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CROFT STRUCTURAL ENGINEERS

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Basement Impact Assessment – Structural Methodology

Site Address: 69 Avenue Road
London
NW8 6HP

Rev	Date	Author	Checker	Comment
-	28.4.20	PH	CT	First issue for comment
1	27.11.20	PH	CT	Section 5.3, 5.5.4 updated

Contents

Basement Impact Assessment (Structural Methodology) for 69 Avenue Rd

1. Non-Technical Summary

1.1. Existing Property, Site & Neighbouring Sites

The site comprises a single, detached domestic dwelling over 3 storeys.

1.2. Proposed Development

The proposal to extend the property includes a singles storey rear/side extension with a new basement storey under. The basement will be formed using reinforced concrete piles and retaining walls, with steel beams to support the ground floor structure.

1.3. Screening, Scoping and Basement Impact Assessment

A Basement Impact Assessment has been carried out by GEA. The results are not included within this report.

2. Report Authors and Qualifications

This structural methodology report has been prepared by Philip Henry, MEng CEng MICE

3. Introduction

3.1. Site & location

The is located to the north of Regents Park, London.

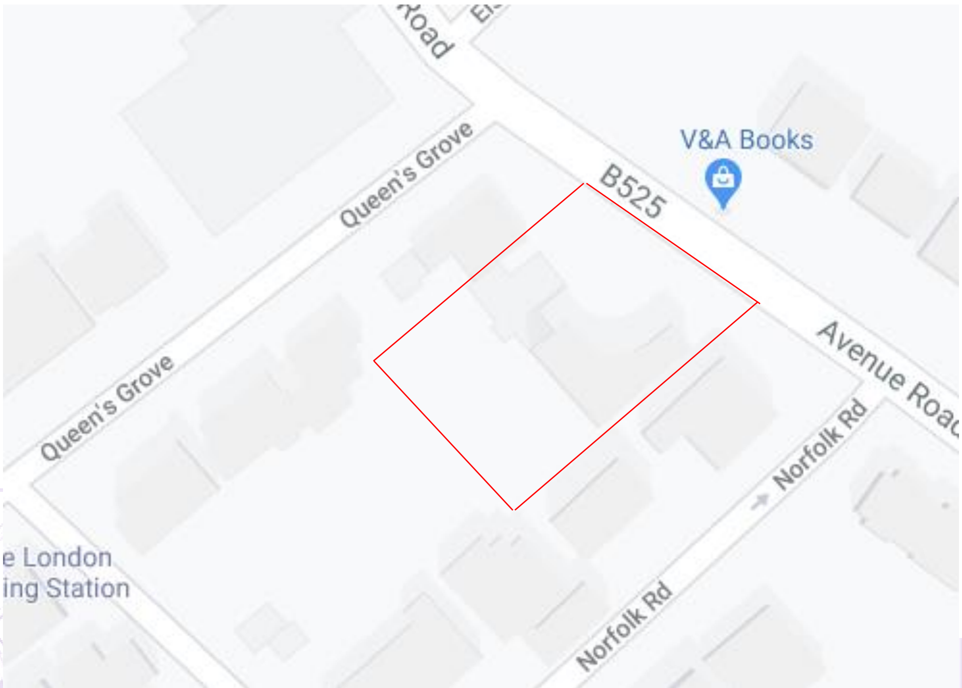

Figure 1: Plan view of site (approx. area outlined in red) and the surrounding properties

For further information refer to the Desk Study Section.

3.2. Proposed works

Refer to the architects plans and Crofts Structural scheme Appendix D to this BIA. The engineering statement & temporary works construction sequence is in Appendices.

The existing building is to be retained, with a new rear and side extension which will replace the existing extension. A new basement will be constructed underneath the existing building, and to extend under the garden.

4. Desk Study & Walk over Survey

4.1. General Desk Study

4.1.1. Site History

The Historical Review is included in the GEA BIA report. The house was constructed between 1915 and 1935, and the extension was added by 1954.

4.1.2. Listed buildings

The existing building is not listed. Data from Historic England shows that there are listed buildings close by, on Queen's Grove and Norfolk Road. These buildings are more than 30m from the proposed structure and will not be affected by any excavations.

The site is not in a conservation area.

4.1.3. London Under Ground and Network Rail Infrastructures

The site is more than 500m away from the nearest national rail line and the nearest subterranean train line. These are unlikely to be affected by the new basement.

4.1.4. Highways

The basement is not within 5m of the public highway.

4.1.5. UK Power Network

There are no significant items of electrical infrastructure (such as pylons, substations or tunnels) in the immediate vicinity.

4.1.6. Utility Search

The basement is more than 10m from the public highway and there are not

4.2. Walk Over Survey

A structural engineer from Croft Structural Engineers visited the site in March 2020 and conducted a visual appraisal of the existing structure, including the roof/attic space. No opening-up works were conducted.

4.2.1. Site and Existing Property

The existing building was constructed between 1915 and 1935. The building is detached and is formed in an L-shape over three storeys; the ground and first floor cover the full footprint, while the second floor is within the roof space. The main load bearing walls are constructed in solid masonry, with timber floors and roof.

On the right hand side of the property is a single storey extension. This is also constructed in solid masonry, with a timber roof supported on steel beams, and a concrete floor. This extension will be demolished and replaced as part of the proposed works.

The boundary walls on the left and right side are formed in solid masonry.

The building is set back from the road/pavement by approximately 10 metres on the short end and 25m on the main front wall. To the front of the building there is a large courtyard with a large area of hardstanding forming a carriageway. To the left hand side is a passage which is fully paved, and to the rear of the property is a patio across the extent of the rear elevation.

Generally, the house is in a good condition. There are some minor cracks on internal finished surfaces, but no cracks were noted that were more than 2mm width. These cracks are generally described as 'non-structural' and are to be expected in a building of this size and age.

4.2.2. Proximity of Trees

To the front of the property is a 23m high Tree of Heaven, which is fully mature, with a diameter of approximately 900mm.

There are other trees within the property boundaries but given the distance to the proposed basement would not be affected by the works.

4.2.3. Adjacent Properties

4.2.3.1. No. 65 – Property to Left

Property age: approx. 1930s or later
Property use: domestic
Number of storeys: 3
Current basement: none. The neighbouring property does not have a basement. The planning portal indicates that there are no plans for a basement.

The property to the left is a detached property, approximately 5m from the left hand flank wall of 69 Avenue Road. It is formed in load bearing masonry walls, with timber mansard roof.

Figure 2 – Neighbouring property, 67 Avenue Road

4.2.3.2. No. 71 – Property to Right

Property age: 1930s or later
Property use: domestic
Number of storeys: 2
Current Basement: No.
The neighbouring property does not have a basement. The planning portal indicates that there are no plans for a basement.

Figure 3 - Neighbouring property, 71 Avenue Road from rear

5. Construction Methodology and Engineer Statements

5.1. Outline Geotechnical Design Parameters

From the Geological report and soil investigation, reasonably conservative geotechnical parameters have been determined, based on the soil investigation: design overall stability to K_a & K_p values.

K_a = 0.333 , K_p = 2.359

5.2. Hydrostatic Pressure

Design temporary condition for water table level, if deeper than basement ignore.

Design permanent condition for water table level:

If deeper than existing, design reinforcement for water table at full basement depth to allow for local failure of water mains, drainage and storm water. Global uplift forces can be ignored when the water table is lower than the basement. BS8102 only indicates guidance.

5.2.1. Intended Use & Loadings

	UDL kN/m²	Concentrated Load kN
Domestic Single Dwellings	1.5	2.0

Below ground level, the reinforced concrete retaining walls are designed to carry the lateral loading applied from above.

The lateral earth pressure exerts a horizontal force on the retaining walls. The retaining walls will be checked for resistance to the overturning force this produces.

Lateral forces will be applied from:

- Soil loads
- Hydrostatic pressures
- Surcharge loading from behind the wall

These forces produce retaining wall thrust. This will be restrained by the opposing retaining wall.

5.2.1.1. Surcharge Loading

The following will be applied as surcharge loads to the front/ front lightwell retaining walls:

- 10kN/m^2 if within 45° of road
- 100kN point loads if under road or within 1.5m
- 5kN/m^2 if within 45° of Pavement
- Garden Surcharge 2.5kN/m^2 + 1 m of soil (if present above basement ceiling) 20kN/m^2

- Surcharge for adjacent property 1.5kN/m^2 + 4kN/m^2 for concrete ground bearing slab

Adjacent Properties:
All adjacent property footings within 45° to have additional geotechnical engineers' input. A line at 45° from the base of the neighbours' wall footing would be intersected by the basement retaining wall. This should be accounted for in the design.

5.3. Permanent Design Proposals

The basement will involve underpinning of some of the existing external walls, while other walls will be formed in a contiguous piled wall, with reinforced concrete lining wall and reinforced concrete slab.

The design of the retaining walls was calculated using software by TEDDS. The software is specifically designed for retaining walls and ensures that the construction is kept to a limit to prevent damage to the adjacent properties.

The overall stability of the walls is designed using K_a & K_p values, while the design of the wall structure uses K_0 values. This approach minimises the level of movement from the concrete affecting the adjacent properties.

The loading from the house is mainly carried on steel frames at ground floor level which span front to back. These frames place the loading onto the retaining walls, which in turn transfer the loading to the ground. In the temporary case, the loadings can be placed onto temporary 'plunge columns'. The contiguous piled wall therefore does not carry any vertical load, and needs be propped only to withstand horizontal loading. An embedment of approximately ¾ of the retained height is expected in the detailed design.

The investigations highlight that water is not likely to be present, but that a water level of two-thirds of the basement depth should be assumed. The walls are designed to resist the hydrostatic pressure. The water table was recorded as low. The design of the walls considers long term scenarios. It is possible that a water main may break causing a local high water table. To account for this, the wall is designed for a water level of two-thirds of the basement depth.

The design also considers floatation as a risk. The design has accounted for the weight of the building and the uplift forces from the water. The weight of the building is greater than the uplift, resulting in a stable structure.

Appendix A shows the calculations of one of the most heavily loaded retaining walls. The most critical parameters have been used for this.

5.3.1. Temporary works

Walls are designed to be structurally stable with top and bottom propping. Temporary propping details will be required to be provided by the contractor and must be completed by a suitability qualified professional.

To demonstrate the feasibility of the works, a proposed basement construction sequence is in included in appendix D, along with a basement method statement in appendix E.

5.4. Ground Movement Assessment

See GEA ground movement Assessment

5.5. Control of Construction Works

5.5.1. Control of Construction Works

A construction sequence has been formulated with Croft's experience of over 500 basements. The procedures described in this statement will mitigate the impacts that the construction of the basement will have on nearby properties.

To reduce the risk to the development:

- Employ a reputable firm that has extensive knowledge of basement works.
- Employ suitably qualified consultants Croft Structural Engineers has completed over 500 basements in the last five years.
- Provide method statements for the contractors to follow
- Investigate the ground this has now been done.
- Record and monitor the properties close by. This is completed by a condition survey under the Party Wall Act, before and after the works are completed.

With the measures listed above, the maximum level of cracking anticipated is 'Hairline' cracking. This can be repaired with normal decorative works. Under the Party Wall Act, minor damage, although unwanted, can be tolerated it is permitted to occur to a neighbouring property as long as repairs are suitability undertaken to rectify this. To mitigate this risk, the Party Wall Act is to be followed and a Party Wall Surveyor will be appointed.

5.5.2. Noise and Nuisance Control

The contractor is to follow the good working practices and guidance laid down in the 'Considerate Constructors Scheme'.

The hours of working will be limited to those allowed; 8am to 5pm Monday to Friday and Saturday Morning 8am to 1pm.

None of the practices cause undue noise that one would typically expect from a construction site (a conveyor belt typically runs at around 70dB).

The site has car parking to the front to which the skip will be stored.

The site will be hoarded with 8' site hoarding to prevent access.

The hours of working will further be defined within the Party Wall Act.

The site is to be hoarded to minimise the level of direct noise from the site.

Working in the basement generally requires hand tools to be used. The level of noise generally will be no greater than that of digging of soil. The noise is reduced and muffled by the works being undertaken underground. The level of noise from basement construction works is lower than typical ground level construction due to this.

5.5.3. Construction Management Plan

For the Construction Phase Management Plan, it may be beneficial to compile a Construction Management Plan (CMP). A suitably qualified person, typically the contractor, would provide the CMP. The items that should be considered are

- Delivers routes and times
- Expected working hours
- Times when local roads may become bust: school times, other construction sites.
- Volume of muck away, how this is managed and when.
- Required plant
- Noise dust and Vibration
- Waste Management

This is outside the brief of the Basement Impact Assessment and is not covered within Croft's brief.

