm}^2}$
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{174 \text{ mm}^2}$
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = \mathbf{4800 \text{ mm}^2}$

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

Crack control - Section 7.3

Maximum crack width	$w_k = \mathbf{0.30 \text{ mm}}$
Design value modulus of elasticity reinf – 3.2.7(4)	$E_s = \mathbf{200000 \text{ N/mm}^2}$
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = \mathbf{3.2 \text{ N/mm}^2}$
Stress distribution coefficient	$k_c = \mathbf{0.4}$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{1.00}$
Actual tension bar spacing	$S_{bar} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1})) / (N_{m1_s1_z1_t_L1} - 1) + \phi_{m1_s1_z1_t_L1} = \mathbf{194 \text{ mm}}$
Maximum stress permitted - Table 7.3N	$\sigma_s = \mathbf{245 \text{ N/mm}^2}$
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = \mathbf{5.87}$
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{196 \text{ mm}}$
Area of concrete in the tensile zone	$A_{ct} = b \times y = \mathbf{58904 \text{ mm}^2}$
Minimum area of reinforcement required - exp.7.1	$A_{s,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{309 \text{ mm}^2}$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_pos_quasi}), \text{abs}(M_{m1_s1_z1_neg_quasi})) = \mathbf{6.1 \text{ kNm}}$
Permanent load ratio	$R_{PL} = M_{QP} / M = \mathbf{0.74}$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{30 \text{ N/mm}^2}$
Maximum bar spacing - Tables 7.3N	$S_{bar,max} = \mathbf{300 \text{ mm}}$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing	$Stop = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_t_L1} \times N_{m1_s1_z1_t_L1})) / (N_{m1_s1_z1_t_L1} - 1) = \mathbf{174.0 \text{ mm}}$
Minimum allowable top bar spacing	$Stop,min = \max(\phi_{m1_s1_z1_t_L1} \times k_{s1}, h_{agg} + k_{s2}, 20 \text{ mm}) = \mathbf{25.0 \text{ mm}}$
	<i>PASS - Actual bar spacing exceeds minimum allowable</i>
Bottom bar spacing	$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z1_v}) + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / (N_{m1_s1_z1_b_L1} - 1) = \mathbf{182.0 \text{ mm}}$

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Minimum allowable bottom bar spacing

$$S_{bot,min} = \max(\phi_{m1_s1_z1_b_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) = \mathbf{25.0 \text{ mm}}$$

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (1250 mm - 3750 mm) Positive moment - section 6.1

Design bending moment

$$M = \text{abs}(M_{m1_s1_z2_max_red}) = \mathbf{4.1 \text{ kNm}}$$

Effective depth of tension reinforcement

$$d = \mathbf{349 \text{ mm}}$$

Redistribution ratio

$$\delta = \min(M_{pos_red_z2} / M_{pos_z2}, 1) = \mathbf{1.000}$$

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

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

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d) = \mathbf{332 \text{ mm}}$$

Depth of neutral axis

$$x = 2 \times (d - z) / \lambda = \mathbf{44 \text{ mm}}$$

Area of tension reinforcement required

$$A_{s,req} = M / (f_{yd} \times z) = \mathbf{29 \text{ mm}^2}$$

Tension reinforcement provided

$$2 \times 16\phi + 2 \times 16\phi$$

Area of tension reinforcement provided

$$A_{s,prov} = \mathbf{804 \text{ mm}^2}$$

Minimum area of reinforcement - exp.9.1N

$$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{175 \text{ mm}^2}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{s,max} = 0.04 \times b \times h = \mathbf{4800 \text{ mm}^2}$$

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

Crack control - Section 7.3

Maximum crack width

$$w_k = \mathbf{0.30 \text{ mm}}$$

Design value modulus of elasticity reinf - 3.2.7(4)

$$E_s = \mathbf{200000 \text{ N/mm}^2}$$

Mean value of concrete tensile strength

$$f_{ct,eff} = f_{ctm} = \mathbf{3.2 \text{ N/mm}^2}$$

Stress distribution coefficient

$$K_c = \mathbf{0.4}$$

Non-uniform self-equilibrating stress coefficient

$$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{1.00}$$

Actual tension bar spacing

$$S_{bar} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1) + \phi_{m1_s1_z2_b_L1} = \mathbf{66 \text{ mm}}$$

