Douglas PARTNERS

GROUNDED EXPERTISE

Report on Geotechnical Desktop Study

Proposed Mixed-Use Development

5 Blue Street, North Sydney NSW

Prepared for Zurich Australian Insurance Properties Pty Ltd

Project 237286.00

25 February 2026

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Document History

Details

Project No.	237286.00
Document Title	Report on Geotechnical Desktop Study
Site Address	5 Blue Street, North Sydney NSW
Report Prepared For	Zurich Australian Insurance Properties Pty Ltd
Filename	237286.00.R.001.Rev1

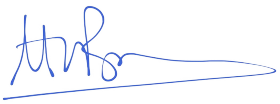
Status and Review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Peter Hunt	Hugh Burbidge	19 February
Revision 1	Peter Hunt	Hugh Burbidge	25 February

Distribution of Copies

Status	Issued to
Revision 0	Zurich Australian Insurance Properties Pty Ltd
Revision 1	Zurich Australian Insurance Properties Pty Ltd

The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

Signature		Date
Author		25 February
Reviewer		25 February

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Report on Geotechnical Desktop Study
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1. Introduction

This report prepared by Douglas Partners Pty Ltd (Douglas) presents the results of a geotechnical desktop study undertaken for a proposed mixed-use development at 5 Blue Street, North Sydney NSW (the site). The desktop study was commissioned by email instructing to proceed from James Hallows on behalf of Zurich Australian Insurance Properties Pty Ltd and was undertaken in accordance with Douglas' proposal 237286.00.P.001.Rev0 dated 20 November 2025.

It is understood that the existing site is to be redeveloped with the site located over the existing North Sydney train station/platforms. The current concept design includes the adaptive reuse of the existing structure.

The aim of the geotechnical desktop assessment comprises:

- A review of provided and available documentation.
- A site inspection to assess the exposed rock quality at the existing carpark level and within the station (from the platforms).
- Preparation of a geotechnical report including the following:
  - Preparation of a conceptual site ground profile.
  - Comments on acid sulphate soil and salinity.
  - Foundation conditions and parameters.
  - Parameters for permanent ground anchoring along Blue Street.
  - Groundwater comments.
  - Comments on seismic design parameters.

It is understood that the report will be used to support a State Significant Development (SSD-66745743) addressing the Planning Secretary's Environment Assessment Requirements (SEARs) Ground and Groundwater Conditions (SEARs Item 13). This report must be read in conjunction with all appendices including the notes provided in Appendix A.

2. Site description

The site is located within the North Sydney Local Government area approximately 2km north of the Sydney Central Business District (refer Figure 1). The site is identified as Lot 4 DP 1134234 and has a 3007 m^2 footage. The site is currently occupied by a multi-storey commercial building. North Sydney Station owned by the Transport Asset Manager (TAM) lies to the east of the site. The platforms extend across and beneath most of the site footprint. Based on client provided drawings, the lowest basement level or podium level sits at a level of RL51.3 m relative to

Australian Height Datum (AHD) and is approximately 4.5 m above the North Sydney station platforms (at approximately RL46.7 m AHD).

Figure 1: Site location (red polygon shaded yellow)

The regional topography slopes south-east towards Lavender Bay. Along Blue Street the topography dips from approximately RL60 m AHD in the north-western corner of the site, to around RL56 m AHD in the north-eastern corner of the site. The southern driveway is level with Blues Point Road (approximately RL56 m AHD) and runs along the southern boundary of the site, which is elevated from the adjacent southern properties which are at approximately RL48 m to RL52 m AHD. Within the site boundary, the site has been excavated down for the station to around RL45.2 m AHD (rail level shown at RL45.7 m AHD).

3. Published data

3.1 Geology

Reference to the Sydney 1:100 000 Series Geological Sheet 9130 indicates that the site is underlain by the Triassic Aged Hawkesbury Sandstone (refer Figure 2). Hawkesbury Sandstone typically consists of medium to coarse grained quartz sandstone with minor shale and laminite lenses. The Hawkesbury Sandstone typically contains orthogonal joint sets:

- Set 1 – NNE striking joints dipping 75° to 90° to the west and east, spaced between 1 to 10 m, but can be spaced closer in the joint swarms. These joints are very prominent and are persistent over many metres, both horizontally and vertically.

- Set 2 – ESE striking joints dipping 75° to 90° to the north and south. These joints are less common, generally strata bound (discontinuous).

Figure 2: Extract from Sydney 1:100,000 Geological Series Sheet 9130 (Site shown by blue marker).

Low angle (25° to 30°) thrust faults, typically dipping to the east-west, are also relatively common. It is noted that the long side of the site is orientated sub-parallel to east-west and therefore the eastern and western site boundaries are orientated sub-parallel to Joint Set 1.

3.2 Soil Landscape

Reference to the Sydney 1:100 000 Soil Landscape Sheet indicates that the site is underlain by Gymea erosional soils which typically comprises shallow to moderately deep yellow earths and earthy sands on crests and inside of benches.

3.3 Hydrogeology

A search of the publicly available registered groundwater bore database indicated that there is one registered groundwater bore (GW107764) within 500 m of the site, however, no useful information on the groundwater level is included in the published database.

Based on the regional topography and the inferred flow direction of nearby water courses, the anticipated flow direction of groundwater beneath the site is to the south, towards Lavender Bay.

3.4 Acid Sulfate Soils

Reference to the NSW Department of Environment and Climate Change (DECC) 1:25,000 Acid Sulfate Soil Risk Mapping indicates that the site is not mapped in an area of known acid sulfate soil risk. Furthermore, based on the location and elevation of the site, it is within an area that is not likely to be affected by acid sulfate soils.

3.5 Salinity

In reference to the Map of Salinity Potential in Western Sydney, the site is not situated within the mapped salinity boundaries and is therefore not considered to be within an area with potential for dryland salinity. Furthermore, based on the location and elevation of the site, it is within an area that is not likely to be affected by salinity.

3.6 Previous Douglas Geotechnical Investigations

Douglas has undertaken previous investigations in the area. The location of some of the previous investigations used for this report are shown in Figure 3.

Figure 3: Location of previous Douglas investigations

4. Site Inspection

A site walkover of the two basement levels and the station from the platforms was carried out by a geotechnical engineer on 28 January 2026. Photos from the site walkover are presented in Appendix B. Key observations are summarised below:

- The site gently slopes to the south-east (refer Photo 1 to 3).
- The neighbours to the south are at a much lower elevation and have medium to high strength rock outcrops along the boundary (refer Photo 4). The top of rock along the southern boundary dips to the east, with medium to high strength rock exposed at podium level

(station roof) at the western end but diminishing to only a few metres high at the eastern end of the site boundary.

- In the upper basement level of the existing building, the following observations were made:
  - The entry level of the upper basement does not contain any exposed rock, but as the basement slab ramps down to the west there is rock observed in the northern boundary wall. The rock was considered to be low to medium and medium strength (refer Photo 5).
  - Along the western boundary of the upper basement level, medium strength rock was observed (refer Photos 6 and 7). This extended across most of the western boundary but dipping in the last 10 m from the southern boundary. Rock strata with cross-bedding dipping at approximately 20° to 30° was observed within the exposed rock face (refer Photo 8).
  - A brick wall and ventilation ducts covered the southern boundary. Based on the dipping rock level in the southern wall near this boundary, it is suspected that there is no rock exposed behind the southern wall on the upper basement level.
- In the lower basement level of the existing building, the following observations were made:
  - The entry level of the lower basement has a small amount of rock exposed along the northern boundary (refer Photo 9). The top of rock rises to the west in steps, roughly following the slope of Blue Street until it is the full height of the basement in the western third of the northern boundary. The rock consisted predominantly of medium strength sandstone, with some low to medium strength bands (refer Photo 10). Both NNE and ESE striking joints were observed in the rock face as described in Section 3.1. Additional 40° to 45° dipping discontinuous joints, striking at between 80° and 100° were observed.
  - All other boundary walls (besides the northern boundary) on the lower basement level were brick walls which obstructed view of the strata behind.
- In the station, medium to high strength rock was observed in the excavation face (refer Photo 11) along the northern boundary (across from Platform 1) for the full height from the track level up to the podium level (station roof). The northern most tunnel at the western end of the station was observed to be concrete lined (refer Photo 12).
- In the station, medium to high strength rock was observed in the excavation face along the southern boundary (across from Platform 4), however it was only full height in the western third of the site (refer Photo 16), with the station area open to the outside where the rock dipped below the podium level in the eastern end of the southern boundary (refer Photo 15). The southern most tunnel at the western end of the station was observed to be concrete lined like the other tunnels (refer Photo 17).
- Columns supporting the podium level and building above are spread throughout the station along the boundaries, platforms and between the two middle tracks (refer Photo 14). Many of the perimeter columns were observed to have widened foundations beneath the columns (refer Photo 18).

5. Preliminary Geotechnical Profile

Based on a review of the available information, nearby investigation and the site inspection, the subsurface profile from natural ground level can be summarised as follows:

Topsoil and Fill:	Typically firm to stiff sandy clay/clayey silt to estimated depths of between 1.5 m and 3.0 m below natural ground level.
Residual Soils	Typically low to medium plasticity, firm to stiff sandy clay/clay residual soils to an estimated depth of between 1.5 m and 3.0 m below natural ground level along Blue Street.
Sandstone	Typically medium strength, grading to medium to high and stronger with depth.

As most of the site has already been excavated down to rail level, it is expected that the platforms are sitting on fill, possibly consisting of ripped sandstone. Medium to high strength or even high strength sandstone is expected to be encountered approximately 0.5 m below track level at the original bulk excavation level for the station.

No free groundwater was observed in the station excavation during the site inspection with the station being a drained excavation. Nearby projects suggest the groundwater could be located approximately 2m to 3 m below the existing station platforms, however this is unlikely to be the regional groundwater table. The regional groundwater table is likely to be well below excavation level due to drained (building) basements in the vicinity of the site and the sloping topography down towards Lavender Bay to the south. Note, seepage along the top of rock should still be expected.

6. Comments

6.1 Key Geotechnical Issues

The key geotechnical issues identified for this site are:

- The impact of the adaptive reuse works on the station/rail easement.
- Reuse of existing structure foundations and the reactivation of settlement due to increased applied loads.
- Transfer of lateral loads into the ground using the existing structure.
- The difficulty accessing and assessing the existing foundations within the station/rail corridor with limited interruption to the working station.
- Assessing the support of the existing soil retaining structures around the perimeter of the site.

These items are discussed further in the following sections

6.2 Excavation Support

The site is located over the existing active train station, with the existing structure to be reused for the proposed redevelopment. As such, it is understood that no further excavation works are proposed.