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5.5.4. Monitoring

In order to safeguard the existing structures during underpinning and new basement construction, movement monitoring using total stations or similar is to be undertaken.

Before the works begin, a detailed monitoring report is required to confirm the implementation of the monitoring. The items that this should cover are:

- Risk Assessment to determine level of monitoring
- Scope of Works
- Applicable standards
- Frequency of Monitoring
- Specification for Instrumentation
- Monitoring of Existing cracks
- Monitoring of movement
- Reporting

We would recommend that the monitoring frequency should follow:

Pre-construction: Monitored once.

During construction: Monitored after every pin is cast for first 4 no. pins to gauge effect of underpinning. If all is well, monitor after every other pin.

Post construction works: Monitored once.

Trigger values and contingency actions are noted in the table below.

MOVEMENT		CATEGORY	ACTION
Vertical	Horizontal		
0mm-3mm	0-3mm	Green	No action required
3mm-5mm	3-5mm	AMBER	Detailed review of Monitoring: Check studs are OK and have not moved. Ensure site staff have not moved studs. If studs have moved reposition. Relevel to ensure results are correct and tolerance is not a concern. Inform Party Wall surveyors of amber readings. Double the monitoring for 2 further readings. If stable revert back. Carry out a local structural review and inspection. Double number of lateral props
5mm-7mm	5-10mm		Implement remedial measures review method of working and ground conditions
>7mm	>10mm	RED	Implement structural support as required; Cease works with the exception of necessary works for the safety and stability of the structure and personnel; Review monitoring data and implement revised method of works

Appendix A: Structural Calculations

Building Regulations will be required after planning. As part of the building control pack full calculations must be undertaken and provided at detailed design stage once planning permission is granted. The calculations must be completed to a recognised Standard (BS or Euro Codes). The calculations must take into account the findings of this report and the recommendations of the auditors.

The design must resist:

- Vertical loads from the proposed works and adjacent properties
- Lateral loads from wind, soil water and adjacent properties
- Loadings in the temporary condition
- All other applied loads on the building
- Uplift forces from hydrostatic effects and soil heave

The final proposed scheme must:

- Provide stability in the temporary condition to all forces
- Provide stability to all forces in the permanent condition

As part of the planning Croft structural engineers has considered some of the pertinent parts of the basement structure to ensure that it can be constructed. The following calculations are not a full set of calculations for the final design which must be provided for building regulations.

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RETAINING WALL DESIGN

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 2.9.11

Retaining wall details

Stem type	Propped cantilever
Stem height	h_{stem} = **4000** mm
Prop height	h_{prop} = **4000** mm
Stem thickness	t_{stem} = **350** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1500** mm
Base thickness	t_{base} = **350** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **4000** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2700** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

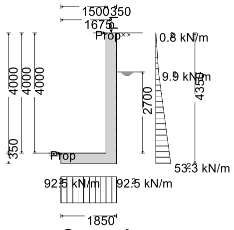
Soil type	Medium dense well graded sand
Moist density	γ_{mr} = **21** kN/m^3
Saturated density	γ_{sr} = **23** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **0** deg

Base soil properties

Soil type	Stiff clay
Soil density	γ_b = **19** 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	$P_{bearing}$ = **140** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2
Vertical line load at 1675 mm	P_{G1} = **100** kN/m
	P_{Q1} = **20** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} =$ **1850** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} =$ **2700** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} =$ **1300** mm
Length of surcharge load	$l_{sur} = l_{heel} =$ **0** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 =$ **1850** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} =$ **4350** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 =$ **2175** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} =$ **1.4** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 =$ **1675** mm
Area of wall base	$A_{base} = l_{base} \times t_{base} =$ **0.648** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 =$ **925** mm

Using Coulomb theory

Active pressure coefficient	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) =$ **0.333**
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	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{35}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{16.2}$ kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{120}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{171.2}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times Surcharge_Q \times h_{eff} = \mathbf{3.6}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{20.4}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{45.6}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{33.7}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-2.7}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{100.7}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{58.6}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{15}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-7.9}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{201}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-20.8}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-46.4}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-62.9}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{136.6}$ kNm/m

Check bearing pressure

Propping force to stem	$F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) = \mathbf{5}$ kN/m
Propping force to base	$F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{95.7}$ kN/m
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{21.7}$ kNm/m
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{925}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{1850}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{92.5}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{92.5}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.513}$

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 2.9.11

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

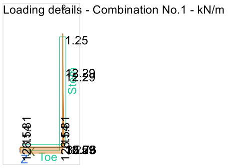
Cover to reinforcement

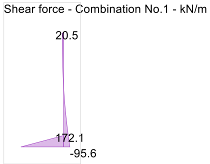
Front face of stem c_{sf} = **40** mm

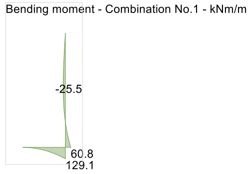
Rear face of stem c_{sr} = **50** mm

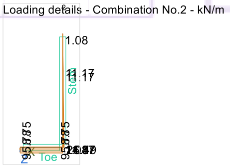
Top face of base c_{bt} = **50** mm

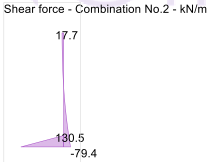
Bottom face of base c_{bb} = **75** mm

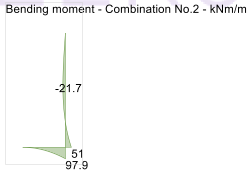












Check stem design at 2128 mm

Depth of section — h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **25.5** kNm/m

Depth to tension reinforcement — $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2$ = **295** mm

$K = M / (d^2 \times f_{ck})$ = **0.010**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **280** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **37** mm

Area of tension reinforcement required $A_{sfM.req} = M / (f_{yd} \times z)$ = **210** mm²/m

Tension reinforcement provided 10 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM})$ = **785** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **444** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sfM.max} = 0.04 \times h$ = **14000** mm²/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov}$ = **0.566**

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 $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**

Required tension reinforcement ratio $\rho = A_{sfM.req} / d$ = **0.001**

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 $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b)$ = **40**

Actual span to depth ratio h_{prop} / d = **13.6**

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} = **18.5** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **83.9** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

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

$A_{c.eff}$ = **104375** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.008**

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 $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **362** 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 = **0.091** mm

w_k / w_{max} = **0.304**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **60.8** kNm/m

Depth to tension reinforcement $d = h - c_{sr} - \phi_{sr} / 2$ = **294** mm

$K = M / (d^2 \times f_{ck})$ = **0.023**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **279** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **37** mm

Area of tension reinforcement required $A_{sr.req} = M / (f_{yd} \times z)$ = **500** mm²/m

Tension reinforcement provided 12 dia.bars @ 100 c/c

Area of tension reinforcement provided $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **1131** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **443** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sr.max} = 0.04 \times h$ = **14000** mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.442**

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 $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**

Required tension reinforcement ratio $\rho = A_{sr.req} / d$ = **0.002**

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 $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.5**

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

Actual span to depth ratio h_{prop} / d = **13.6**

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} = **44.2** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **139.8** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

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

	$A_{c.eff}$ = **104417** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.011**
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	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **358** 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 = **0.15** mm
	w_k / w_{max} = **0.501**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **95.6** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.473** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **145.5** kN/m
	$V / V_{Rd.c}$ = **0.657**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section	h = **350** mm

Rectangular section in shear - Section 6.2

Design shear force	V = **20.5** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr1.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.473** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **145.5** kN/m
	$V / V_{Rd.c}$ = **0.141**

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)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/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** mm²/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 = **129.1** kNm/m

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

$K = M / (d^2 \times f_{ck})$ = **0.061**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **250** mm

Depth of neutral axis $x = 2.5 \times (d – z)$ = **38** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **1188** mm²/m

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

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

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **399** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **14000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.378**

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} = **94.3** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **120.1** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

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

$A_{c.eff}$ = **104003** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.030**

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 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **368** 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 = **0.137** mm

w_k / w_{max} = **0.458**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **172.1** kN/m

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

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

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.012**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.490** N/mm²

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

$V_{Rd.c}$ = **195.4** kN/m

$V / V_{Rd.c}$ = **0.881**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

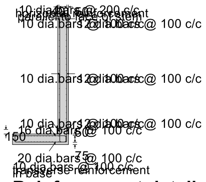
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **628** mm²/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) s_{bx_max} = **450** mm

Transverse reinforcement provided 10 dia.bars @ 100 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **785** mm²/m

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



Reinforcement details

Appendix B: Construction programme

<u>The Contractor is responsible for the final construction programme</u>

Outline construction Program

(For planning purposes only)

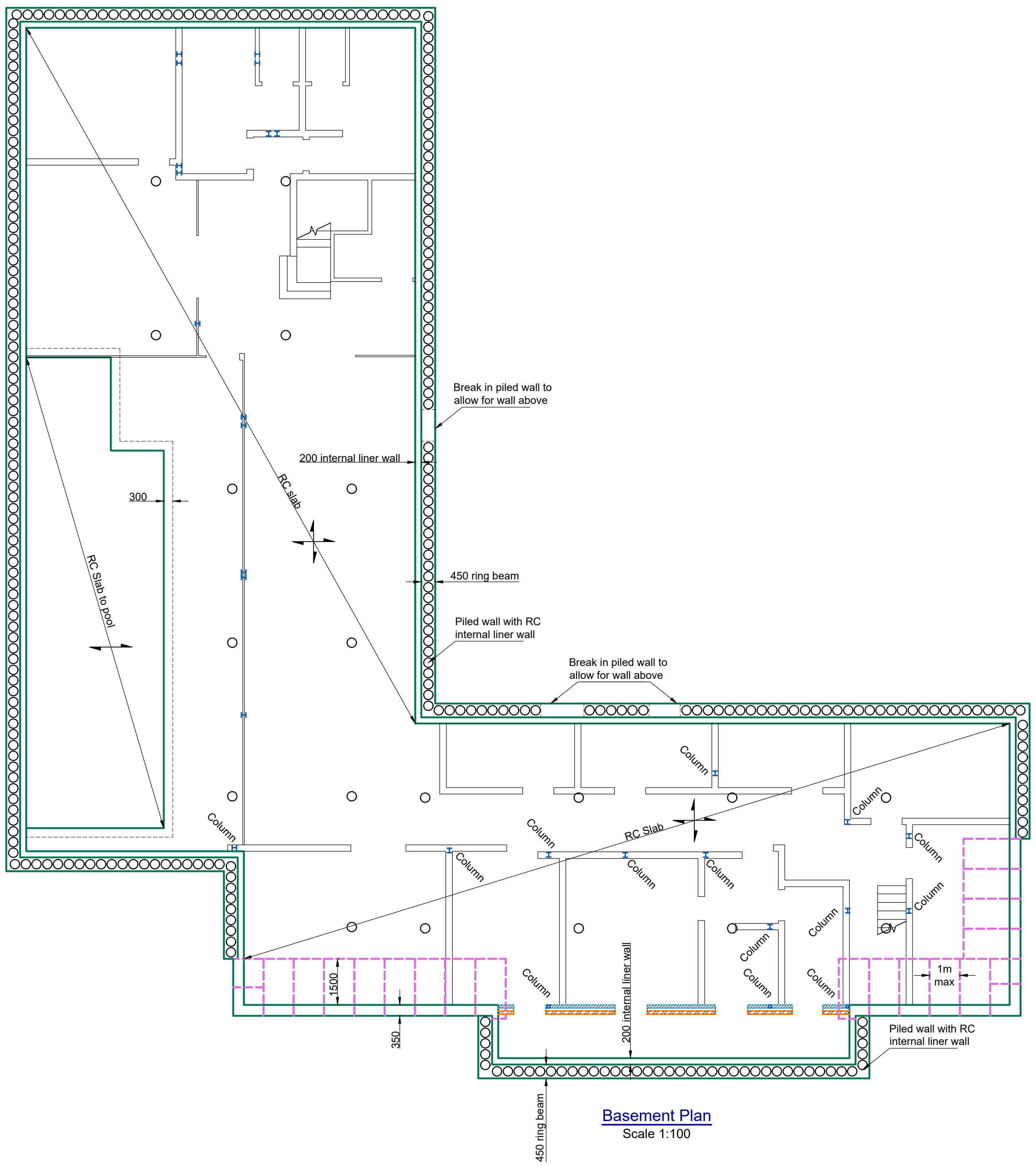
	Months															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Planning approval																
Detailed Design																
Tender																
Party Walls																
Monitoring of Adjacent structures																
Enabling works																
Basement Construction																
Superstructure construction																