Maximum stress permitted - Table 7.3N

$$\sigma_s = \mathbf{347 \text{ N/mm}^2}$$

Steel to concrete modulus of elast. ratio

$$\alpha_{cr} = E_s / E_{cm} = \mathbf{5.87}$$

Distance of the Elastic NA from bottom of beam

$$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{195 \text{ mm}}$$

Area of concrete in the tensile zone

$$A_{ct} = b \times y = \mathbf{58587 \text{ mm}^2}$$

Minimum area of reinforcement required - exp.7.1

$$A_{s,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{217 \text{ mm}^2}$$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment

$$M_{QP} = \text{abs}(M_{m1_s1_z2_pos_quasi}) = \mathbf{3.1 \text{ kNm}}$$

Permanent load ratio

$$R_{PL} = M_{QP} / M = \mathbf{0.74}$$

Service stress in reinforcement

$$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{11 \text{ N/mm}^2}$$

Maximum bar spacing - Tables 7.3N

$$S_{bar,max} = \mathbf{300 \text{ mm}}$$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Deflection control - Section 7.4

Reference reinforcement ratio

$$\rho_{m0} = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.00592}$$

Required tension reinforcement ratio

$$\rho_m = A_{s,req} / (b \times d) = \mathbf{0.00027}$$

Required compression reinforcement ratio

$$\rho'_m = A_{s2,req} / (b \times d) = \mathbf{0.00000}$$

Structural system factor - Table 7.4N

$$K_b = \mathbf{1.0}$$

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Basic allowable span to depth ratio

$$\text{span_to_depth}_{\text{basic}} = K_b \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}] = \mathbf{1970.288}$$

Reinforcement factor - exp.7.17

$$K_s = \min(A_{s,\text{prov}} / A_{s,\text{req}} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5) = \mathbf{1.500}$$

Flange width factor

$$F1 = 1 = \mathbf{1.000}$$

Long span supporting brittle partition factor

$$F2 = 1 = \mathbf{1.000}$$

Allowable span to depth ratio

$$\text{span_to_depth}_{\text{allow}} = \min(\text{span_to_depth}_{\text{basic}} \times K_s \times F1 \times F2, 40 \times K_b) = \mathbf{40.000}$$

Actual span to depth ratio

$$\text{span_to_depth}_{\text{actual}} = L_{m1_s1} / d = \mathbf{14.327}$$

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

Minimum bar spacing (Section 8.2)

Top bar spacing

$$\text{Stop} = (b - (2 \times (C_{\text{nom}_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_t_L1} \times N_{m1_s1_z2_t_L1})) / (N_{m1_s1_z2_t_L1} - 1) = \mathbf{174.0 \text{ mm}}$$

Minimum allowable top bar spacing

$$\text{Stop}_{\text{min}} = \max(\phi_{m1_s1_z2_t_L1} \times K_{s1}, h_{\text{agg}} + K_{s2}, 20\text{mm}) = \mathbf{25.0 \text{ mm}}$$

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing

$$\text{Sbot} = (b - (2 \times (C_{\text{nom}_s} + \phi_{m1_s1_z2_v}) + \phi_{m1_s1_z2_b_L1} \times N_{m1_s1_z2_b_L1} + \phi_{m1_s1_z1_b_L1} \times N_{m1_s1_z1_b_L1})) / ((N_{m1_s1_z2_b_L1} + N_{m1_s1_z1_b_L1}) - 1) = \mathbf{50.0 \text{ mm}}$$

Minimum allowable bottom bar spacing

$$\text{Sbot}_{\text{min}} = \max(\phi_{m1_s1_z2_b_L1} \times K_{s1}, h_{\text{agg}} + K_{s2}, 20\text{mm}) = \mathbf{25.0 \text{ mm}}$$

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (3750 mm - 5000 mm) Positive moment - section 6.1

Design bending moment

$$M = \text{abs}(M_{m1_s1_z3_max_red}) = \mathbf{1.0 \text{ kNm}}$$

Effective depth of tension reinforcement

$$d = \mathbf{349 \text{ mm}}$$

Redistribution ratio

$$\delta = \min(M_{\text{pos_red_z3}} / M_{\text{pos_z3}}, 1) = \mathbf{1.000}$$

$$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.001}$$

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

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d) = \mathbf{332 \text{ mm}}$$