The site currently has a number of existing retaining walls supporting soil around the northern, western and part of the southern boundaries. Based on the site visit and the drawings received, it is anticipated that the existing retaining walls are supported at the toe by rock and by the adjoining slabs.

These existing walls should be assessed by a structural engineer to confirm whether they can remain in place or whether they need to be replaced based on their age and design life. It is suggested that the walls should be assessed based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 1. Replacement retaining walls can also be designed based on these parameters provided they are cantilevered or have one row of support. 'Active' earth pressure coefficient (K_a) values may be used where some movement is acceptable, however, an 'at rest' earth pressure coefficient (K_o) should be used where movement of the excavation face needs to be limited.

Table 1: Recommended Design Parameters for Shoring Systems

Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_o)
Fill	20	0.35	0.50
Residual soils and weathered rock	20	0.3	0.45
Medium strength or higher sandstone	24	0*	0*

Notes: The values above assume a level surface behind the wall.
*It is assumed that the rock mass is free of adverse dipping joints and seams. Anchoring or spot bolts may be required where jointing is unfavourable.
It should also be noted that the K_a and the K_o designs will not prevent stress relief movement in the rock.

If the current design is to be modified or replaced using a retaining wall with multiple rows of slabs / struts / propping or multiple anchors, a rectangular or trapezoidal pressure distribution should be adopted. When adopting a trapezoidal distribution, the pressure distribution should increase from zero at the surface to the maximum value at a depth of 0.25 H. Where there are no movement sensitive structures near the excavation, the maximum pressure can also be calculated using 4H kPa (where H equals the depth to the top of self-supporting medium strength or stronger rock). Where the wall movement is to be minimised (i.e. close to adjacent buildings) the maximum pressure can be calculated using 6H kPa. For the heritage building, the tunnels or other more movement sensitive structures, where it is critical that deformation is controlled, it may be necessary to calculate the maximum pressure using 8H kPa.

The earth pressure loading described above does not include earthquake loads, hydrostatic pressures or surcharge loads. The existing retaining wall's drainage measures should be maintained to prevent hydrostatic pressures acting on the walls. If no drainage measures are in place, they should be assessed with a full hydrostatic pressure to surface. Additional surcharge loads such construction loads, traffic loads etc., must be allowed for in the assessment, applied as a uniform rectangular earth pressure distribution over the depth of influence.

6.2.1.1 Vertical Excavations in Sandstone

The existing basement and station excavation contains vertically excavated medium strength or stronger sandstone. Generally, vertical excavations in medium strength or stronger sandstone are considered self-supporting provided there are no adversely oriented joints or other defects present. The existing excavation faces should be inspected by a suitably qualified geotechnical engineer / engineering geologist to confirm they are and will remain stable for the design life of the redevelopment. The purpose of the inspections is to assess the stability of the rock faces and safety of the works. The geotechnical engineer / engineering geologist will map the exposed faces and comment on the need for any further stabilisation measures.

It is noted that the two major joint sets (NNE and ESE as described in Section 3.1) can form wedges or feather edges (slivers of rock) in the excavation faces. The NNE joint set is between 5° to 10° off the orientation of the western excavation face and has the potential for large adverse rock wedges. The ESE joint set is between 5° to 10° degrees off the orientation of the northern and southern excavation faces. These potentially adversely oriented joints, however, would likely have been identified during the excavation of the station and suitably stabilised at the time. In this case, consideration should be given to the serviceability of any existing ground support and its design life. These joints, however, still have the potential to reduce the lateral capacity of the rock. Assessment of each individual case will be required where these conditions exist.

6.2.1.2 Anchor / Rockbolt Design

Post-tensioned ground anchors, rockbolts and/or dowels (support elements) can be used to laterally support existing retaining walls, unstable rock masses or to support lateral loads. These support elements should be bonded in the stronger rock, inclined as required, but preferably not steeper than 30° below the horizontal. An allowable bond stress of 350 kPa or an ultimate bond stress of 800 kPa may be used in medium strength or stronger sandstone for preliminary design purposes for both temporary and permanent anchors/rockbolts. The bond stress should be confirmed by pull-out tests prior to installation of support elements. Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the support system and method of installation) are used and that the support element holes are carefully cleaned prior to grouting.

The anchors should be designed and installed as per the relevant standards and specifications (i.e. AS4678 and TS02163). Where permanent anchors/rockbolts are proposed, they will need to have adequate double corrosion protection measures installed to suit the design life.

Where permanent anchors that extend across the boundary are relied upon, a permanent easement will be required to maintain the long-term capacity of the anchors.

6.3 Ground-borne Vibration

It will be necessary to use appropriate methods and equipment to keep ground-borne vibration generated during demolition work within acceptable limits at the station platforms, adjacent buildings, utilities, structures and infrastructure.

The level of acceptable vibration is dependent on various factors including the type of structure (e.g. reinforced concrete or brick structures etc.), its structural condition, the frequency range of vibration produced by the construction equipment, the intensity of the vibration, natural

frequency of the structure and the vibration transmitting medium. As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of excavation. The trial may indicate that smaller or different types of excavation equipment is needed.

Ground-borne vibration can be strongly perceptible to humans at levels above 2.5 mm/s vector sum peak particle velocity (VSPPV). This is generally much lower than the vibration levels required to cause structural damage to buildings. It is suggested that vibrations be provisionally limited to a VSPPV of 8mm/s at the foundation level of existing buildings to protect architectural features and for human comfort considerations.

This provisional level complies with AS/ISO 2631.2 – 2014 and is below the normal building damage threshold level. This vibration limit may need to be reduced for any sensitive buildings and structures (i.e. the church buildings to the south and the rail track).

A Vibration Monitoring Plan (VMP) should be prepared for the site before works commence. The vibration monitoring plan should take into account the specific vibration levels for the site including the rail track infrastructure.

6.4 **Foundations**

The existing foundation arrangement is shown in Appendix C. It is understood that the existing structure has been designed to take all vertical loads down to the base of the station structure with footings designed for an allowable bearing pressure of either 3200 kPa or 2100 kPa depending on the rock encountered (sizing adjusted to match). It is understood that, as the existing structure is being reused, all vertical loads from the new development will be borne by the existing foundations (either as is or with increased loads).

The re-use of existing foundations will require investigation of founding levels and founding rock Class as well as structural condition (concrete strength, reinforcement capacity etc.) and remaining design life. Currently we have no testing of the rock at the depth of the existing foundations. Based on nearby information, it is expected that medium to high strength sandstone would have been encountered at the original bulk excavation level for the station and at the likely level base of the foundations.

Preliminary design parameters for the allowable bearing capacity of the medium to high strength sandstone beneath the existing foundations are given in Table 2. It should be noted that extensive investigation drilling within the site will be required to confirm the rock quality below the proposed founding levels.

Table 2: Recommended Design Parameters for Foundation Design

Foundation Stratum	Allowable End Bearing Pressure (Serviceability) (kPa)	Ultimate End Bearing Pressure (kPa)	Young's Modulus, E (MPa)
Medium to high strength sandstone	6,000	60,000	1200

Notes:
- Allowable end bearing pressures to cause settlement of less than 1% of minimum footing dimension.
- Ultimate parameters are mobilized at large settlements (i.e., >5% foundation width).
- Additional analysis is required to calculate the modulus of subgrade reaction for individual footings.

Footings proportioned based on ultimate bearing pressures given in Table 2 would be expected to experience total settlements of >5% of the footing width and up to 1% settlement for the above allowable bearing pressures under the respective ultimate and serviceability loads. Differential settlements between adjacent columns would be expected to be less than half of this value. It is noted that the footings have already undergone the majority of this settlement based on the existing building loads and analysis will be required to assess the additional settlement for footings designed with higher loads.

Defects such as bedding planes and weak seams can have a significant impact on the performance of footings and should be taken account when assessing the design allowable bearing pressure of the rock. For Douglas to provide confirmation of the allowable bearing pressure of the existing footings subjected to an increase in load, a cored borehole should be drilled through each footing (if permitted by the structural engineer) to a depth below the base of at least 1.5 times the minimum plan dimension of the footing or 2.5 m below the footing base, whichever is shallower. These ('proving') boreholes should be drilled using a geotechnical core barrel to maximise recovery, especially if there are any weak seams encountered, and spoon testing should also be carried out. It will also be necessary to confirm the plan dimension of these footings. This can be done with slot excavations and NDD to expose the top of the footing at a minimum two locations per footing.

6.5 **Lateral Support**

It is understood, based on discussions with the structural engineer, that the existing building relies on the lateral capacity of the rock to the north, west and limited rock to the south of the site, with lateral loads transferred down to the podium level slab directly over the station. Lateral loads acting towards the north are understood to be resisted by the rock mass along the northern boundary. Lateral loads acting towards the west are understood to be resisted by the rock mass along the western boundary, together with the rock around the columns along the northern boundary and four columns along the southern boundary at the western end. Lateral loads acting towards the east are understood to be resisted by the rock around the columns along the northern boundary and four columns along the southern boundary at the western end. Lateral loads acting towards the south are understood to be resisted by permanent rockbolts installed under Blue Street. It is noted that the columns along the northern boundary and four of the columns along the southern boundary at the western end are embedded on three sides within the rock and exposed on the fourth side into the station.

The new development is understood to be designed to follow the same lateral load paths as the existing building.

Passive resistance for lateral loads acting on confined medium or greater strength sandstone (i.e. lateral loads acting perpendicular to the site boundaries into the rock) may be based on a working passive resistance of 3,000 kPa or an ultimate passive resistance of 6,000 kPa provided they are not adversely affected by discontinuities in the rock or have excavations/trenches in the zone of influence. It should be noted that this lateral capacity is limited by the size of the resisting rock mass, which will need to be considered in the detailed design.

The east-west lateral loads acting on the columns along the northern perimeter, and the four columns at the western end of the southern boundary, have a reduced capacity as these loads are acting on rock that is unconfined on one side due to the excavated faces adjacent. The passive resistance for these columns may be based on a working passive resistance of 1,500 kPa or an ultimate passive resistance of 3,000 kPa provided they are not adversely affected by discontinuities in the rock or have additional excavations within the zone of influence. Based on observations during the site visit, there are areas where jointing has impacted the lateral capacity, however, a significant number of these columns were hidden behind billboards so a more detailed appraisal was not possible. As preliminary guidance, the lateral capacity of these columns as a whole should be reduced by a further one third to account to any adverse jointing (including ESE jointing). Furthermore, where this lateral resistance is relied upon and/or increased by the new loads, detailed mapping of the rock faces adjacent to the columns will need to be carried out.