Appendix C: Structural Drawings

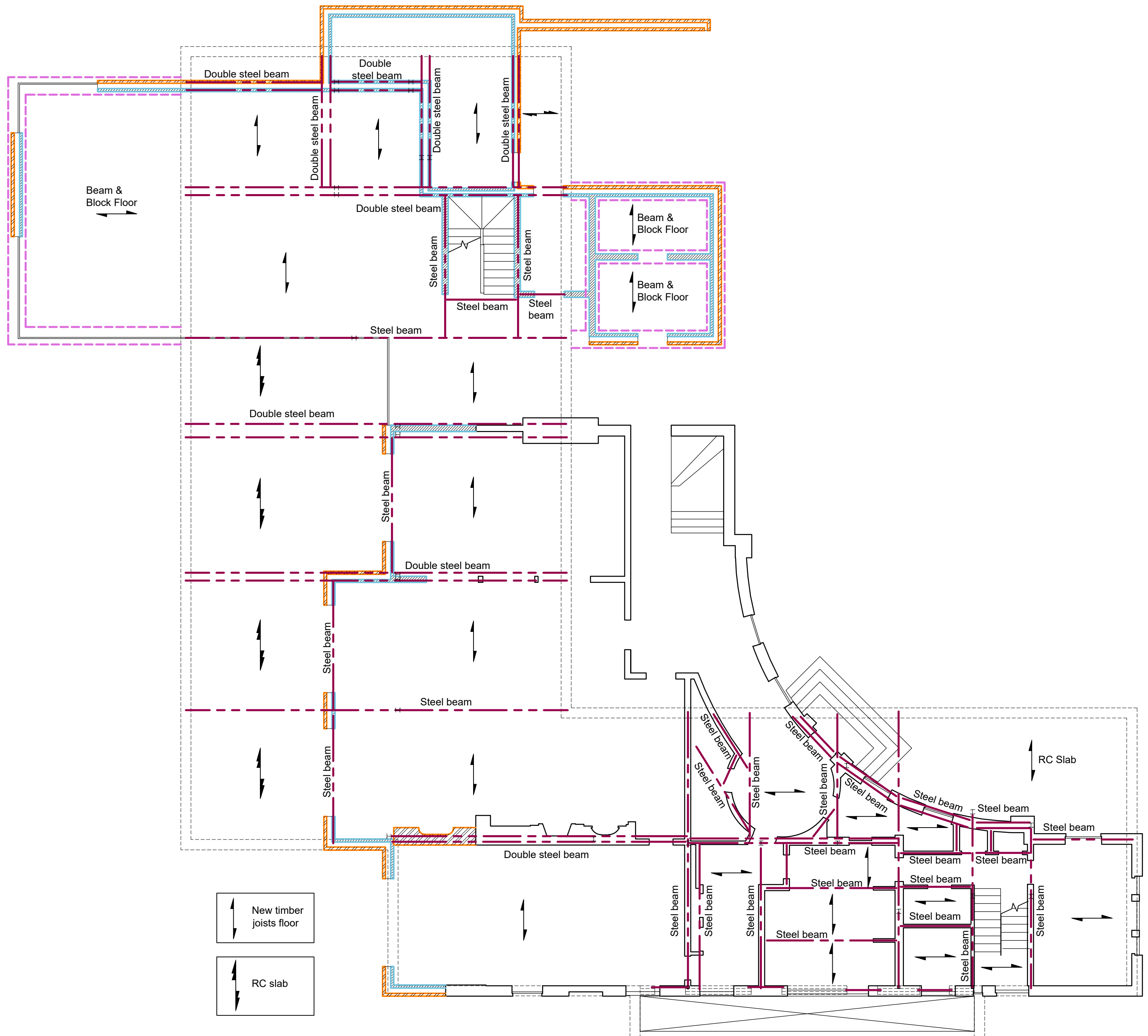
1:100 Basement Plan on A1

1:100 Ground Floor plan on A1

1:50 Section on A3 Including section through Neighbouring Footings



Basement Plan
Scale 1:100



Ground Floor Plan
Scale 1:100

Rev	Date	Amendments
-	21/04/2020	First issue for comment

Client: Mr Nicholas Goulandris
Project: 69 Avenue Road
Title : Basement Scheme for BIA

Job Number: 200203
Date: Apr 20
Dwg Number: SK-01
Rev: -
Drawn: SB
Chk'd: PH
Scale: As shown @ A1

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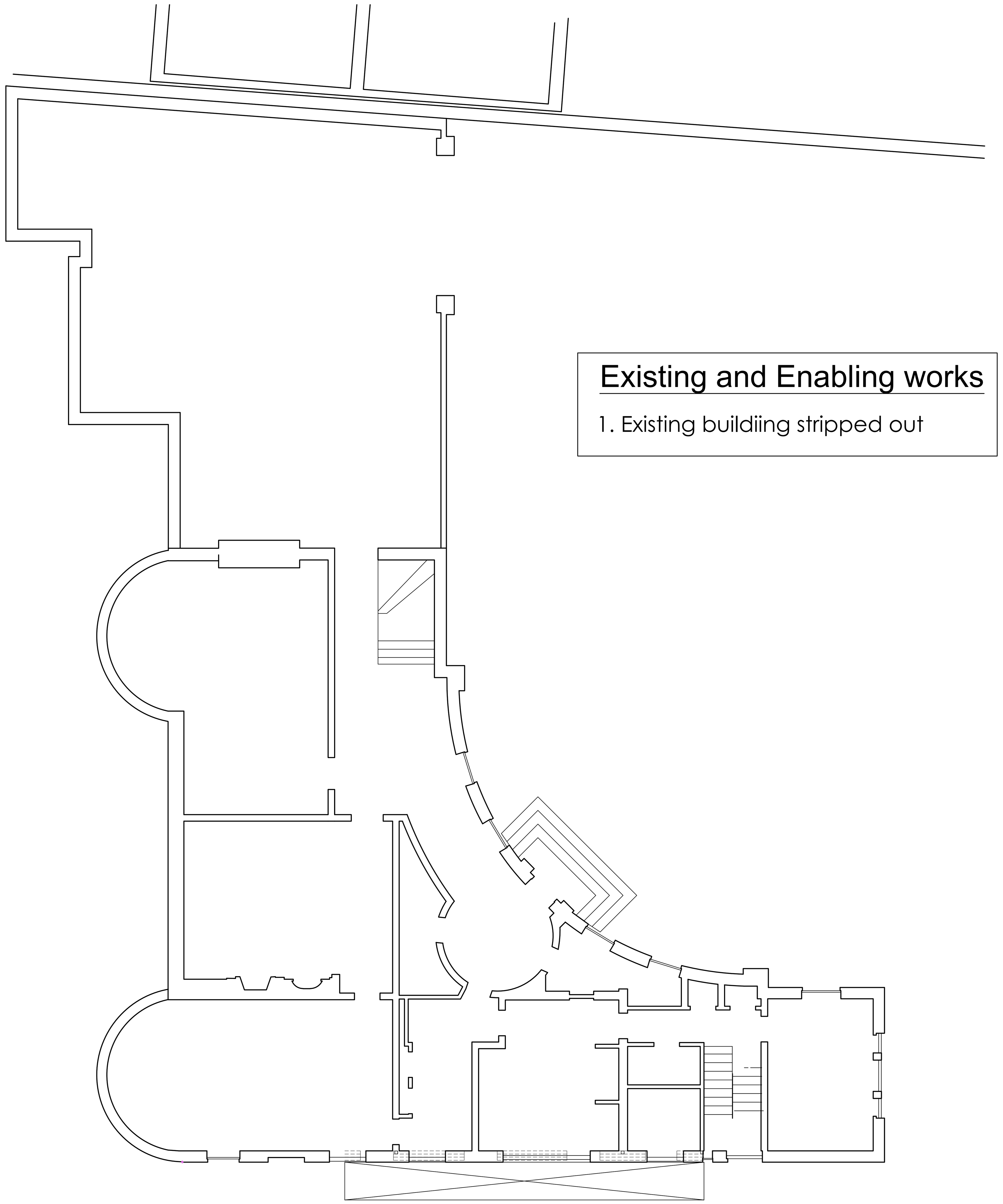
Appendix D: Temporary Works Sequence

- Construction Sequencing – Plans at 1:100 on A1
- Party Wall sections at 1:50 on A1



Existing and Enabling works

1. Existing buildiing stripped out

Ground Floor Plan

Scale 1:100

Rev	Date	Amendments
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris

Project: 69 Avenue Road

Title : Basement Scheme for BIA

Job Number 200203	Date Apr 20
Dwg Number TW-01	Rev -
Drawn SB	Ch'kd PH
Scale As shown @ A1	

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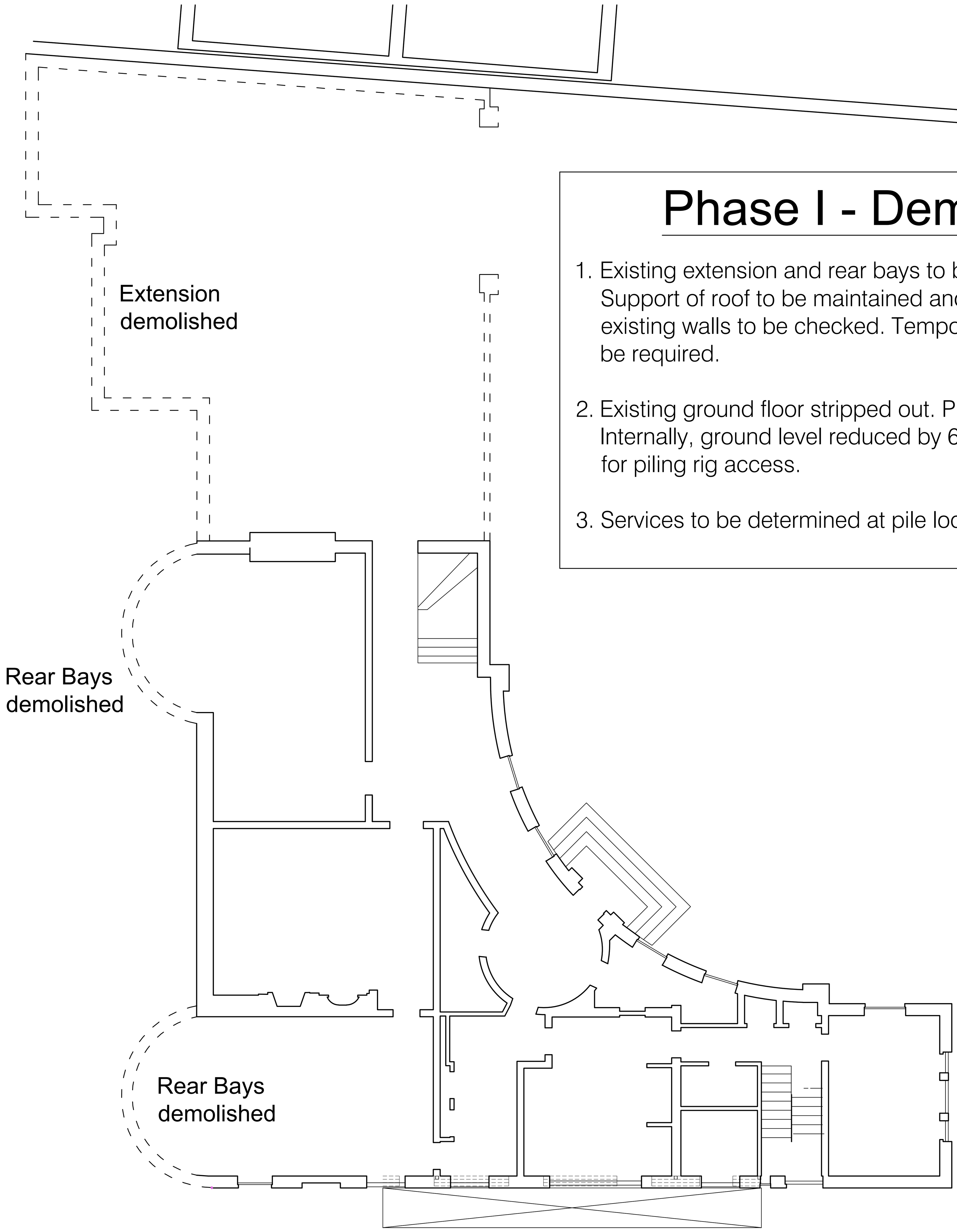

Phase I - Demolition

1. Existing extension and rear bays to be demolished. Support of roof to be maintained and stability of existing walls to be checked. Temporary racing ma be required.