Depth of neutral axis

$$x = 2 \times (d - z) / \lambda = \mathbf{44 \text{ mm}}$$

Area of tension reinforcement required

$$A_{s,\text{req}} = M / (f_{yd} \times z) = \mathbf{7 \text{ mm}^2}$$

Tension reinforcement provided

$$2 \times 16\phi$$

Area of tension reinforcement provided

$$A_{s,\text{prov}} = \mathbf{402 \text{ mm}^2}$$

Minimum area of reinforcement - exp.9.1N

$$A_{s,\text{min}} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{175 \text{ mm}^2}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{s,\text{max}} = 0.04 \times b \times h = \mathbf{4800 \text{ mm}^2}$$

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

Crack control - Section 7.3

Maximum crack width

$$w_k = \mathbf{0.30 \text{ mm}}$$

Design value modulus of elasticity reinf - 3.2.7(4)

$$E_s = \mathbf{200000 \text{ N/mm}^2}$$

Mean value of concrete tensile strength

$$f_{ct,\text{eff}} = f_{ctm} = \mathbf{3.2 \text{ N/mm}^2}$$

Stress distribution coefficient

$$K_c = \mathbf{0.4}$$

Non-uniform self-equilibrating stress coefficient

$$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{1.00}$$

Actual tension bar spacing

$$S_{\text{bar}} = (b - (2 \times (C_{\text{nom}_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1) + \phi_{m1_s1_z3_b_L1} = \mathbf{198 \text{ mm}}$$

Maximum stress permitted - Table 7.3N

$$\sigma_s = \mathbf{242 \text{ N/mm}^2}$$



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Steel to concrete modulus of elast. ratio

$$\alpha_{cr} = E_s / E_{cm} = \mathbf{5.87}$$

Distance of the Elastic NA from bottom of beam

$$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{198 \text{ mm}}$$

Area of concrete in the tensile zone

$$A_{ct} = b \times y = \mathbf{59282 \text{ mm}^2}$$

Minimum area of reinforcement required - exp.7.1

$$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{315 \text{ mm}^2}$$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment

$$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z3_pos_quasi})) = \mathbf{0.8 \text{ kNm}}$$

Permanent load ratio

$$R_{PL} = M_{QP} / M = \mathbf{0.74}$$

Service stress in reinforcement

$$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{6 \text{ N/mm}^2}$$

Maximum bar spacing - Tables 7.3N

$$S_{bar,max} = \mathbf{300 \text{ mm}}$$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 3 (3750 mm - 5000 mm) Negative moment - section 6.1

Design bending moment

$$M = \max(\beta_1 \times \text{abs}(M_{m1_s1_max_red}), \text{abs}(M_{m1_s1_z3_min_red})) = \mathbf{8.3 \text{ kNm}}$$

Effective depth of tension reinforcement

$$d = \mathbf{347 \text{ mm}}$$

Redistribution ratio

$$\delta = \min(M_{neg_red_z3} / M_{neg_z3}, 1) = \mathbf{1.000}$$

$$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.007}$$

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

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d) = \mathbf{330 \text{ mm}}$$

Depth of neutral axis

$$x = 2 \times (d - z) / \lambda = \mathbf{43 \text{ mm}}$$

Area of tension reinforcement required

$$A_{s,req} = M / (f_{yd} \times z) = \mathbf{58 \text{ mm}^2}$$

Tension reinforcement provided

$$2 \times 20\phi$$

Area of tension reinforcement provided

$$A_{s,prov} = \mathbf{628 \text{ mm}^2}$$

Minimum area of reinforcement - exp.9.1N

$$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{174 \text{ mm}^2}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{s,max} = 0.04 \times b \times h = \mathbf{4800 \text{ mm}^2}$$