It is not generally recommended to utilise the lateral capacity of rock around the perimeter of sites due to the risk of the rock being removed by neighbouring excavations. Removing the rock would also remove the lateral support potentially causing structural failure. Doing so would be at the Structural Engineer's/Client's risk. Where rock around the perimeter of the site is relied upon for lateral support, a sufficient easement will be essential to make sure that rock remains in place for the life of the building. A detailed assessment will be required to determine the easement dimensions based on the required lateral load to be resisted.

Similarly, where permanent anchors extending across the site boundary are relied upon, a permanent easement will be required to ensure long-term support. That being said, skewed anchors beneath Blue Street would be able to assist in resisting east/west lateral loads and perpendicular anchors beneath Blue Street able to assist in resisting south lateral loads.

6.6 **Earthquake Design**

A preliminary site hazard factor (z) of 0.08 would be appropriate for preliminary design in accordance with Australian Standard AS 1170.4 – 2024 Structural design actions – Part 4: Earthquake actions in Australia.

The site sub-soil class of shallow rock (Be) may be adopted for the site in accordance with AS 1170.4 – 2024.

6.7 **Groundwater**

The existing station excavation is a drained excavation. The proposed redevelopment is to reuse the existing structure located above the station and hence will not impact the groundwater table.

It is further noted that no free groundwater was observed in the station excavation faces during the site inspection. The regional groundwater table is likely to be well below excavation level due to drained (building) basements in the vicinity of the site and the sloping topography down towards Lavender Bay to the south.

No signs of significant seepage were observed during the site visit, however it is expected that water seepage will occur along the top of the soil and rock interface and through any permeable joints and bedding planes within the rock exposed in the existing basement, particularly after wet weather. This seepage is expected to be minor and no more than is experienced by the current development (allowing for some changes due to climate change).

As the proposed development is located over the existing drained station excavation, the redevelopment of the site will have no further potential impact on soil resources and related infrastructure and riparian land on or near the site. It will also have no further impact on groundwater than that of the existing site.

6.8 Dilapidation Surveys

Dilapidation surveys should be carried out on the station structure, adjacent buildings, pavements, utilities and infrastructure that may be affected by excavation, ground-borne vibration and/or demolition works.

Dilapidation surveys should be undertaken before the commencement of any demolition work, to document any existing defects so that claims for damage due to construction related activities can be accurately assessed.

It should be noted that a dilapidation survey for the North Sydney train station may only be possible during a scheduled “rail line shutdown”. Consideration should be given to installing vibration monitors and any other survey or monitoring equipment (such as survey targets, tilt beams etc) during the shutdown.

6.9 Geotechnical Considerations Relating to the Rail Corridor

The site will be subject to Sydney Trains corridor and infrastructure protection zones and is restricted in depth to horizontal and inclined planes. Structural impacts to North Sydney Station and the railway corridor beneath the site are required to be assessed (generally through a geotechnical and structural assessment which is likely to require 3D modelling). The ground will be partly unloaded during demolition causing it to heave which would uplift the tracks slightly and then reloaded during construction causing the tracks to settle again. It is recommended that the structural engineer and the design team familiarise themselves with the TfNSW technical note (TN004:2015) document.

It is understood that the current concept design includes no works within the rail corridor, however, close coordination with Sydney Trains / Transport for NSW will be required for any investigation works and inspections for the existing columns/foundations and rock faces within the rail corridor. Much of the investigation works will likely have to be carried out during a rail shutdown (usually during the night or at weekends), which will need significant planning in advance and a lengthy lead-in to approval.

Monitoring of all the Sydney Trains structures and rail tracks will be required throughout demolition and construction.

7. Recommended Additional Geotechnical Work

Further investigation of the existing footings will be required to confirm their founding conditions. It is also recommended that boreholes be completed during the design stage, both from street level to confirm the soil and weathered profile, and from platform level to confirm the depth and strength of the rock beneath the station.

Detailed mapping of the exposed rock faces within the station will be required in order to assess the lateral load capacity of the columns. This will require removing billboards.

Numerical modelling will be required to predict ground movements due to the unloading and reloading of the existing columns in order to assess differential settlement between footings and the impact on the rail infrastructure.

A Geotechnical Inspection and Test plan (GITP) and a Geotechnical Monitoring Plan (GMP) should be prepared once design has been finalised and prior to commencing with construction.

8. Limitations

Douglas Partners Pty Ltd (Douglas) has prepared this report (or services) for this project at 5 Blue Street, North Sydney NSW in line with Douglas' proposal dated 20 November 2025 and acceptance received from James Hallows on behalf of Zurich Australian Insurance Properties Pty Ltd. The work was carried out under Douglas' Engagement Terms. This report is provided for the exclusive use of Zurich Australian Insurance Properties Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of Douglas, does so entirely at its own risk and without recourse to Douglas for any loss or damage. In preparing this report Douglas has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after Douglas' field testing has been completed.

Douglas' advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by Douglas in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design

advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. Douglas cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by Douglas. This is because this report has been written as advice and opinion rather than instructions for construction.

This report provides specialist advice only and no part of it is considered a Regulated Design under the Design and Building Practitioner Act 2020 (NSW).

Appendix A

About this Report

About this Report

October 2024

Introduction

These notes have been provided to amplify Douglas' report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

Douglas' reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, Douglas will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, Douglas cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, Douglas will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, Douglas requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. Douglas would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

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Appendix B

Site Photographs

Photo 1: Photograph of the site looking south-east from the intersection of Blue Street and Blues Point Road

Photo 2: Photograph of the site at street level looking south-west

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	1
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

Photo 3: Photograph of the driveway along the southern boundary of the site

Photo 4: Photograph of the site from neighbours southern property looking north-west

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	2
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

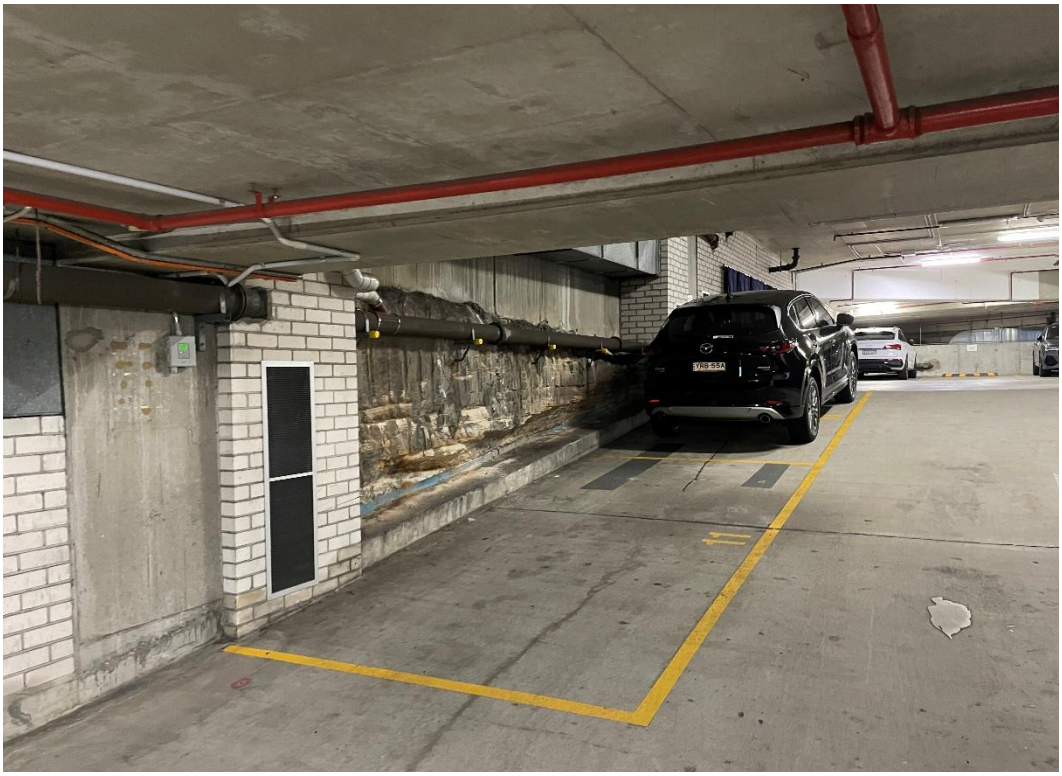


Photo 5: Photograph of the rock outcrop along the northern wall of the upper basement level

Photo 6: Photograph of the rock outcrop along the western wall (northern end) of the upper basement level

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	3
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

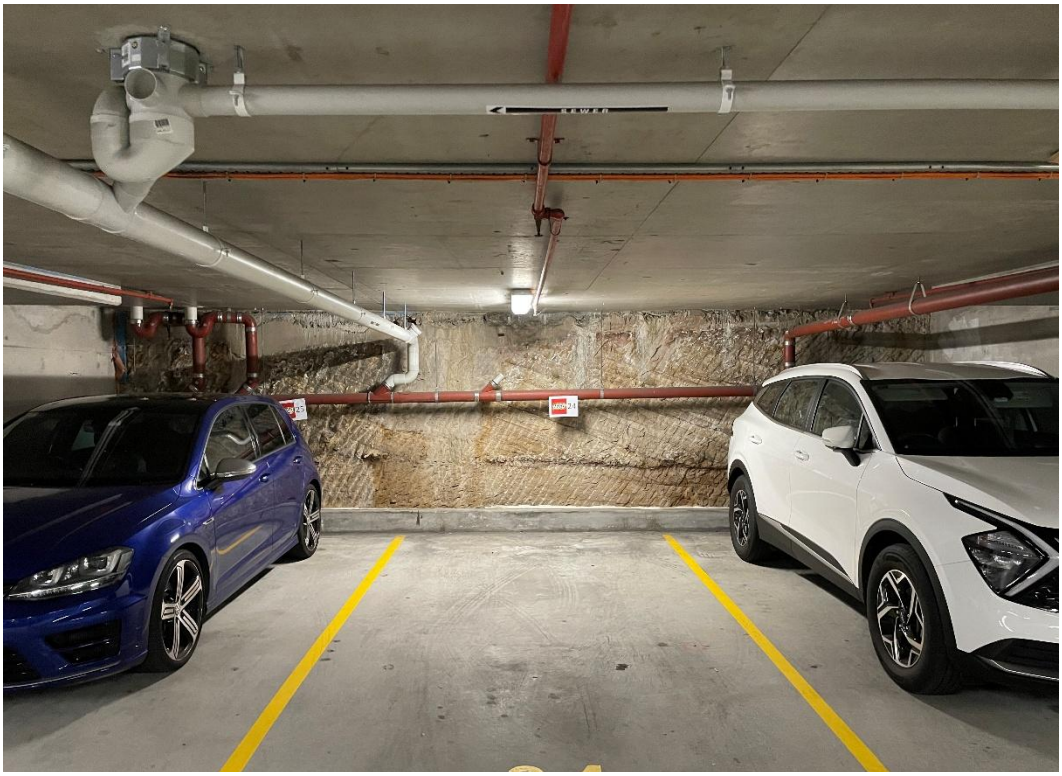


Photo 7: Photograph of the rock outcrop along the western wall (middle) of the upper basement level