2. Existing ground floor stripped out. Piling mat placed. Internally, ground level reduced by 600mm to allow for piling rig access.

3. Services to be determined at pile locations.

Ground Floor Plan
Scale 1:100

Basement Floor Plan
Scale 1:100

Rev	Date	Amendments
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris
Project: 69 Avenue Road
Title : Basement Scheme for BIA

Job Number 200203	Date Apr 20
Dwg Number TW-02	Rev -
Drawn SB	Ch'kd PH
Scale As shown @ A1	

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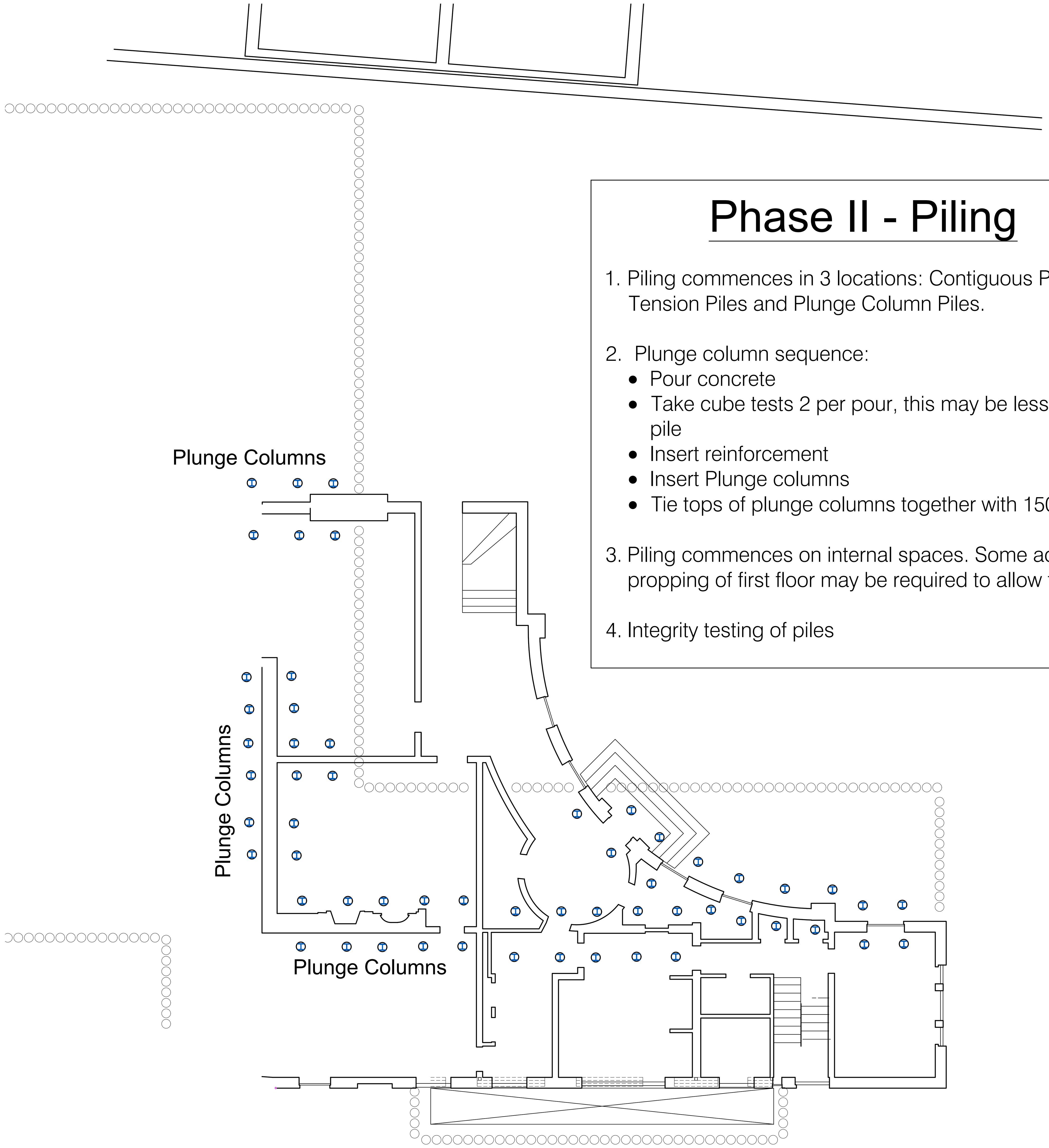
Phase II - Piling

1. Piling commences in 3 locations: Contiguous Piled Walls, Tension Piles and Plunge Column Piles.

2. Plunge column sequence:
   - Pour concrete
   - Take cube tests 2 per pour, this may be less than 2 per pile
   - Insert reinforcement
   - Insert Plunge columns
   - Tie tops of plunge columns together with 150x90 PFC

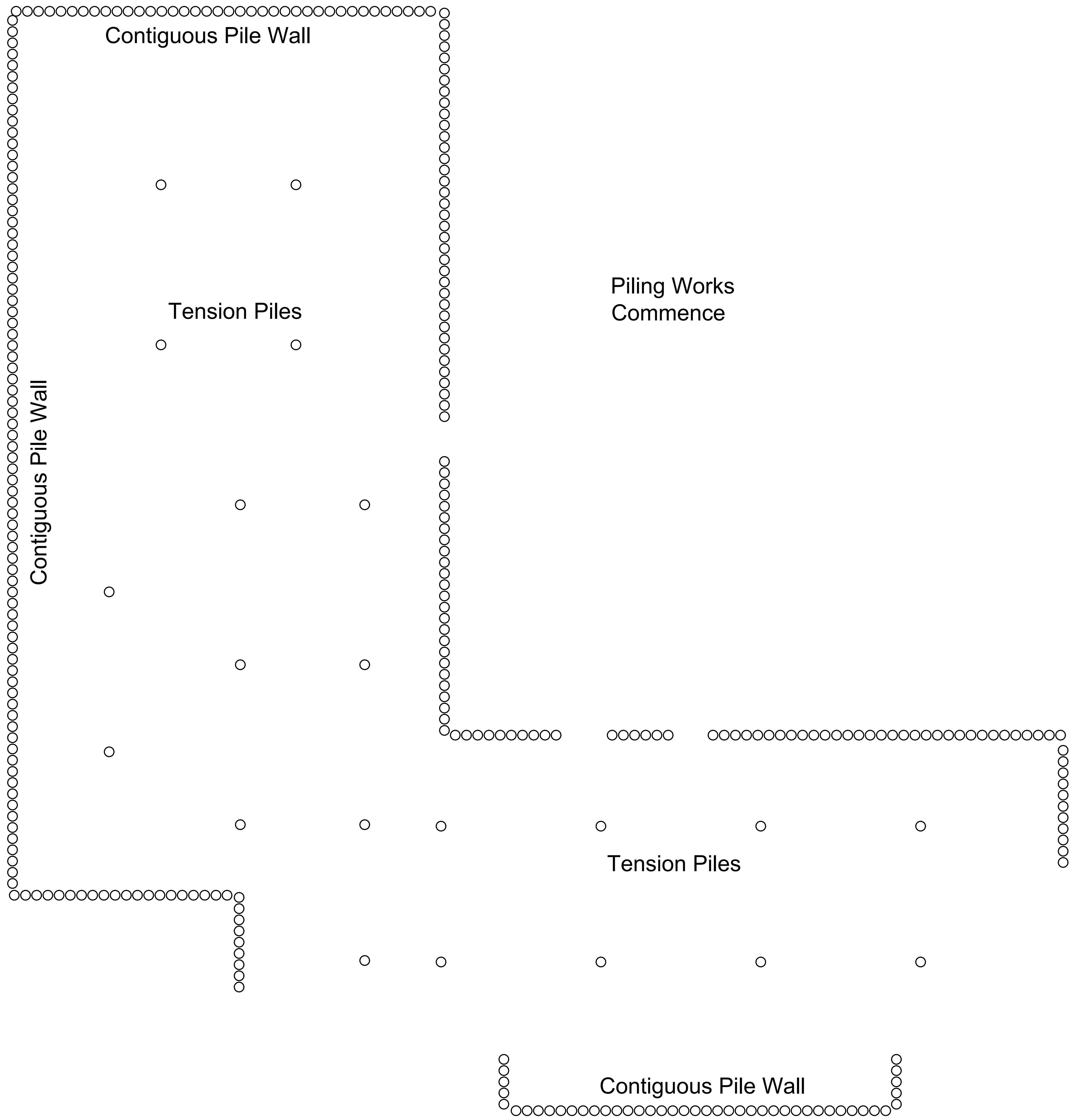
3. Piling commences on internal spaces. Some additional propping of first floor may be required to allow for holes.

4. Integrity testing of piles



Ground Floor Plan

Scale 1:100



Basement Floor Plan

Scale 1:100

Rev	Date	Amendments
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris

Project: 69 Avenue Road

Title : Basement Scheme for BIA

Job Number	Date
200203	Apr 20
Dwg Number	Rev
TW-03	-
Drawn	Ch'kd
SB	PH
Scale	As shown @ A1