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

Crack control - Section 7.3

Maximum crack width

$$w_k = \mathbf{0.30 \text{ mm}}$$

Design value modulus of elasticity reinf - 3.2.7(4)

$$E_s = \mathbf{200000 \text{ N/mm}^2}$$

Mean value of concrete tensile strength

$$f_{ct,eff} = f_{ctm} = \mathbf{3.2 \text{ N/mm}^2}$$

Stress distribution coefficient

$$k_c = \mathbf{0.4}$$

Non-uniform self-equilibrating stress coefficient

$$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{1.00}$$

Actual tension bar spacing

$$S_{bar} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1})) / (N_{m1_s1_z3_t_L1} - 1) + \phi_{m1_s1_z3_t_L1} = \mathbf{194 \text{ mm}}$$

Maximum stress permitted - Table 7.3N

$$\sigma_s = \mathbf{245 \text{ N/mm}^2}$$

Steel to concrete modulus of elast. ratio

$$\alpha_{cr} = E_s / E_{cm} = \mathbf{5.87}$$

Distance of the Elastic NA from bottom of beam

$$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{196 \text{ mm}}$$

Area of concrete in the tensile zone

$$A_{ct} = b \times y = \mathbf{58904 \text{ mm}^2}$$

Minimum area of reinforcement required - exp.7.1

$$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{309 \text{ mm}^2}$$

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment

$$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_pos_quasi}), \text{abs}(M_{m1_s1_z3_neg_quasi})) = \mathbf{6.1 \text{ kNm}}$$

Permanent load ratio

$$R_{PL} = M_{QP} / M = \mathbf{0.74}$$



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Service stress in reinforcement

$$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{30 \text{ N/mm}^2}$$

Maximum bar spacing - Tables 7.3N

$$S_{bar,max} = \mathbf{300 \text{ mm}}$$

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing

$$S_{top} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_t_L1} \times N_{m1_s1_z3_t_L1})) / (N_{m1_s1_z3_t_L1} - 1) = \mathbf{174.0 \text{ mm}}$$

Minimum allowable top bar spacing

$$S_{top,min} = \max(\phi_{m1_s1_z3_t_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) = \mathbf{25.0 \text{ mm}}$$

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing

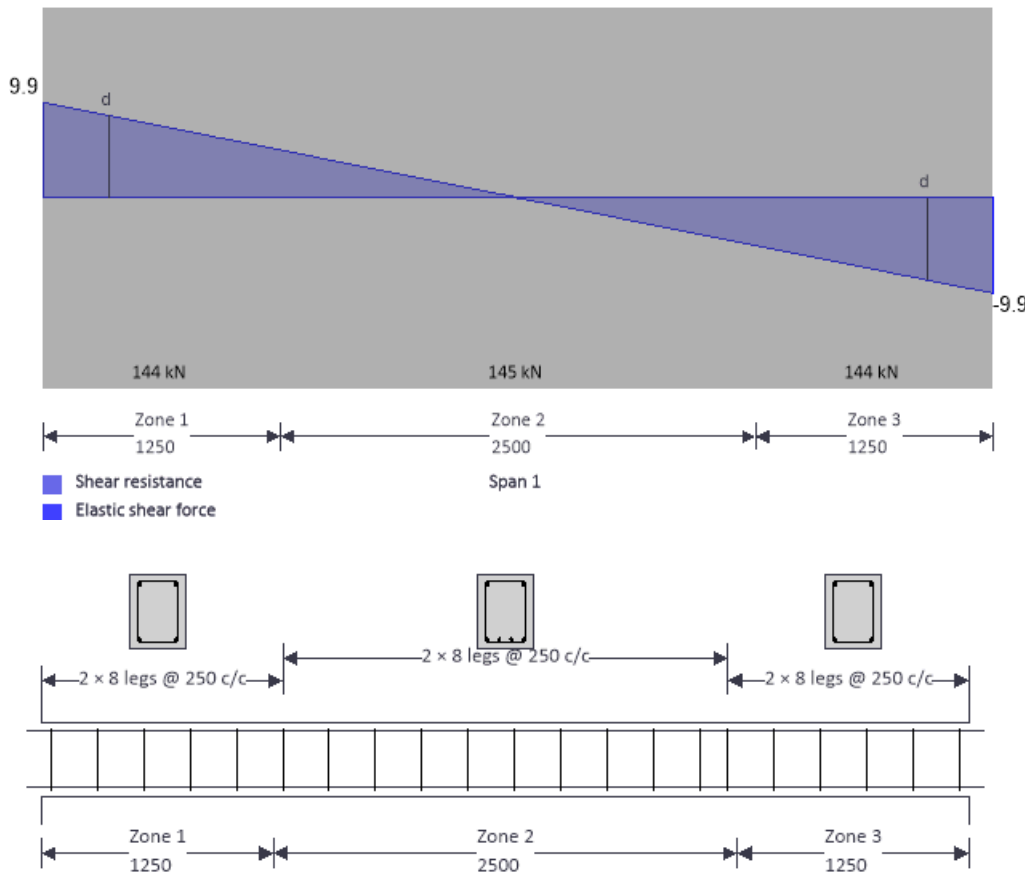
$$S_{bot} = (b - (2 \times (C_{nom_s} + \phi_{m1_s1_z3_v}) + \phi_{m1_s1_z3_b_L1} \times N_{m1_s1_z3_b_L1})) / (N_{m1_s1_z3_b_L1} - 1) = \mathbf{182.0 \text{ mm}}$$