Photo 8: Photograph of the rock outcrop along the western wall of the upper basement level showing cross-bedding

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	4
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

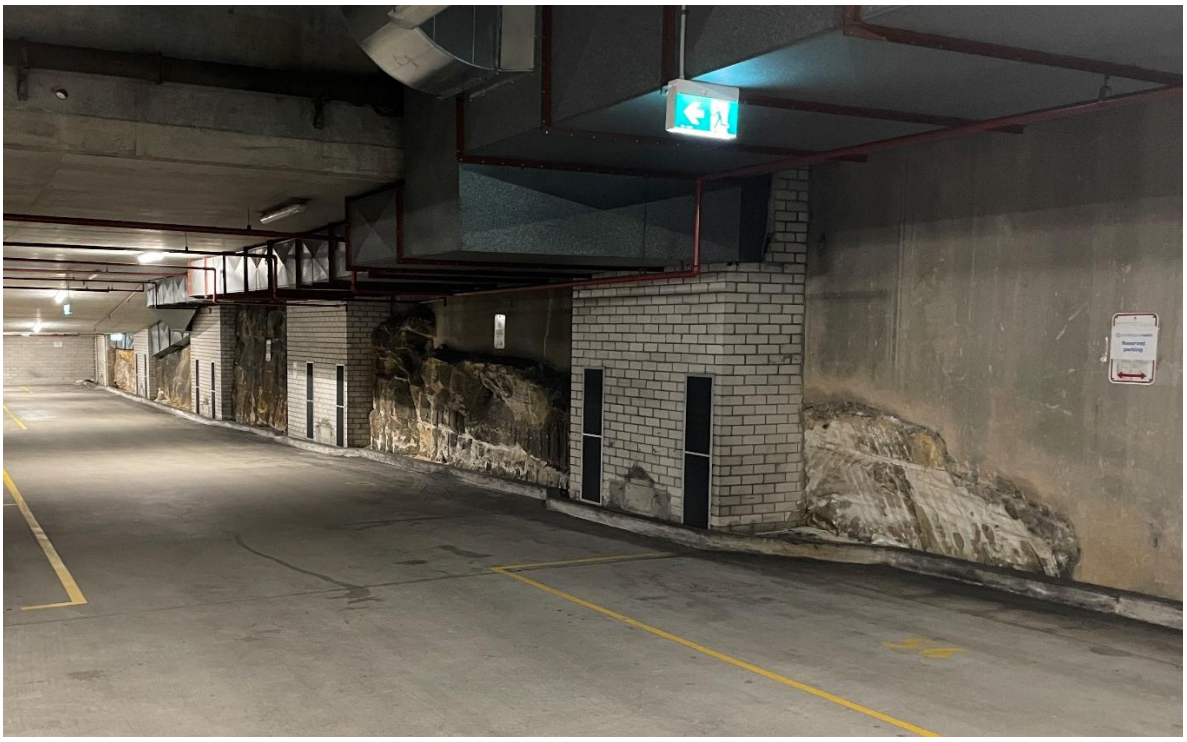


Photo 9: Photograph of the rock outcrop along the northern wall of the lower basement level

Photo 10: Photograph of the rock outcrop at the western end along the northern wall of the lower basement level

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	5
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

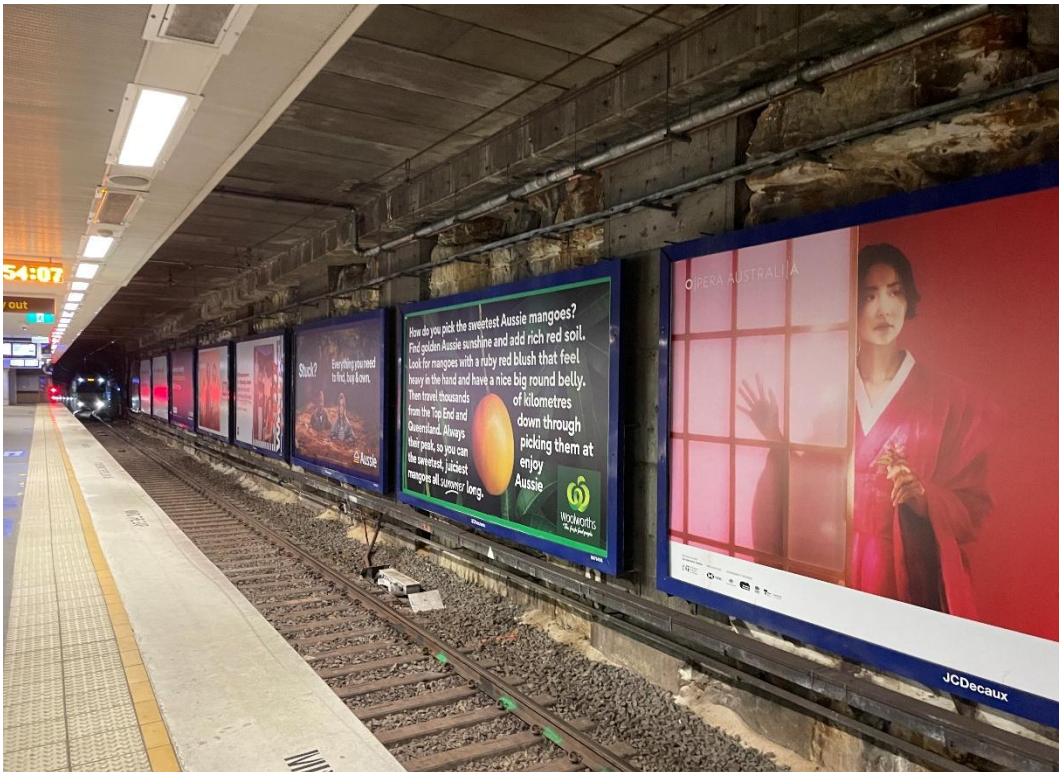


Photo 11: Photograph of the rock exposed along the northern boundary from Platform 1

Photo 12: Photograph of the northern tunnel portal at the western end of Platform 1

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	6
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

Photo 13: Photograph of the rock outcrop along the northern wall to the east of the site in the station area

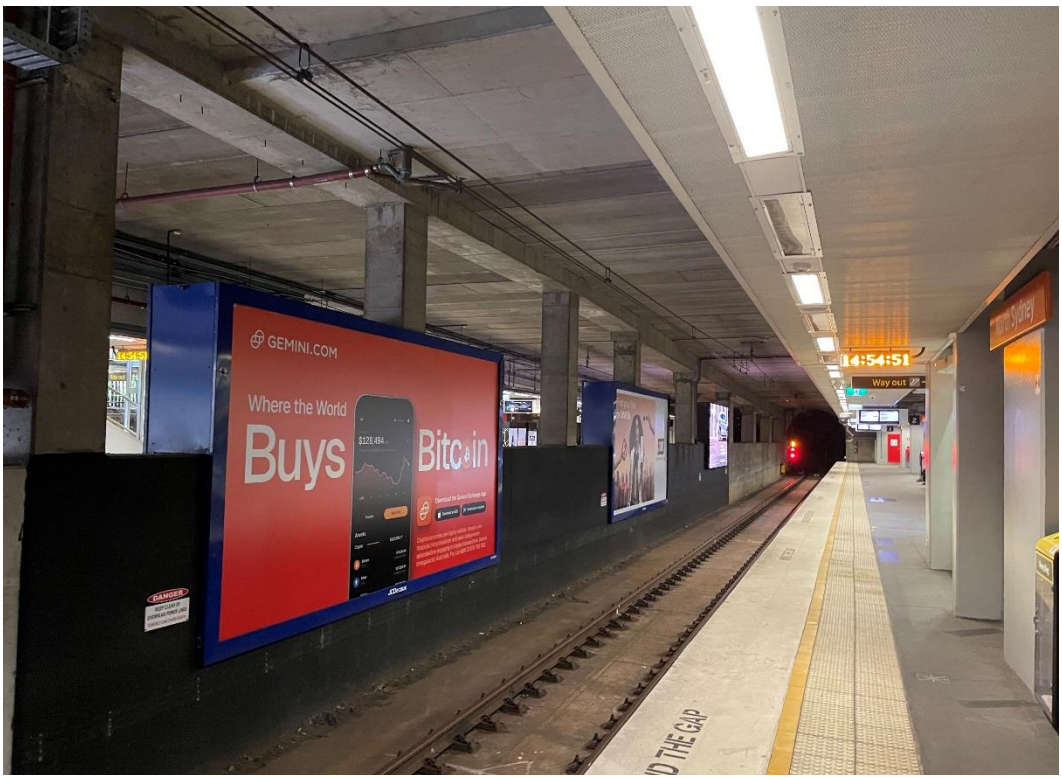


Photo 14: Photograph of the columns located between the platforms from Platform 2

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	7
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

Photo 15: Photograph of the rock along the southern boundary from Platform 4 at approximately the eastern end of the site looking west

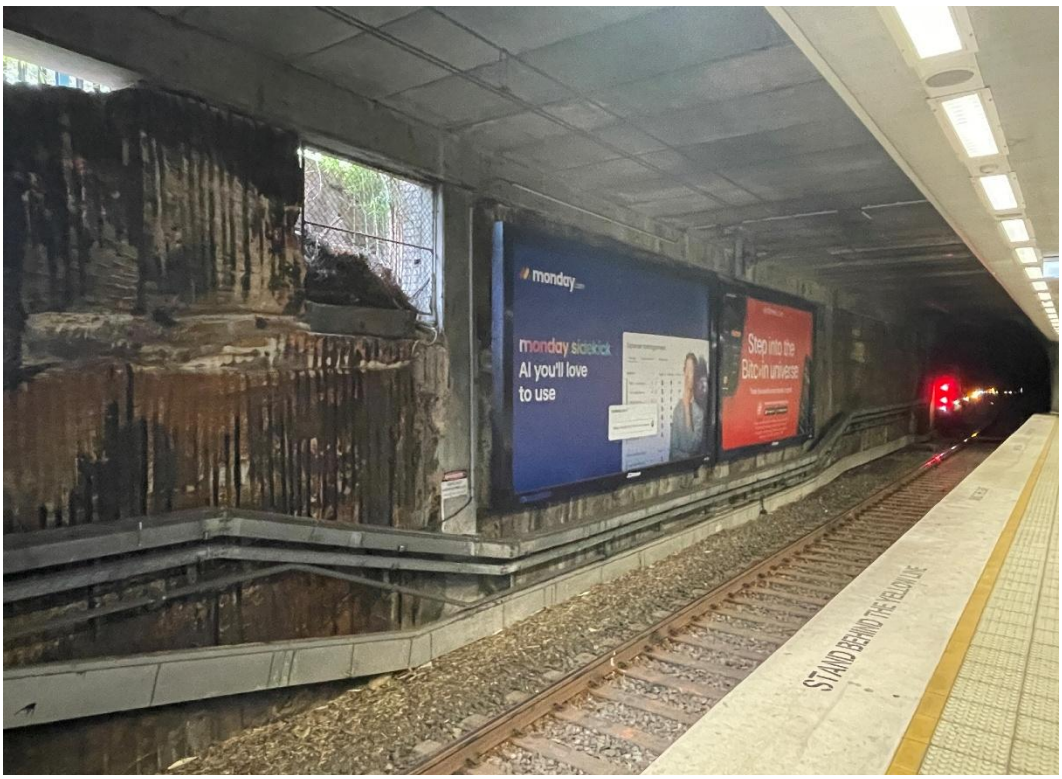


Photo 16: Photograph of the rock along the southern boundary from Platform 4 at approximately midway along the site boundary looking west