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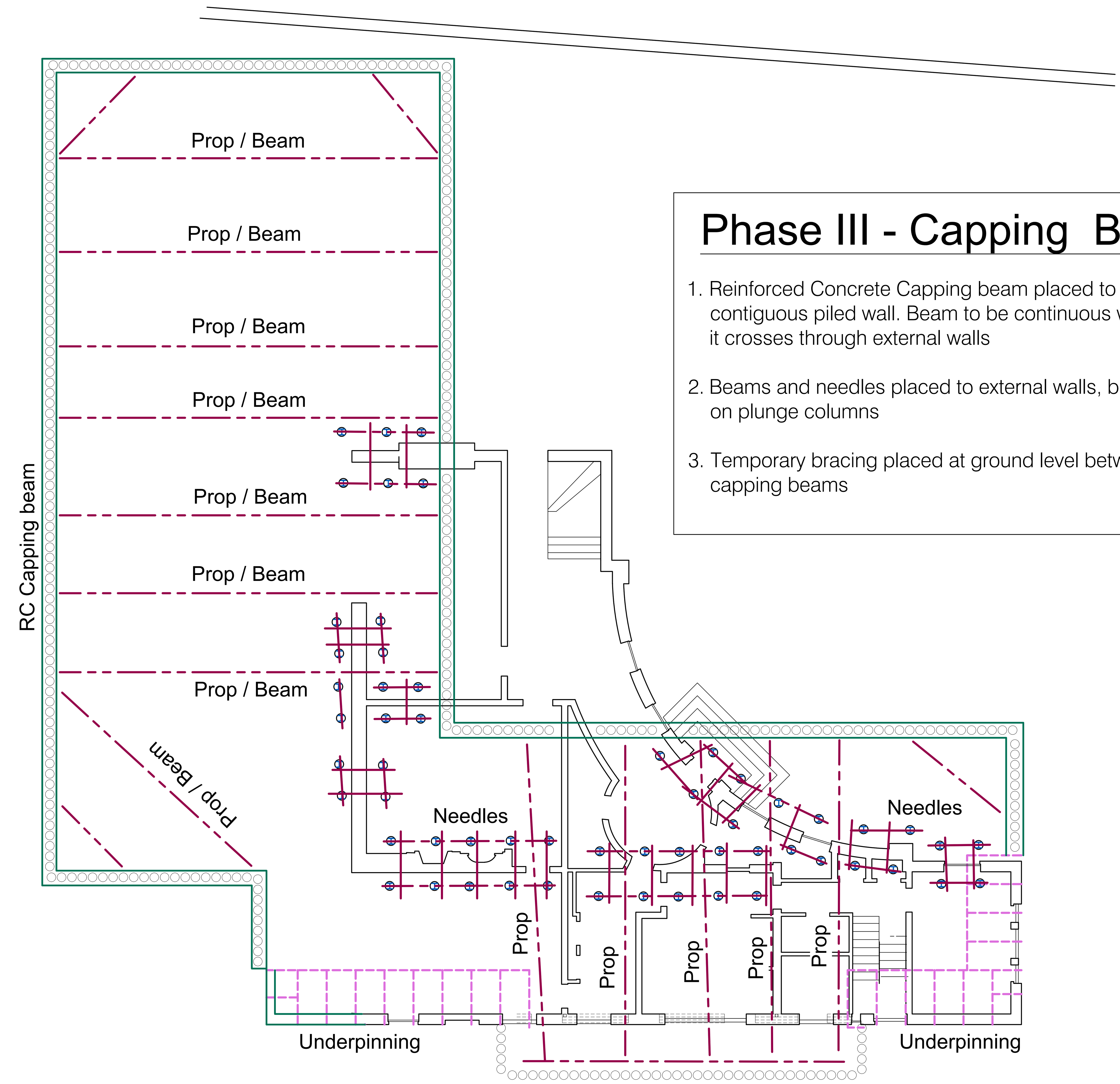
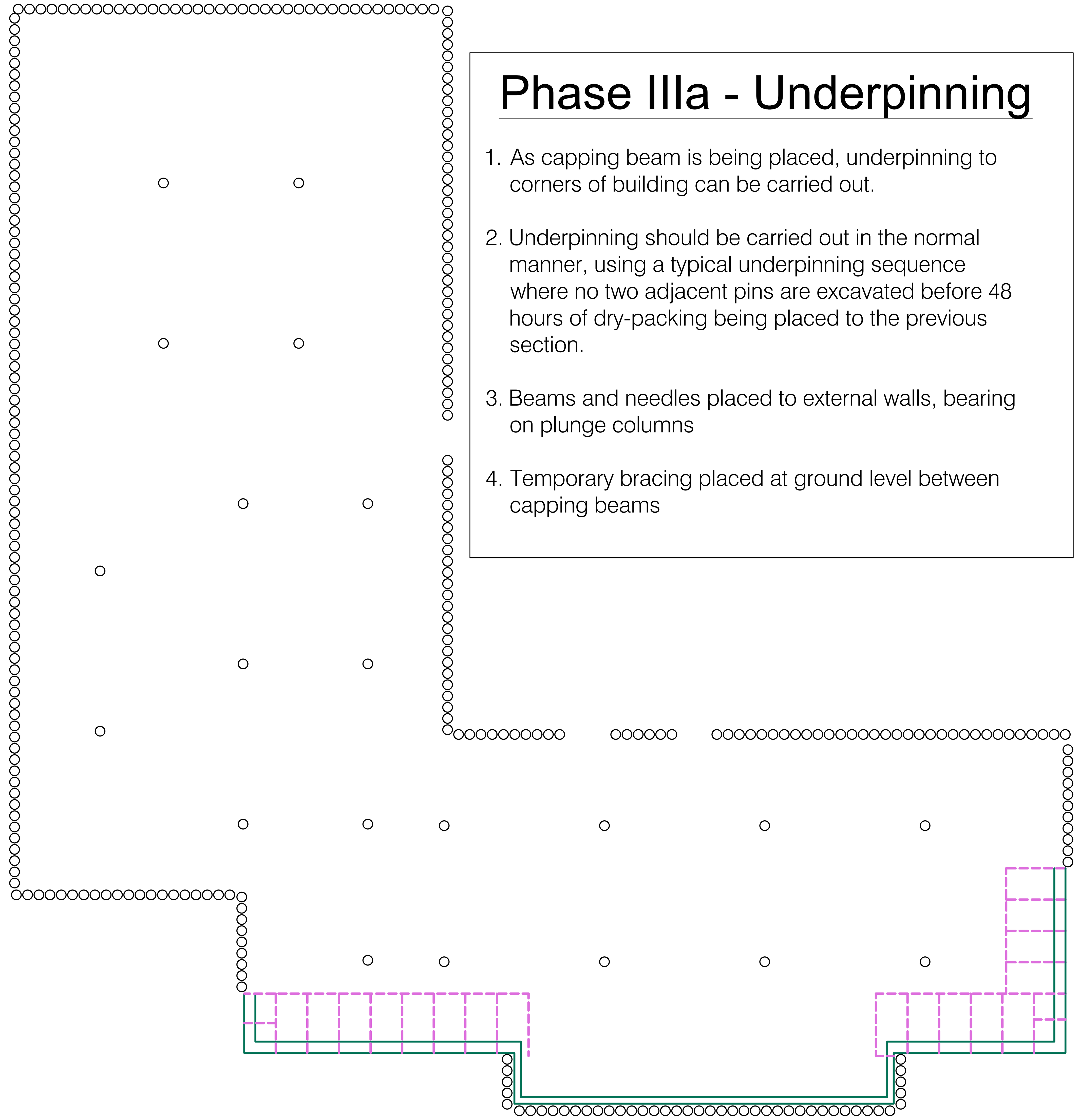


Phase III - Capping Beam

1. Reinforced Concrete Capping beam placed to top of contiguous piled wall. Beam to be continuous where it crosses through external walls
2. Beams and needles placed to external walls, bearing on plunge columns
3. Temporary bracing placed at ground level between capping beams

Ground Floor Plan

Scale 1:100



Phase IIIa - Underpinning

1. As capping beam is being placed, underpinning to corners of building can be carried out.
2. Underpinning should be carried out in the normal manner, using a typical underpinning sequence where no two adjacent pins are excavated before 48 hours of dry-packing being placed to the previous section.
3. Beams and needles placed to external walls, bearing on plunge columns
4. Temporary bracing placed at ground level between capping beams

Basement Floor Plan

Scale 1:100

Rev	Date	Amendments
-	21/04/2020	First issue for comment

Client: Mr Nicholas Goulandris

Project: 69 Avenue Road

Title : Basement Scheme for BIA

Job Number 200203	Date Apr 20
Dwg Number TW-04	Rev -
Drawn SB	Ch'kd PH
Scale As shown @ A1	

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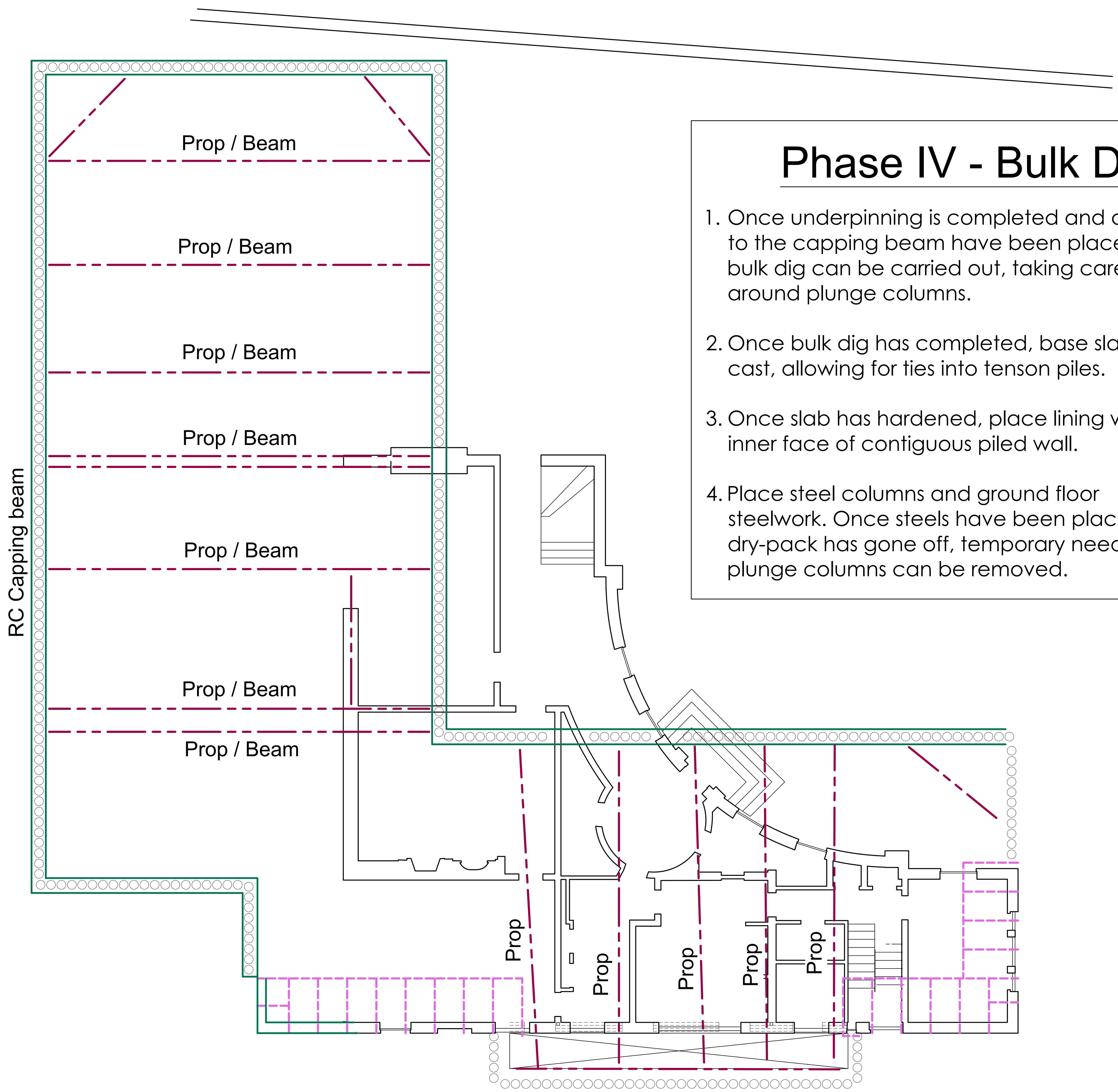
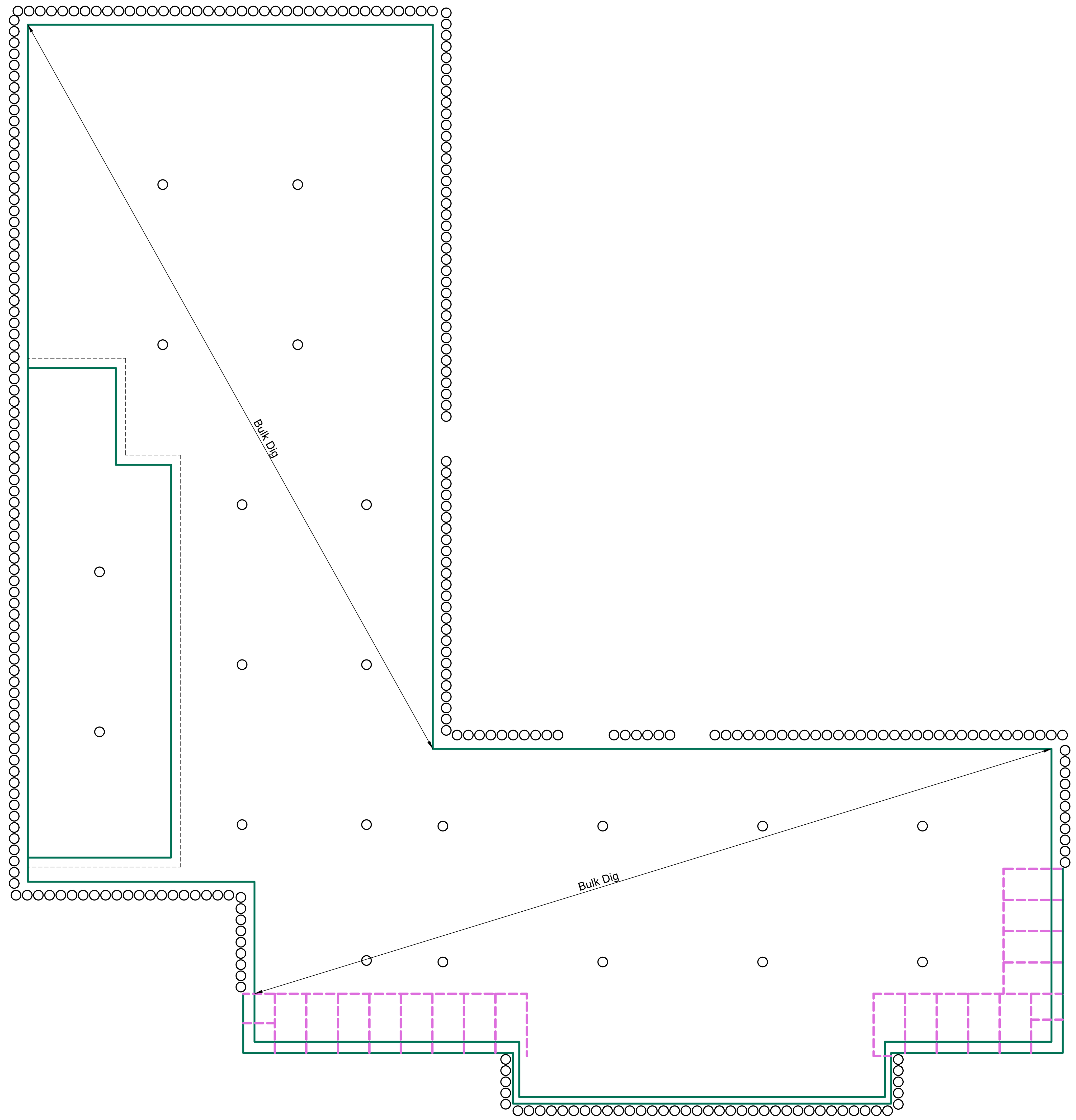


Ground Floor Plan
Scale 1:100

Phase IV - Bulk Dig

1. Once underpinning is completed and all props to the capping beam have been placed, the bulk dig can be carried out, taking care around plunge columns.
2. Once bulk dig has completed, base slab to be cast, allowing for ties into tenson piles.
3. Once slab has hardened, place lining walls to inner face of contiguous piled wall.
4. Place steel columns and ground floor steelwork. Once steels have been placed and dry-pack has gone off, temporary needles and plunge columns can be removed.