Minimum allowable bottom bar spacing

$$S_{bot,min} = \max(\phi_{m1_s1_z3_b_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) = \mathbf{25.0 \text{ mm}}$$

PASS - Actual bar spacing exceeds minimum allowable

Shear design



Angle of comp. shear strut for maximum shear

$$\theta_{max} = \mathbf{45 \text{ deg}}$$

Strength reduction factor - cl.6.2.3(3)

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = \mathbf{0.516}$$

Compression chord coefficient - cl.6.2.3(3)

$$\alpha_{cw} = \mathbf{1.00}$$

Minimum area of shear reinforcement - exp.9.5N

$$A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{284 \text{ mm}^2/\text{m}}$$

Zone 1 (0 mm - 1250 mm) shear - section 6.2

Design shear force at support

$$V_{Ed,max} = \max(\text{abs}(V_{z1_max}), \text{abs}(V_{z1_red_max})) = \mathbf{10 \text{ kN}}$$

Min lever arm in shear zone

$$z = \mathbf{330 \text{ mm}}$$

Maximum design shear resistance - exp.6.9

$$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{c wd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{595 \text{ kN}}$$



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

Design shear force at 347mm from support

$$V_{Ed} = 9 \text{ kN}$$

Design shear stress

$$v_{Ed} = V_{Ed} / (b \times z) = 0.086 \text{ N/mm}^2$$

Angle of concrete compression strut - cl.6.2.3

$$\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1)), 1)), 21.8 \text{ deg}), 45 \text{ deg}) = 21.8 \text{ deg}$$

Area of shear reinforcement required - exp.6.8

$$A_{sv,des} = V_{Ed} \times b / (f_{yd} \times \cot(\theta)) = 24 \text{ mm}^2/\text{m}$$

Area of shear reinforcement required

$$A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = 284 \text{ mm}^2/\text{m}$$

Shear reinforcement provided

$$2 \times 8 \text{ legs @ } 250 \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov} = 402 \text{ mm}^2/\text{m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N

$$s_{vl,max} = 0.75 \times d = 260 \text{ mm}$$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 2 (1250 mm - 3750 mm) shear - section 6.2

Design shear force at support

$$V_{Ed,max} = \max(\text{abs}(V_{z2,max}), \text{abs}(V_{z2,red,max})) = 5 \text{ kN}$$

Min lever arm in shear zone

$$z = 332 \text{ mm}$$

Maximum design shear resistance - exp.6.9

$$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = 599 \text{ kN}$$

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

Design shear force within zone

$$V_{Ed} = 5 \text{ kN}$$

Design shear stress

$$v_{Ed} = V_{Ed} / (b \times z) = 0.050 \text{ N/mm}^2$$

Angle of concrete compression strut - cl.6.2.3

$$\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1)), 1)), 21.8 \text{ deg}), 45 \text{ deg}) = 21.8 \text{ deg}$$

Area of shear reinforcement required - exp.6.8

$$A_{sv,des} = V_{Ed} \times b / (f_{yd} \times \cot(\theta)) = 14 \text{ mm}^2/\text{m}$$

Area of shear reinforcement required

$$A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = 284 \text{ mm}^2/\text{m}$$