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	8
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

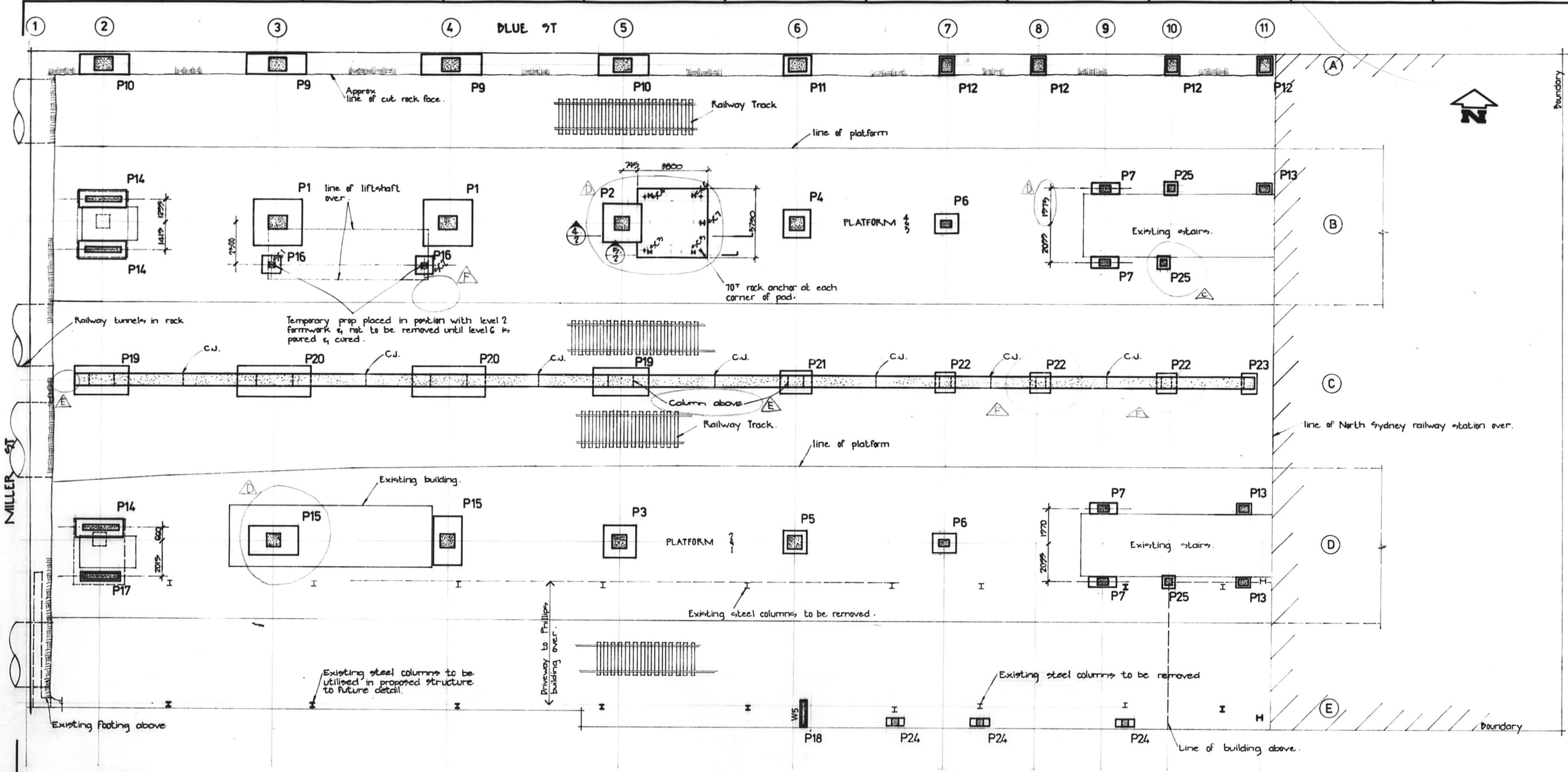
Photo 17: Photograph of the rock along the southern boundary and southern tunnel portal at the western end of Platform 4

Photo 18: Photograph of the rock along the southern boundary from Platform 4 including the existing tapered foundations of the structure above

Douglas PARTNERS	**Site Photographs**	PROJECT:	237286.01
	Proposed Mixed Use Development	PLATE No:	9
	5 Blue Street, North Sydney NSW	REV:	0
	CLIENT	Zurich Australian Insurance Properties Pty Ltd	

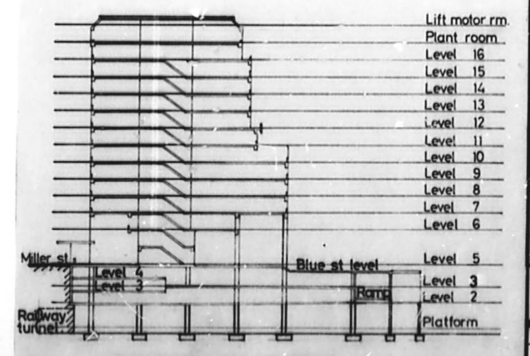
Appendix C

Existing Foundation Drawings



FOOTING PLAN

FOR GRID AND COLUMN SET-OUT DIMENSIONS REFER TO ARCHITECTS SET-OUT DRAWING.
NOTE: THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. 4155/2.



DIAGRAMMATIC SECTION
NO SCALE

CONSTRUCTION NOTES

GENERAL:

G1 This drawing shall be read in conjunction with all other working drawings and specifications and with such other written instructions as may be issued during the course of the construction. All discrepancies and variations shall be referred to the Engineer before proceeding with the work.
G2 All structural work shall be in accordance with requirements of all relevant and current S.A.A. Codes.
G3 All dimensions relevant to setting out and off-site work shall be verified before construction and fabrication is commenced.
G4 Dimensions shall not be obtained by scaling the structural drawings.
G5 During construction the structure shall be maintained in a stable condition and no part shall be overstressed.
G6 The structural elements shown on these drawings have been designed for superimposed loads as follows:

REFER RELEVANT PLANS.

CONCRETE:

C1 All workmanship and materials shall be in accordance with current editions of A.S.1480 except as varied by the contract documents.
C2 Concrete components and quality shall be as follows:

BUILDING ELEMENT	FOOTINGS.
F'c MPa	50
Density Kg/m³	DENSE
Max. agg. size mm	20
Slump mm	80
Cement type	A
Admixtures	NIL
Control testing type	PLANT

C3 Minimum clear cover to reinforcement, including tie-bars, unless otherwise shown in details shall be as follows*:

Structural Member	NOT EXPOSED to weather and/or water	EXPOSED
Footings, pad	—	75
strip	—	65
Columns	40	50
Walls	25	30
Beams	30	40
Slabs	20	30
To embedments	20	30

* High Fire-resistance Ratings or severe exposure conditions require greater covers; refer to AS 1480.
C4 Construction joints shall be properly formed and used only where shown or specifically approved by the Engineer.
C5 No holes, chases or embedment of pipes, other than those shown on the structural drawings, shall be made in concrete members without the prior approval of the Engineer.
C6 Splices in reinforcement shall be made only in the positions shown or as otherwise approved by the Engineer.
C7 Minimum lap for all fabrics shall be one mesh plus 30 mm.
C8 Reinforcement is shown diagrammatically; it is not necessarily shown in true projection.
C9 All reinforcement shall be to A.S.1302. Symbols: F: wire reinforcing fabric, R: structural grade round bar, S: structural grade deformed bar, C: cold worked deformed bar. Designation code of reinforcement bars as in example:
no. of bars in group — bar grade and type
17C20-350
nominal bar size in mm — spacing in mm
U.N.O. stands for "unless noted otherwise."
C10 Where TRANSVERSE TIE-BARS are not shown provide C-12-300, U.N.O. splice where necessary and lap with main bars: 400 mm.
C11 All concrete shall be placed and 'cured' in accordance with Section 7 A.S.1480. Where curing compound is used it must be applied (a) onto slabs within 2 hours of finishing operation (b) onto walls and columns immediately after removal of formwork.
C12 Horizontal formwork shall be stripped when approved by the Engineer.
C13 Slabs and beams shall bear only on the beams, walls, etc., shown on the drawings; all other building elements shall be kept 15 mm clear from soffits of structure.

FOUNDATIONS:

F1 Footings have been designed for a uniform bearing pressure of * kilopascals. Foundation material shall be approved for this pressure before placing concrete in footings.

* - REFER SCHEDULE.

MASONRY:

M1 Where slabs or beams bear on masonry the top course shall be level, smooth and covered with approved sliding joint material unless noted otherwise.
M2 No masonry walls to be erected on suspended slabs and beams until all propping has been removed.
M3 Bricks used in loadbearing construction shall have minimum compressive strength (as per A.S.A21) of 20 MPa unless noted otherwise.

ISSUE	AMENDMENT	BY	DATE
F	Amendment as shown	S.C.	25·2·83
E	Amendment as shown	B.S.	11·2·83
D	Amendment as shown	B.S.	24·1·83
C	Grid 1 columns removed. Amended as noted	B.S.	11·1·83
B	Construction joint on grid C as indicated	V.G.	14·12·82
A	Generally Amended	S.C.	17·11·82

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NEW SOUTH WALES — TEL. 929-7144

ARCHITECTS
HENRY POLLACK & ASSOC. P/L.