Basement Floor Plan
Scale 1:100

Rev	Date	Amendments
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris
Project: 69 Avenue Road
Title : Basement Scheme for BIA

Job Number 200203	Date Apr 20
Dwg Number TW-05	Rev -
Drawn SB	Ch'kd PH
Scale As shown @ A1	

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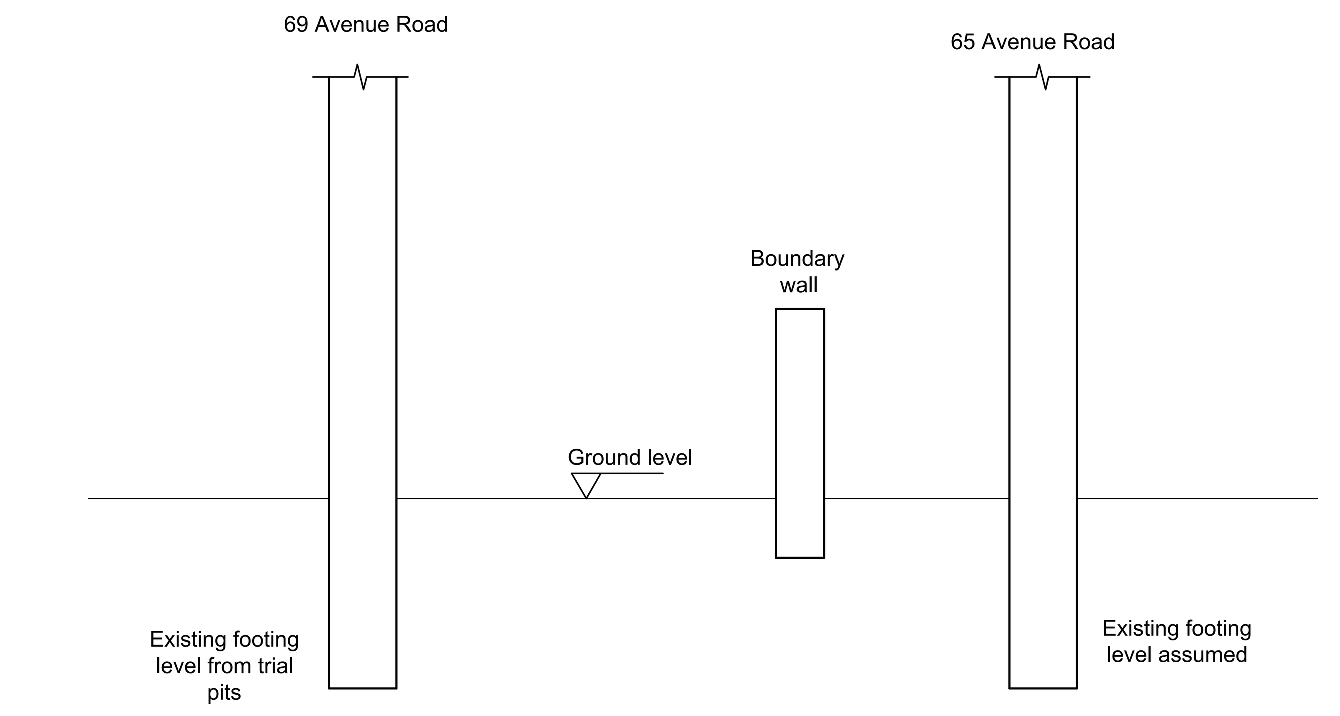
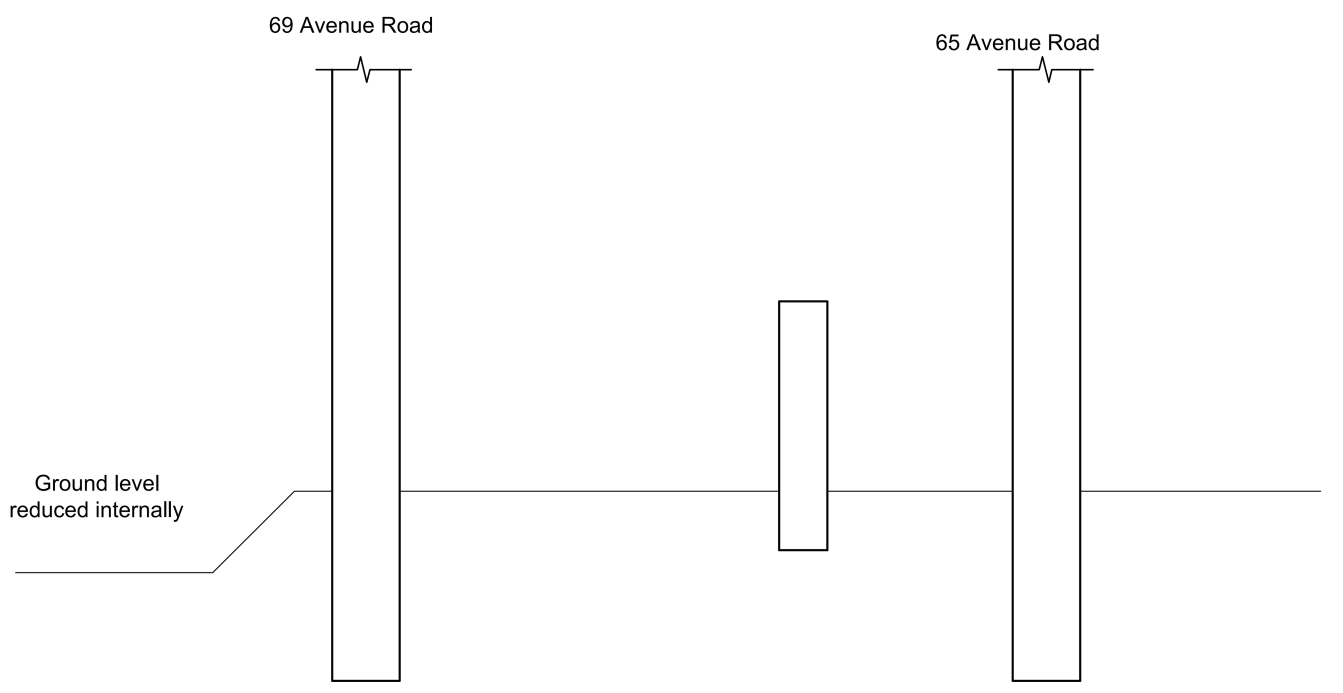
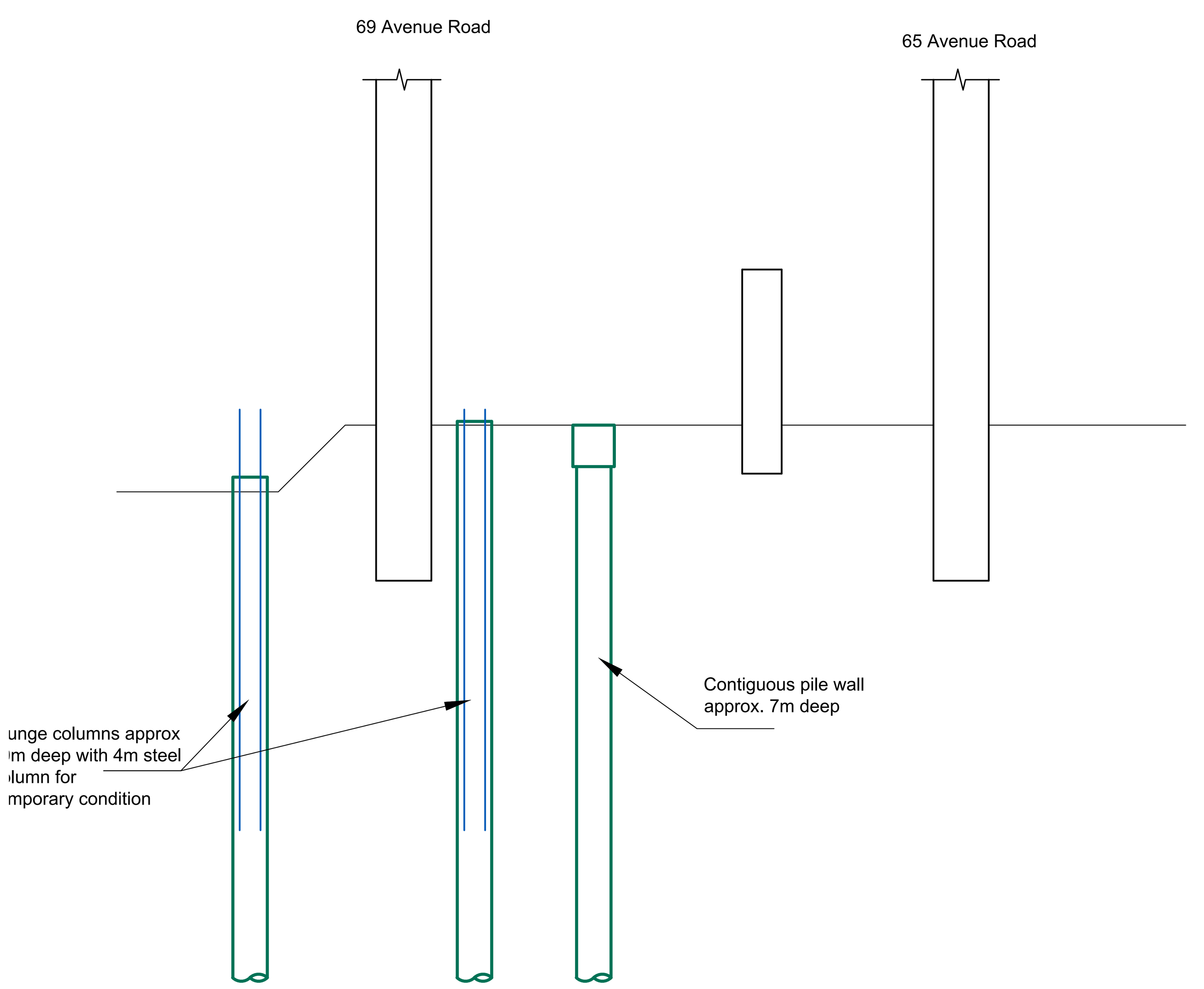