Shear reinforcement provided

$$2 \times 8 \text{ legs @ } 250 \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov} = 402 \text{ mm}^2/\text{m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N

$$s_{vl,max} = 0.75 \times d = 262 \text{ mm}$$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Zone 3 (3750 mm - 5000 mm) shear - section 6.2

Design shear force at support

$$V_{Ed,max} = \max(\text{abs}(V_{z3,max}), \text{abs}(V_{z3,red,max})) = 10 \text{ kN}$$

Min lever arm in shear zone

$$z = 330 \text{ mm}$$

Maximum design shear resistance - exp.6.9

$$V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = 595 \text{ kN}$$

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

Design shear force at 347mm from support

$$V_{Ed} = 9 \text{ kN}$$

Design shear stress

$$v_{Ed} = V_{Ed} / (b \times z) = 0.086 \text{ N/mm}^2$$

Angle of concrete compression strut - cl.6.2.3

$$\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1)), 1)), 21.8 \text{ deg}), 45 \text{ deg}) = 21.8 \text{ deg}$$

Area of shear reinforcement required - exp.6.8

$$A_{sv,des} = V_{Ed} \times b / (f_{yd} \times \cot(\theta)) = 24 \text{ mm}^2/\text{m}$$

Area of shear reinforcement required

$$A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = 284 \text{ mm}^2/\text{m}$$

Shear reinforcement provided

$$2 \times 8 \text{ legs @ } 250 \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov} = 402 \text{ mm}^2/\text{m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N

$$s_{vl,max} = 0.75 \times d = 260 \text{ mm}$$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Appendix D

Demolition Schedule

Prior to demolition of the full 3 storey rear section of the building. A lower level demolition of the annex would be performed as to allow for a trial pit to be dug in the rear of the structure as to ascertain the foundation buildup of the rear section of the building. Additionally on the 1st and 2nd floor of the structure it would be required to open up sections of the floor and roof in a small section as to allow for confirmation of the steel member build-up and connections between the rear section and the proposed retained main section of the structure.

This low roof new
 Section would need
 to be decided prior
 to the full demolition
 of the building to
 allow for analysis
 of the foundation below
 the main building

STAGE 1
STAGE 2
STAGE 3

Propose trial
 pit location
 put across foundations
 of main building

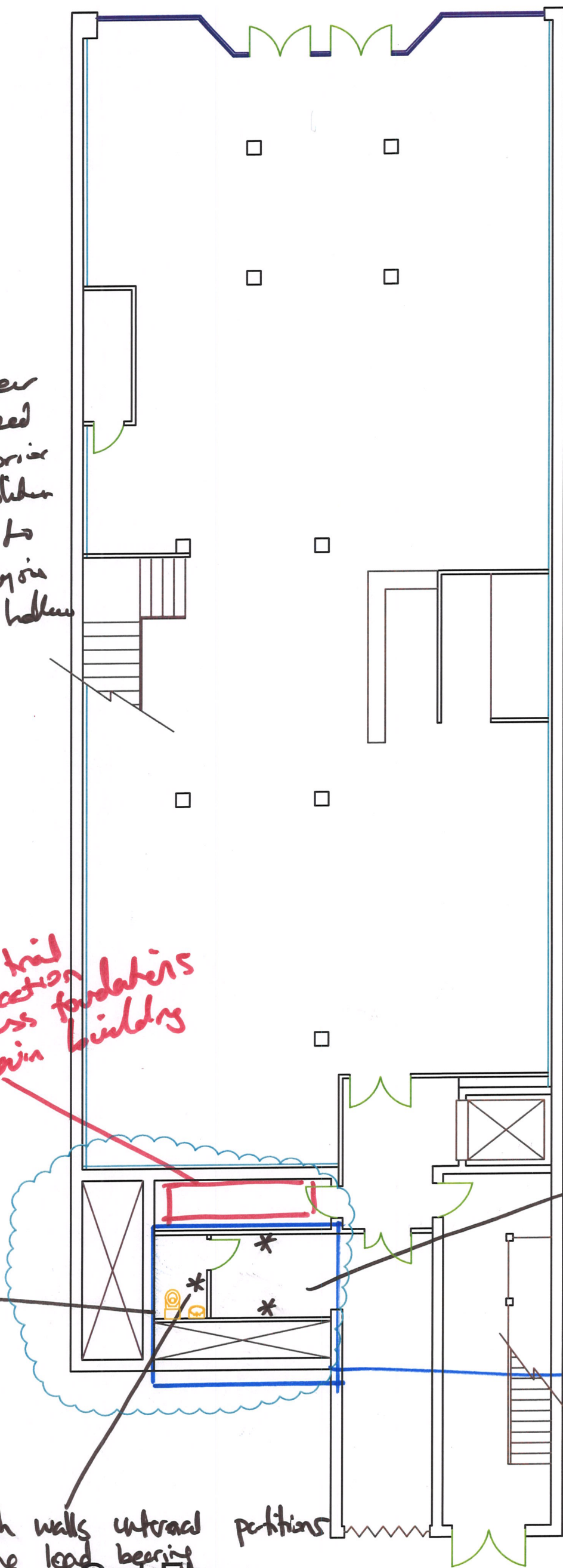
Remove Bathroom
 Appliances &
 tap off water.