PROJECT
COMMERCIAL DEVELOPMENT
AT
BLUE ST, NORTH SYDNEY
FOR
MIRVAC PTY. LTD.

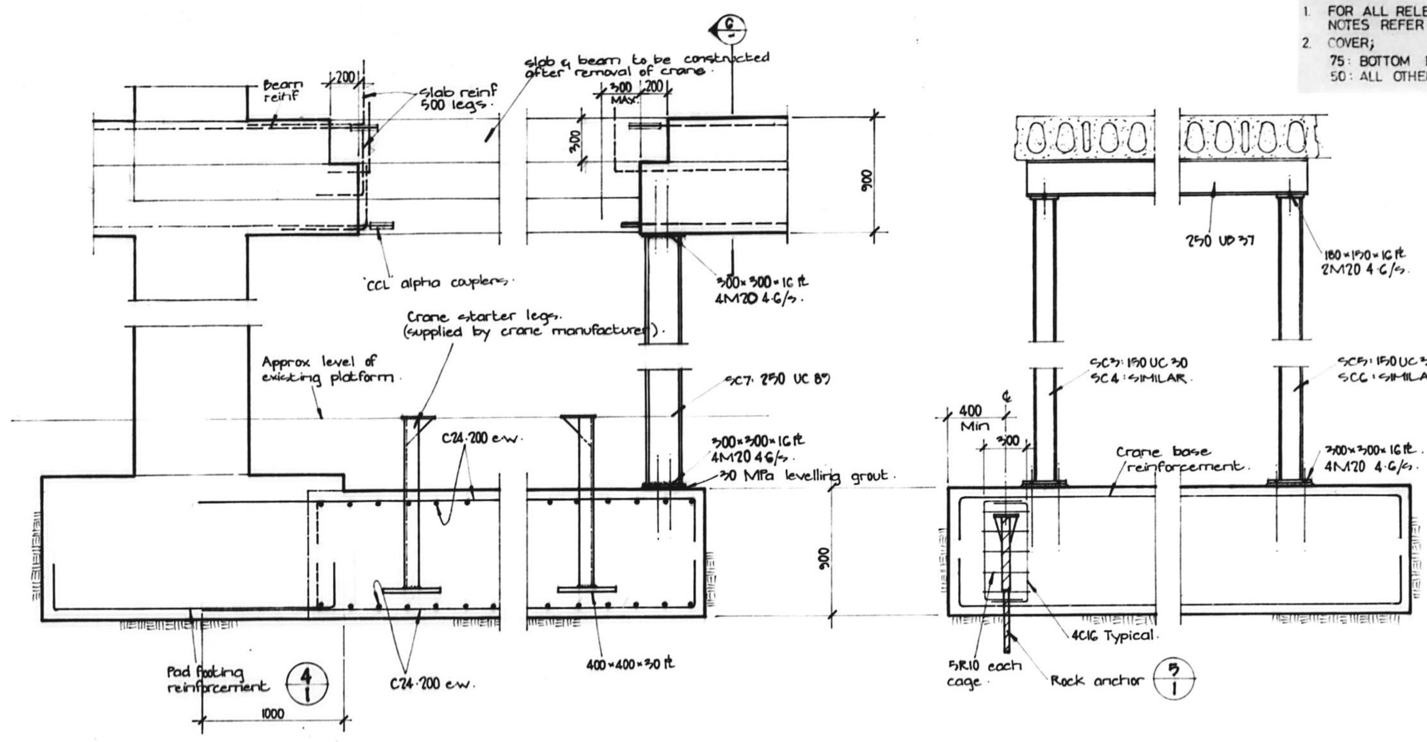
FOOTING PLAN

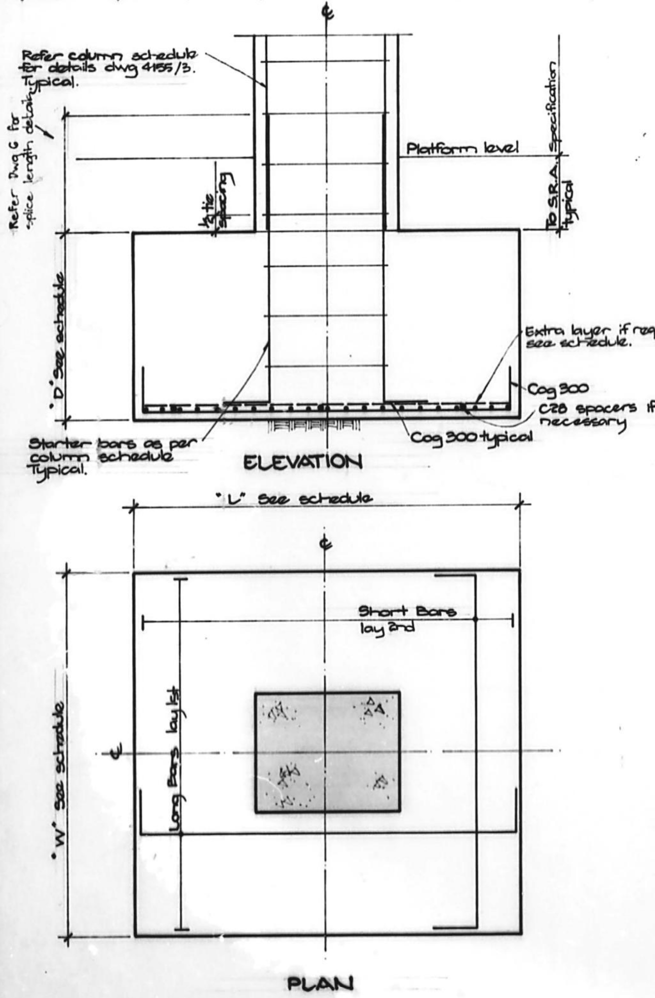
DESIGN: N.F.M	DRAWN: B.S.	SCALES: 1:100	
CHKD.: NFM	No. IN SET	mpn JOB	DRAWING No.
APPRD.: [illegible]		4155	1F
DATE: 5.10.82			

FOOTING TYPE.	3200 kPa BEARING PRESSURE					2100 kPa BEARING PRESSURE				
	L	W	D	LONG BARS	SHORT BARS	L	W	D	LONG BARS	SHORT BARS
P1	2700	2500	1300	19C28	21C28	3300	3100	1500	24C28	26C28
P2	2100	2100	1000	13C28	13C28	2600	2600	1200	16C28	16C28
P3	1800	1800	800	13C28	13C28	2200	2200	950	11C28	11C28
P4	1500	1500	600	10C20	10C20	1800	1800	700	9C24	9C24
P5	1300	1300	450	7C20	7C20	1600	1600	600	9C20	9C20
P6	1100	900	450	5C20	6C20	1300	1100	550	6C20	7C20
P7	1500	650	500	9C24	8C16	2100	700	750	5C28	11C16
P8	800	800	450	6C16	6C16	1200	800	600	6C20	8C16
P9	3300	1100	1150	2×9C28	13C20	5000	1100	1800	3×8C32	20C20
P10	2700	1100	900	2×7C28	11C20	4100	1100	1500	2×8C28	16C20
P11	1500	1100	400	7C20	6C16	2200	1100	600	9C24	9C16
P12	1000	800	300	4C16	4C16	1000	800	300	4C16	4C16
P13	800	600	300	4C16	4C16	1200	600	400	4C20	5C16
P14	2700	1000	600	7C20	18C20	3900	1000	1000	8C28	16C20
P15	2700	1600	1000	2×11C28	13C20	4100	1600	1600	2×12C28	16C20
P16	1000	1000	600	7C20	7C20	1250	1250	650	9C20	9C20
P17	2300	600	600	3C16	10C12	2400	600	400	3C16	10C12
P18	1400	300	300	3C12	5C12	1400	300	300	3C12	5C12
P19	3000	1500	500	6C16	25C24	4000	1700	600	8C16	28C24
P20	4000	1700	700	8C16	30C28	5000	2000	900	8C16	35C28
P21	1700	1200	400	5C16	13C20	2100	1300	500	6C16	12C24
P22	1100	1100	400	5C12	6C20	1300	1100	400	5C12	6C20
P23	1100	800	400	5C12	4C20	1100	800	400	5C12	5C20
P24	1100	400	400	3C24	5C12	1400	400	550	3C28	6C12
P25	700	700	400	4C16 T&B	4C16 T&B	700	700	600	4C16 T&B	4C16 T&B