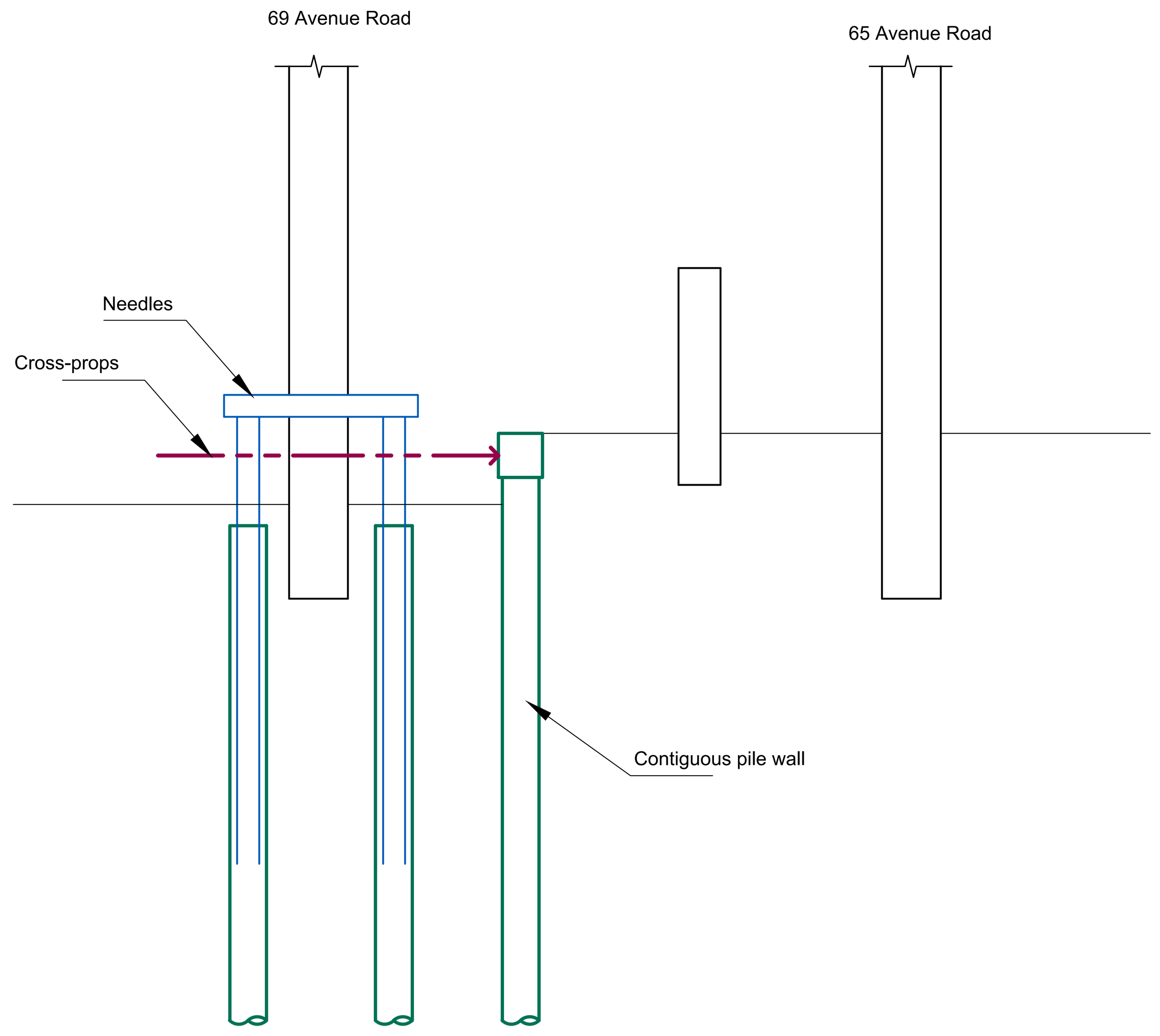
Phase 0 - Existing



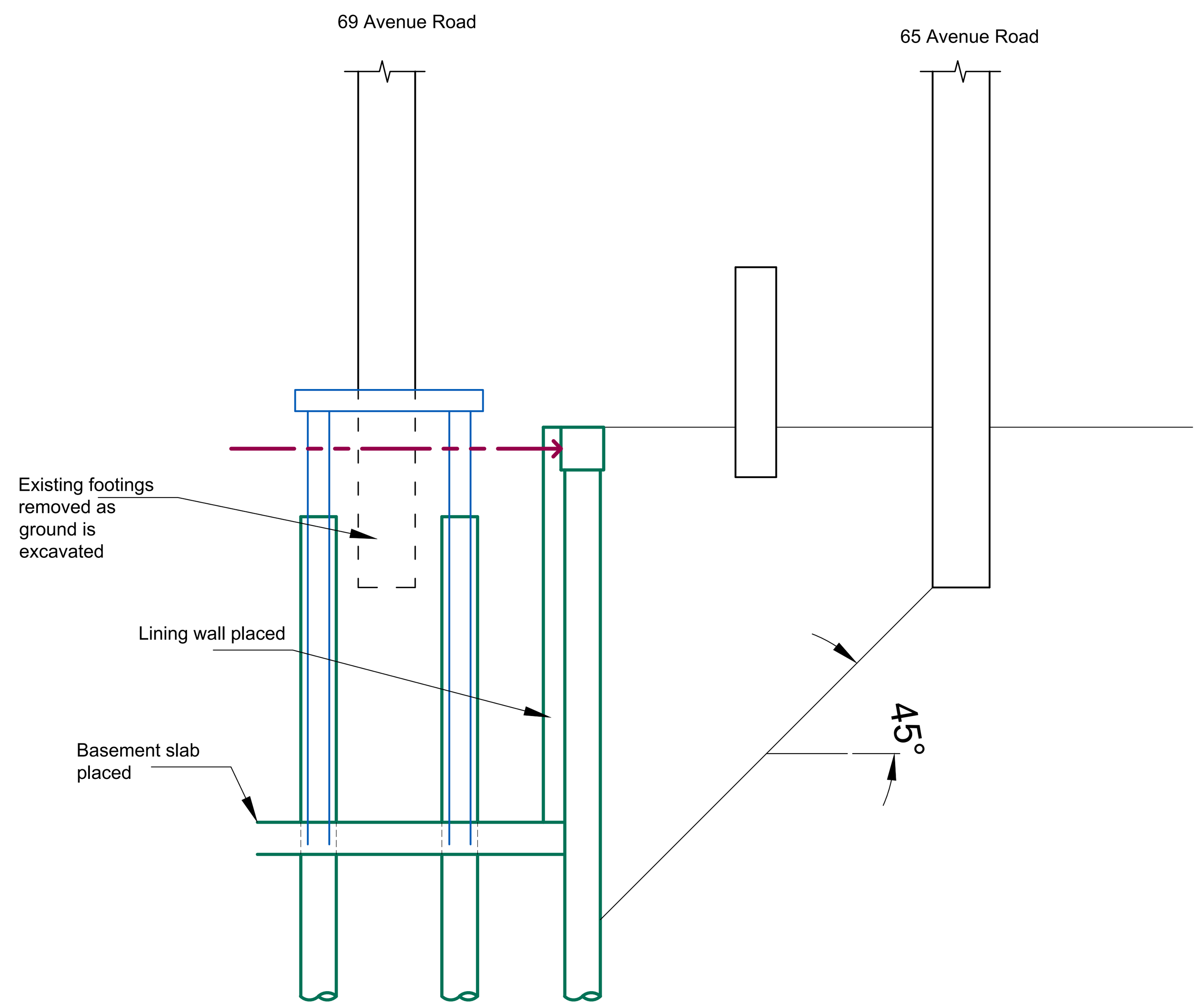
Phase I - Demolition



Phase II - Piling



Phase III - Capping Beam & Propping



Phase IV - Bulk Dig

Rev	Date	Amendments
1	27/11/2020	Pile length altered
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris
Project: 69 Avenue Road
Title : Temporary Works Scheme for BIA - 65/69 Avenue Road

Job Number	Date
200203	Apr 20
Dwg Number: TW-06	Rev: 1
Drawn: SB	Ch'kd: PH
Scale: 1:50@ A1	

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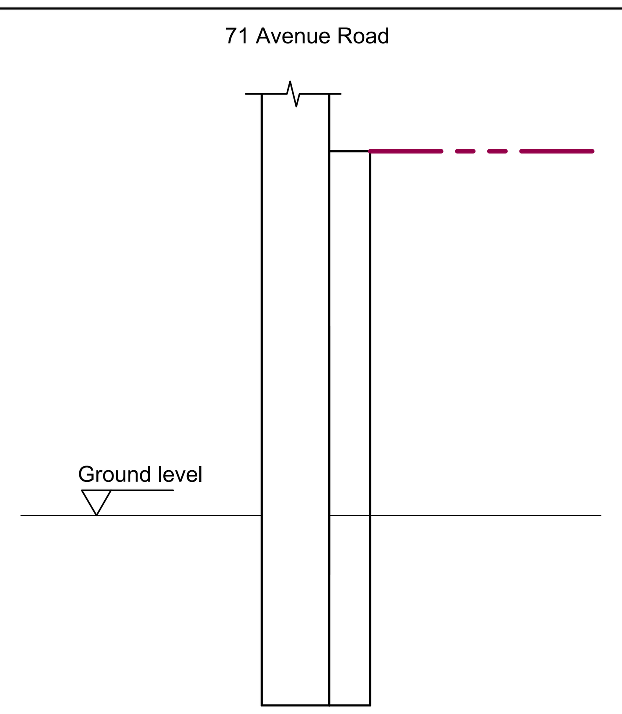
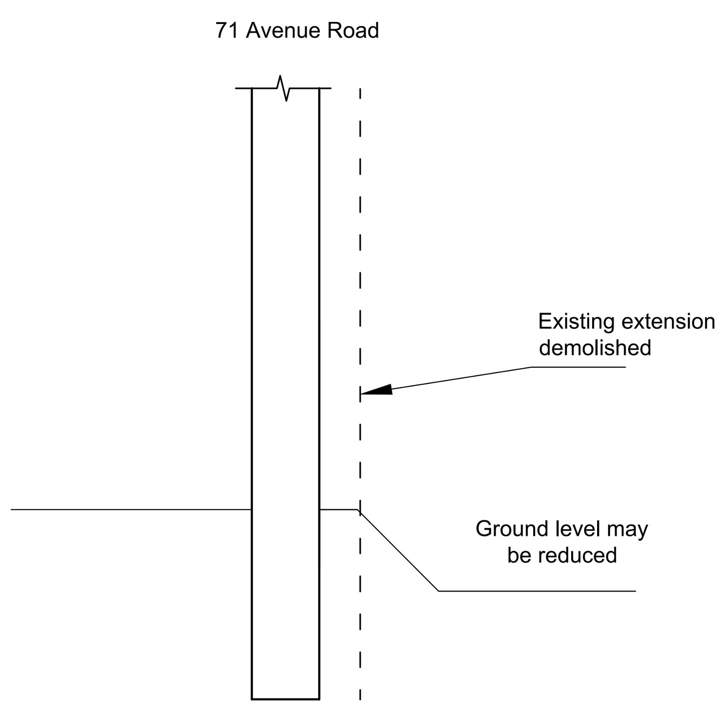
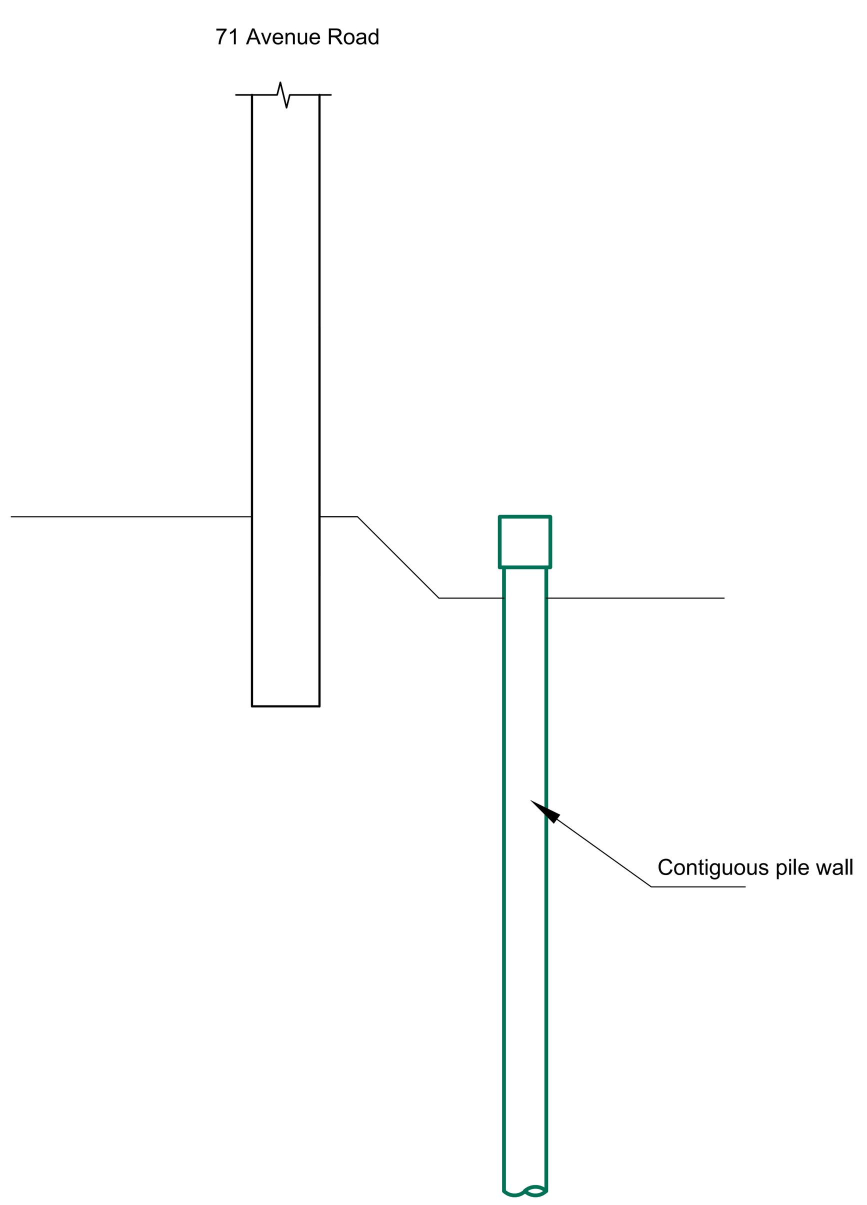