Remove Roof Material
 Joists & Truss Sheets

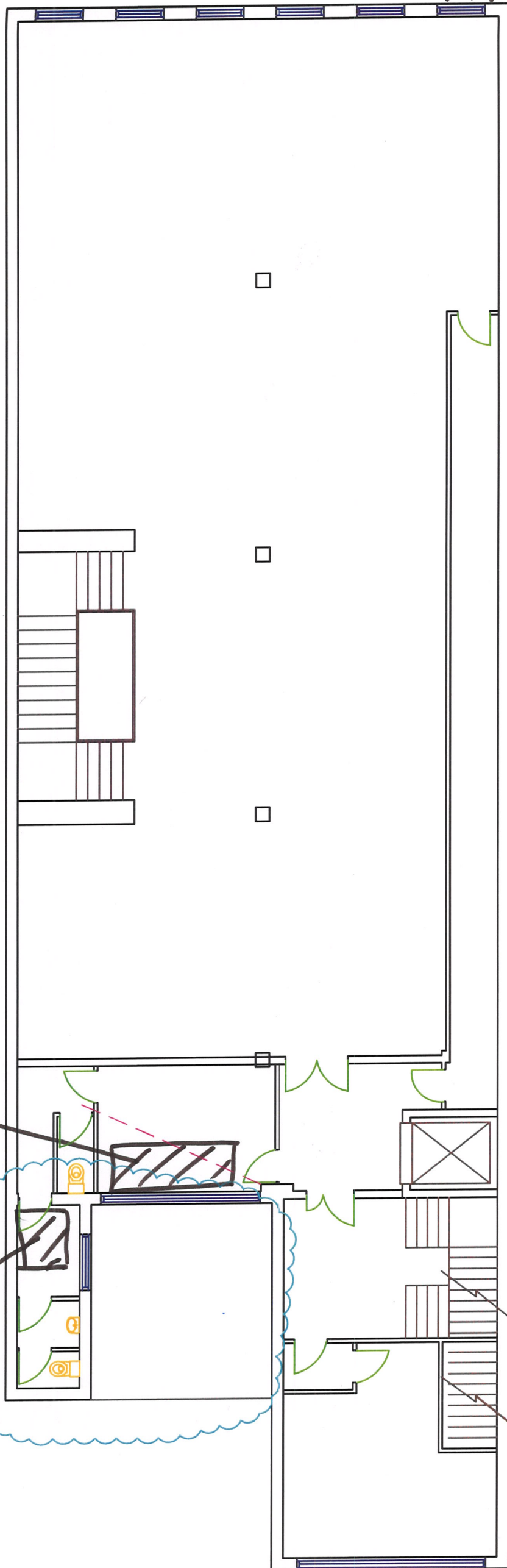
Remove external walk
 in this area.

* Demolish walls internal partitions
 as none load bearing

Grd Floor



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DEMOLITION SCHEDULE
1st / 2nd Floor.



STAGE 1

Expose section of floor to allow to determine beams below.

Strip section of roof to allow to determine steel connections.

1st Floor / 2nd Floor

This would need performing on both floors for clarity prior to demolition.