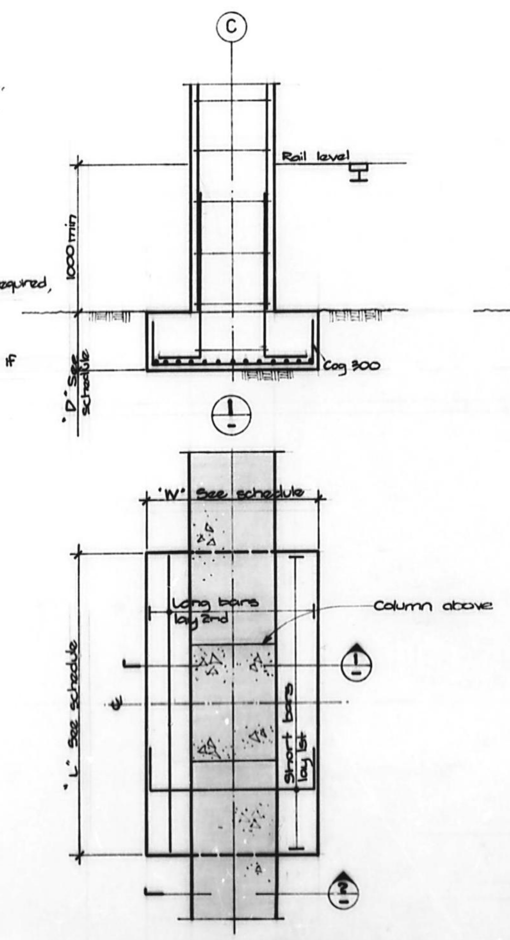
PAD FOOTING SCHEDULE F'c = 30 MPa

NOTES:
1. FOR ALL RELEVANT CONSTRUCTION NOTES REFER DWG. Nº 4155/1.
2. COVER;
   75: BOTTOM BARS
   50: ALL OTHER BARS

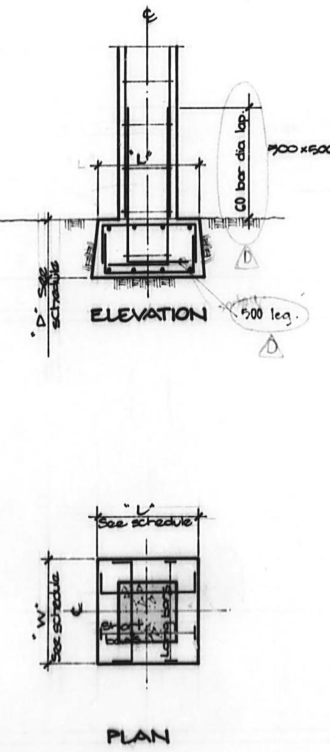




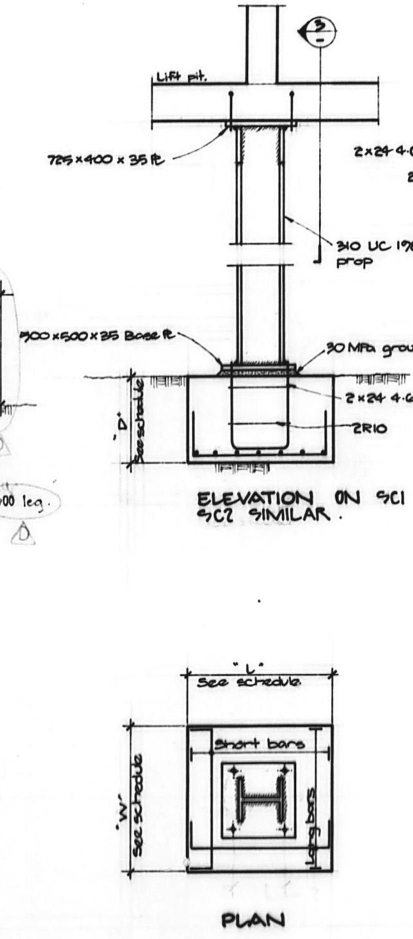
TYPICAL PAD FOOTING DETAIL TYPES P1 to P15, P17, P18 & P24



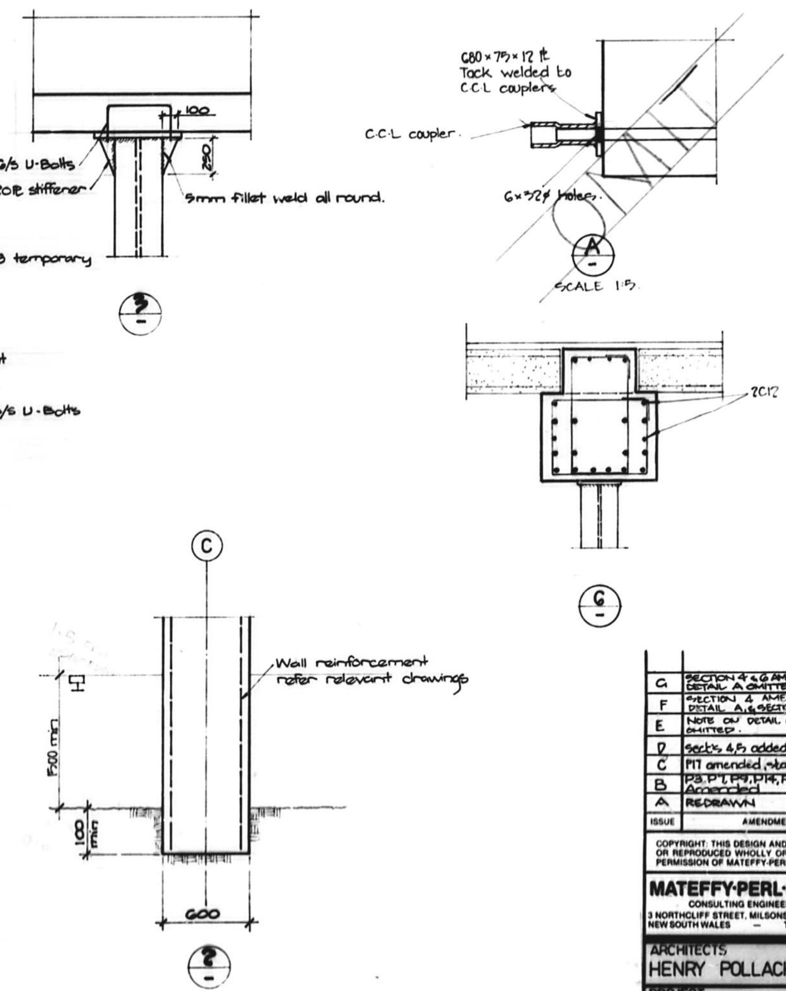
TYPICAL DETAIL FOR FOOTING TYPES P19 to P25



PAD TYPE P25



PAD TYPE P16



ISSUE	AMENDMENT	BY	DATE
G	SECTION 4 & 6 AMENDED DETAIL A OMITTED	S.C.	14.4.83
F	SECTION 4 AMENDED & DETAIL A, & SECTION 6 ADDED.	B.S.	7.3.83
E	NOTE ON DETAIL OF P19-P23 OMITTED.	B.S.	11.2.83
D	Sect's 4,5 added, amended A	B.S.	24.1.83
C	P17 amended, starter note added	B.S.	11.1.83
B	P3, P7, P9, P14, P20, Amended	V.G.	14.12.82
A	REDRAWN	S.C.	17.11.82

MATEFFY·PERL·NAGY PTY. LTD.
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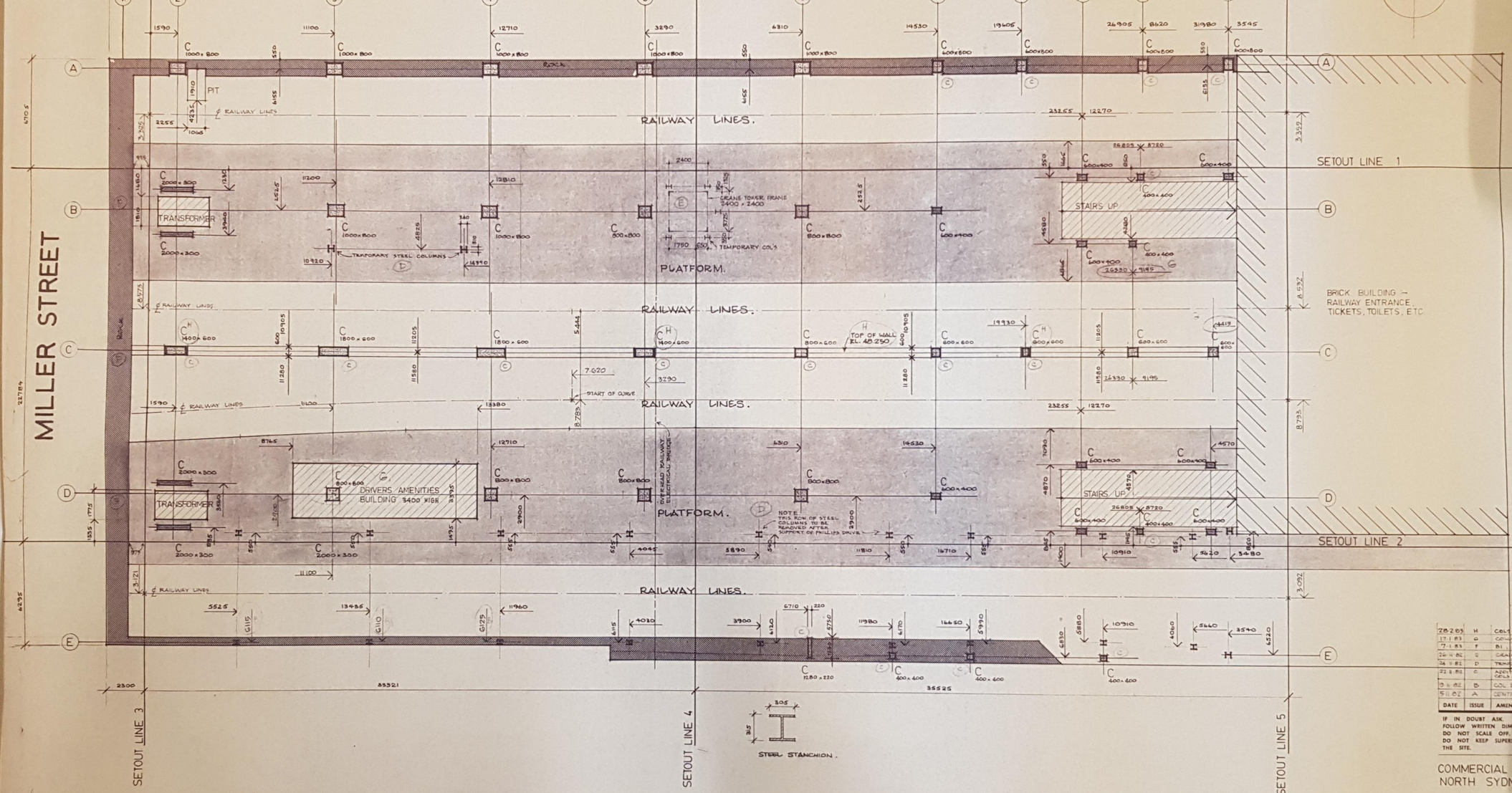
FOR
MIRVAC PTY. LTD.

FOOTING DETAILS

DESIGN: N.F.M.	DRAWN: S.C.	SCALES 1:20	
CHKD.: NFM	No. IN SET	mpn JOB	DRAWING No.
APPRD.:		4155	2G
DATE: 18.11.82			



DATE	ISSUE	AMENDMENT
28-2-83	H	COL'S + WALL ON ℄ C
17-1-83	G	COLUMN POSITIONS
7-1-83	F	B1, C1, D1 DELETED
20-9-82	E	CRANE BASE RELOCATED
24-9-82	D	TEMPORARY STEEL COLS + CRANE BASE
22-8-82	C	ADDITIONAL COLS + CAREER GRID COLS + WALL ON ℄ C
5-8-82	B	COL'S ON LINE B REVISED
5-8-82	A	CENTRE LINE RAILWAY LINES

IF IN DOUBT ASK.
FOLLOW WRITTEN DIMENSIONS ONLY.
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COMMERCIAL DEVELOPMENT
NORTH SYDNEY STATION