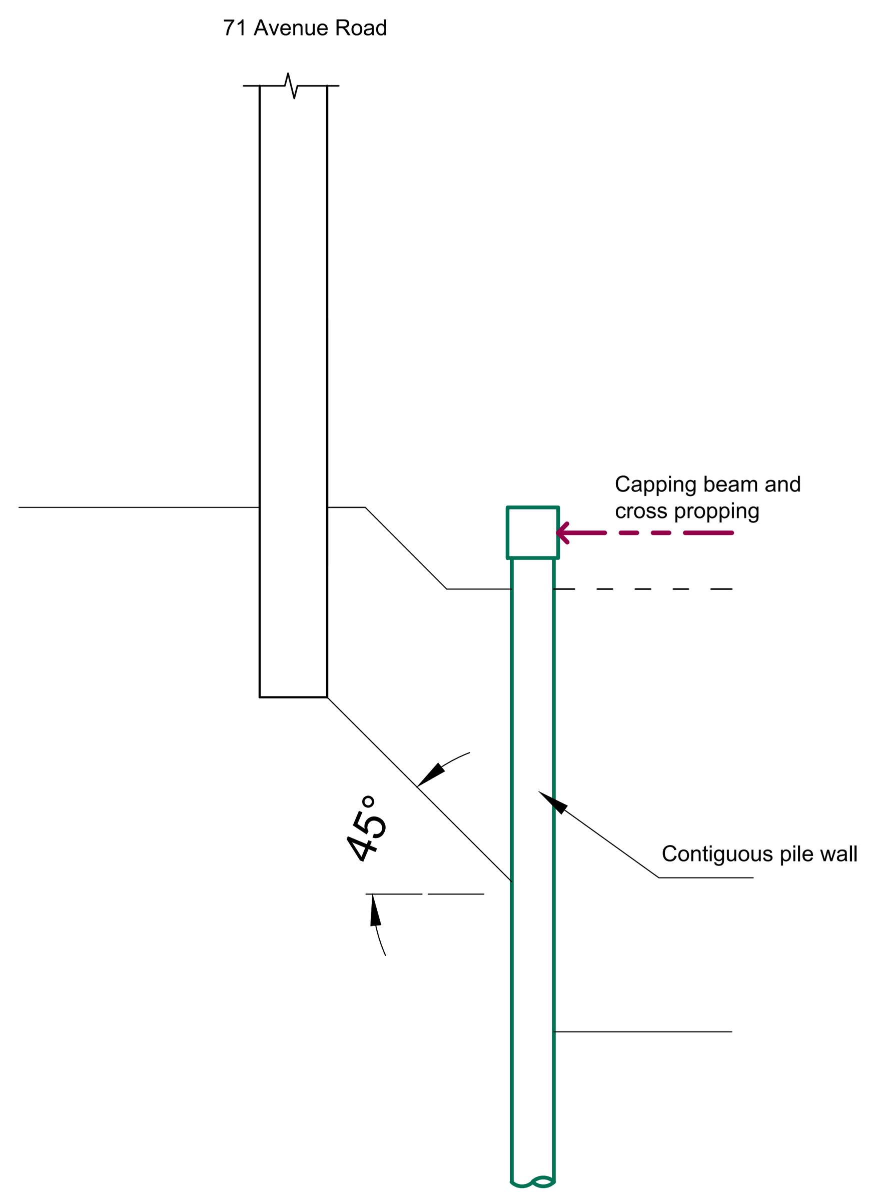
Phase 0 - Existing



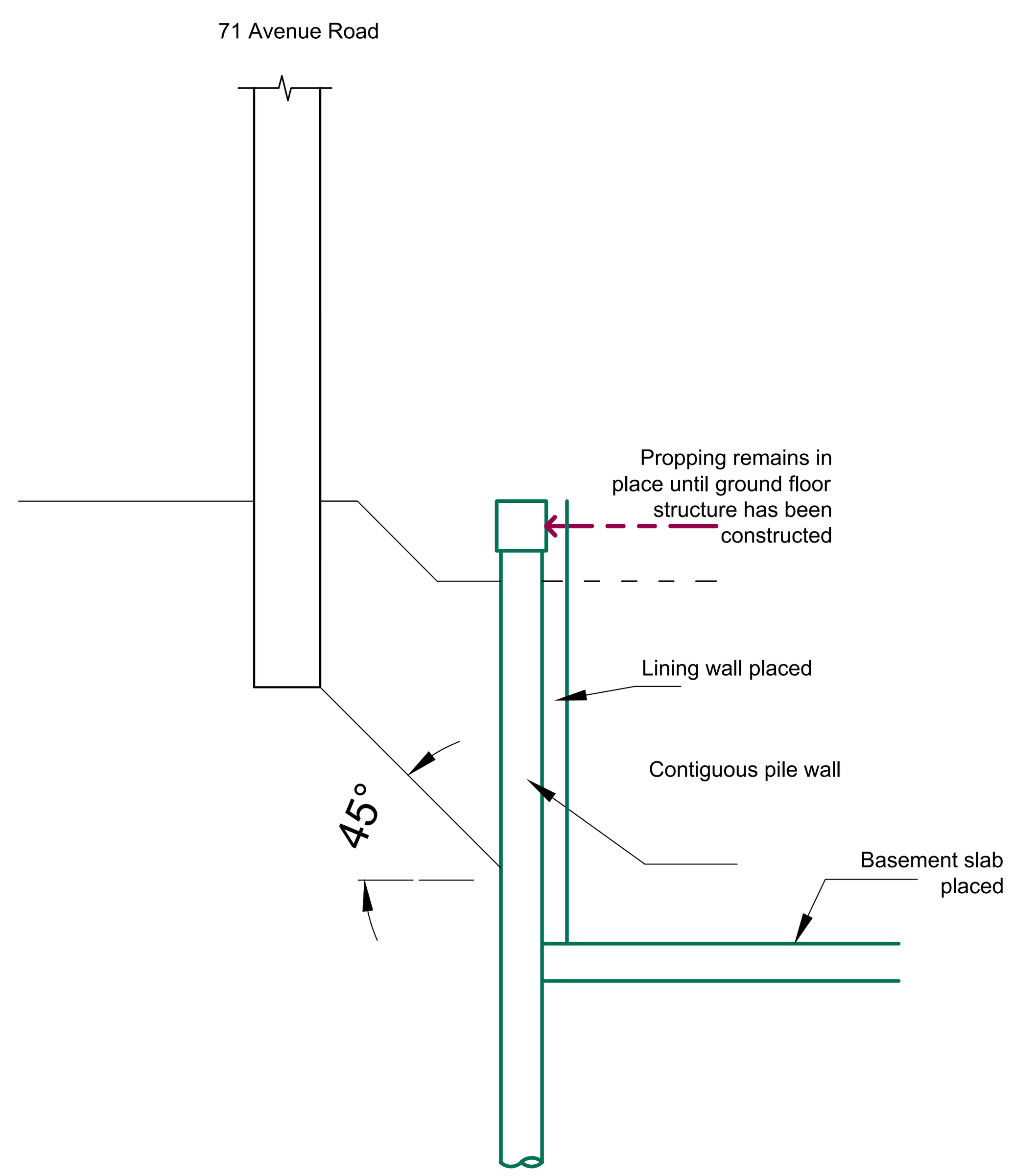
Phase I - Demolition



Phase II - Piling



Phase III - Capping Beam & Propping



Phase IV - Bulk Dig

Rev	Date	Amendments
-	28/04/2020	First issue for comment

Client: Mr Nicholas Goulandris

Project: 69 Avenue Road

Title : Temporary Works Scheme for BIA - 69/71 Avenue Road

Job Number 200203	Date Apr 20
Dwg Number TW-07	Rev -
Drawn SB	Ch'kd PH
Scale 1:50@ A1	

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Appendix E: Method Statement

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Basement Method Statement

Site Details
69 Avenue Road
London
NW8 6HP

CROFT STRUCTURAL ENGINEERS

Basement Sequence for 69 Avenue Road

1. Basement Formation Suggested Method Statement

1.1. This method statement provides an approach that will allow the basement design to be correctly considered during design, temporary design & construction. This statement is for planning purposes only. Once planning and Building control has been completed the responsibility for the temporary works will transfer to the Contractor during works on site.

1.2. This sequence has been written by a Chartered Engineer. The sequencing has been developed using guidance from ASUC (Association of Specialist Underpinning Contractors).

1.3. This method has been produced to demonstrate the feasibility of the works at planning and for inclusion in the Basement Impact Assessment at Planning for Camden.

2. Enabling Works

2.1. The site is to be hoarded to prevent unauthorised public access.

2.2. Licences for skips and conveyors should be posted on the hoarding.

2.3. Dewater: It is possible that water may be encountered on site during the works.

2.3.1. Localised removal of water may be required to deal with rain from perched water or localised water. This is to be dealt with by localised pumping. Typically achieved by a small sump pump in a bucket.

2.4. On commencement of construction, the contractor will determine the foundation type, width and depth. Any discrepancies will be reported to the structural engineer in order that the detailed design may be modified as necessary.

3. Basement Sequencing

The majority of the basement will be piled, with some underpinning on the south side of the building. Temporary props will be provided along the head of the piles in the temporary condition. Before the base is cast cross props are needed. The base/ground slab provides propping in the final condition. The central soil mass is to be removed in portions (thirds but no greater than 8m) and cross propping subsequently added as the central soil ass is removed

3.1. Phase I – Demolition of existing extension

See Drawing TW-02

1. Existing extension and rear bays to be demolished. Support of roof to be maintained and stability of existing walls to be checked. Temporary bracing may be required.

2. Existing ground floor stripped out. Piling mat placed. Internally, ground level reduced by 600mm to allow for piling rig access.

3. Services to be determined at pile locations.

3.2. Phase II – Piling

See Drawing TW-03

1. Piling commences in 3 locations: Contiguous Piled Walls, Tension Piles and Plunge Column Piles.

2. Plunge column sequence:
- Pour concrete
- Take cube tests 2 per pour, this may be less than 2 per pile
- Insert reinforcement
- Insert Plunge columns
- Tie tops of plunge columns together with 150x90 PFC

3. Piling commences on internal spaces. Some additional propping of first floor may be required to allow for holes.

4. Integrity testing of piles

3.3. Phase III – Underpinning to existing walls

See Drawing TW-04

1. Reinforced Concrete Capping beam placed to top of contiguous piled wall. Beam to be continuous where it crosses through external walls

2. Beams and needles placed to external walls, bearing on plunge columns

3. Temporary bracing placed at ground level between capping beams

3.4. Phase III – Underpinning to existing walls

See Drawing TW-04

3.4.1. As capping beam is being placed, underpinning to corners of building can be carried out.

3.4.2. Underpinning should be carried out in the normal manner, using a typical underpinning sequence where no two adjacent pins are excavated before 48 hours of dry-packing being placed to the previous section.

3.4.3. Beams and needles placed to external walls, bearing on plunge columns

3.4.4. Temporary bracing placed at ground level between capping beams

3.5. Phase IV – Bulk Dig

See Drawing TW-05

3.5.1. Once underpinning is completed and all props to the capping beam have been placed, the bulk dig can be carried out, taking care around plunge columns.

3.5.2. Once bulk dig has completed, base slab to be cast, allowing for ties into tension piles.

3.5.3. Once slab has hardened, place lining walls to inner face of contiguous piled wall.

3.5.4. Place steel columns and ground floor steelwork. Once steels have been placed and dry-pack has gone off, temporary needles and plunge columns can be removed.

3.5.5. Place below-slab drainage. Croft recommends that all drainage is encased in concrete below the slab and cast monolithically with the slab. Placing drainage on pea shingle below the slab allows greater penetration for water ingress.

3.5.6. Place reinforcement for basement slab.

3.5.7. Building Control Officer and Engineer are to be informed five working days before reinforcement is ready and invited for inspection.

3.5.8. Once inspected, pour concrete.

3.5.9. Check 14-day cube test results for slab. If concrete is of sufficient strength remove lower level props.

3.5.10. One ground floor grillage is completed and ground floor diaphragm has been constructed removed temporary top props.