CONCRETE SETOUT PLAN

HENRY POLLACK
& ASSOCIATES PTY.LTD.
ARCHITECTS
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TEL. (02) 358 5366

DRAWING TITLE
FLOOR PLAN AT PLATFORM LEVEL1

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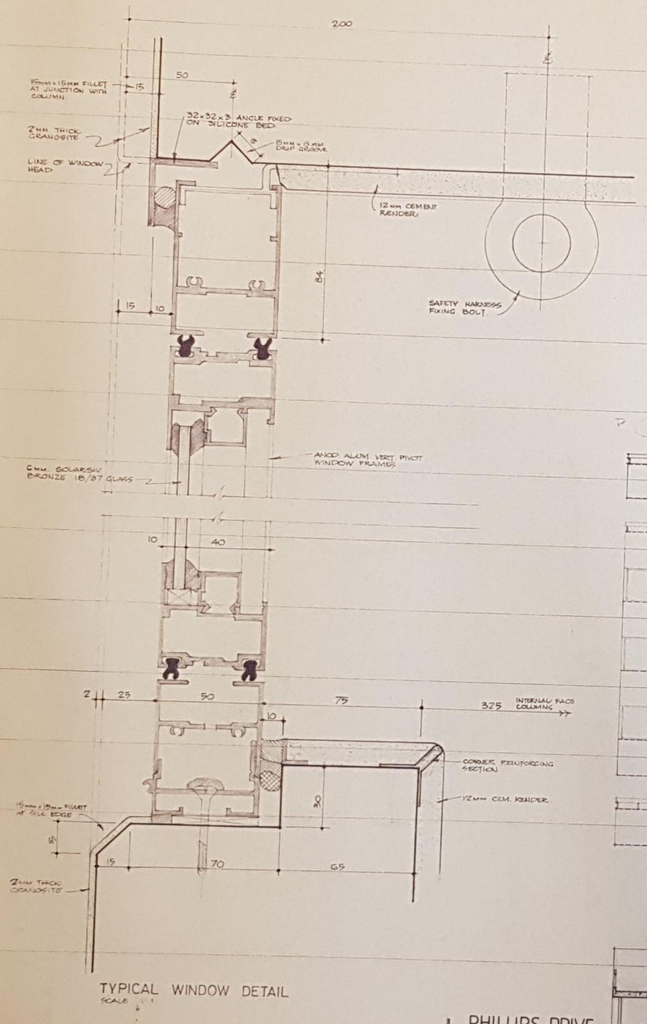
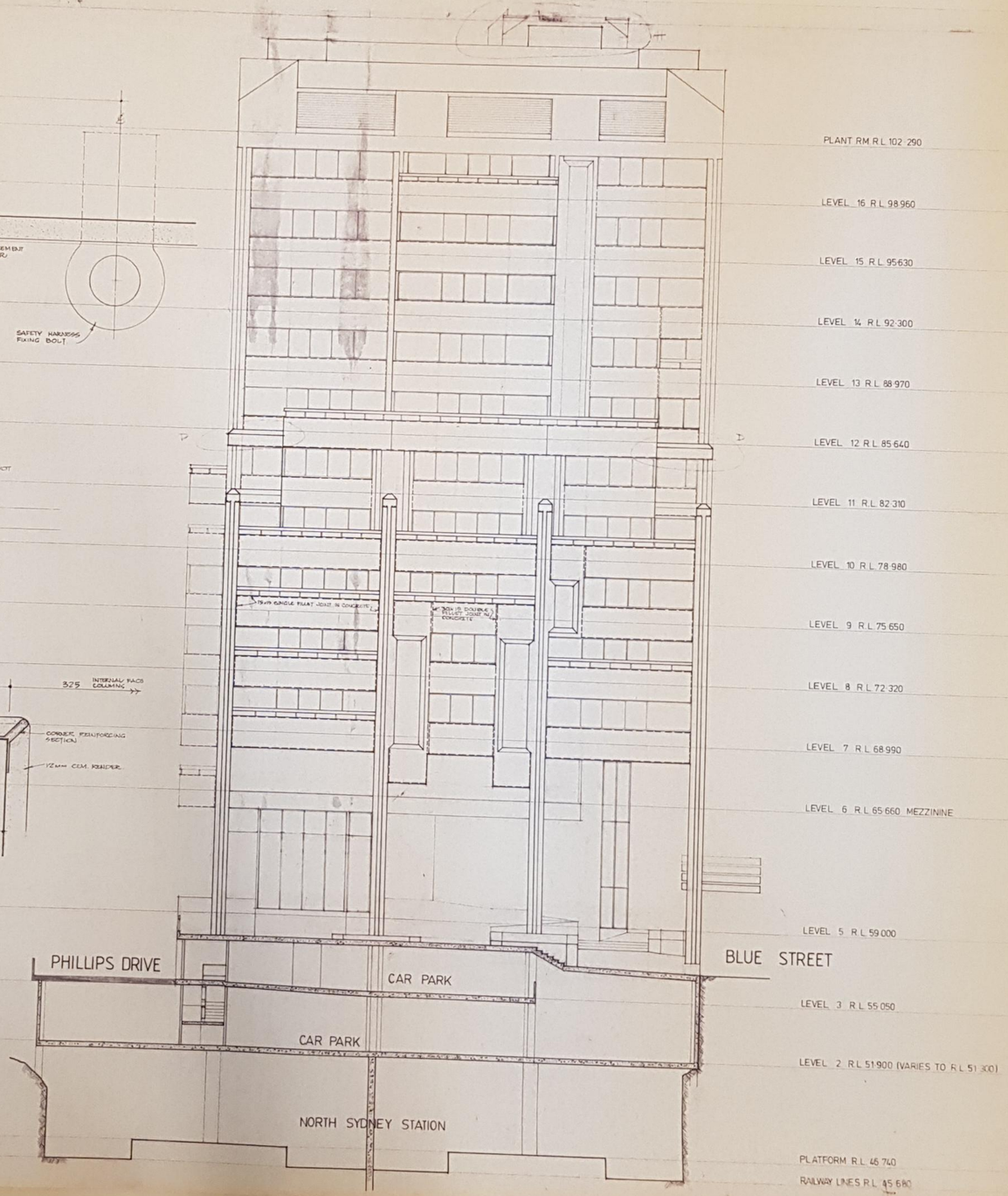
JOB No. 80/18	DRAWING No. 18		ISSUE



TYPICAL WINDOW DETAIL
SCALE 1:1



DATE	ISSUE	AMENDMENT
13.10.82	N	LIFT MOTOR ROOM ROOF
19.7.82	G	GRANOLITE THICKNESS
15.7.82	F	BALCONY BALUSTRADE DETAIL AND WINDOW HEAD DETAIL
30.6.82	E	FILLET JOINTS IN LINE
23.6.82	D	WINDOW DETAIL ADDED
22.6.82	C	DRAWN AT 1:50 BALCONY
8.6.82	B	MASONRY REVISED
3.3.82	A	ROOF PARAPET WALL ADDED
		LEVEL 11 REVISED

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DRAWING TITLE

EAST ELEVATION AND SECTION B-B

SCALE	DATE	DRAWN	CHKED
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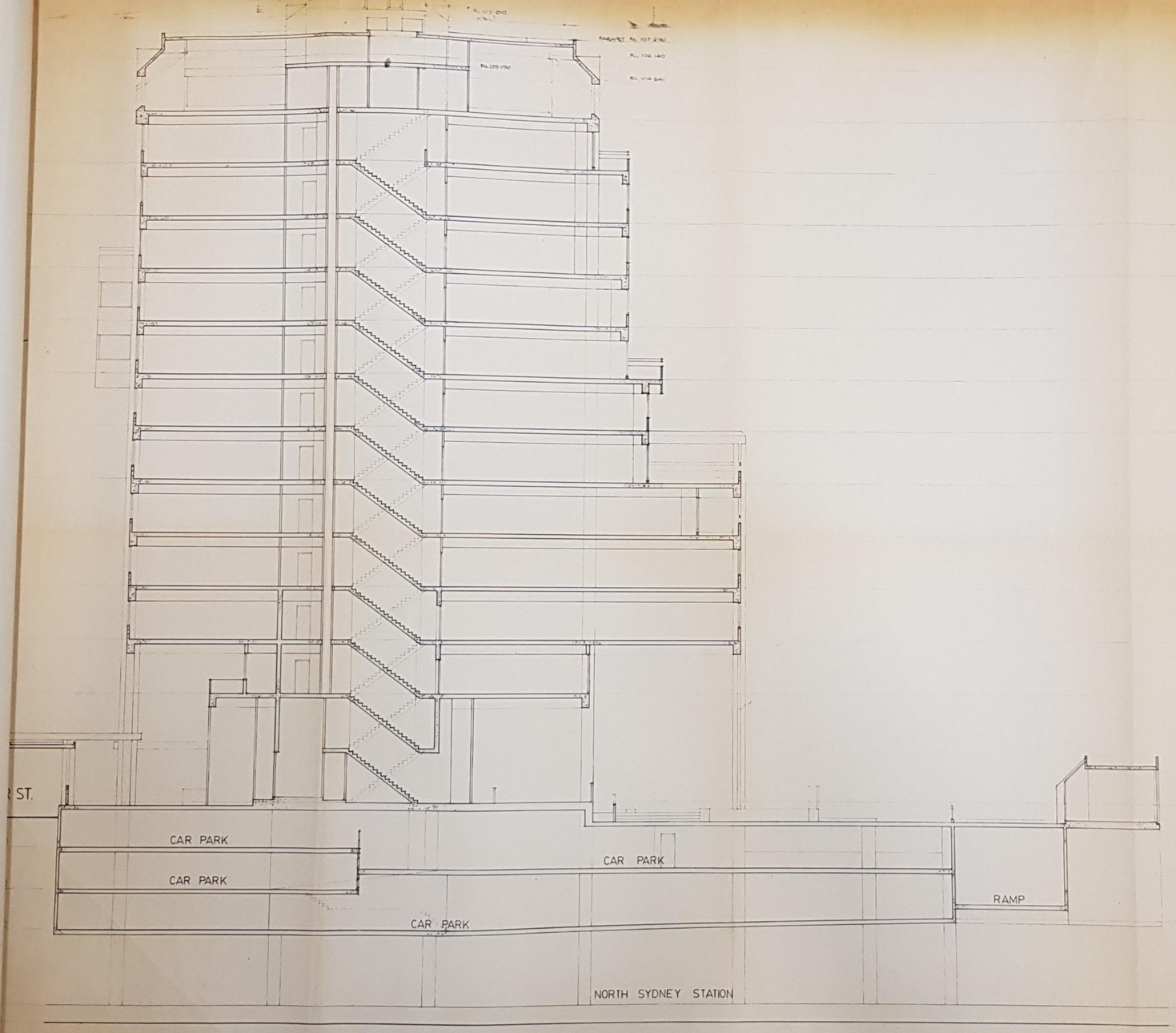
JOB No.	DRAWING No.	ISSUE	
80/18	16	G	



TOP R.L. 105.240

LIFT MOTOR RM. R.L. 105.910

PLANT RM. R.L. 102.290

LEVEL 16 R.L. 98.960

LEVEL 15 R.L. 95.630

LEVEL 14 R.L. 92.300

LEVEL 13 R.L. 88.970

LEVEL 12 R.L. 85.640

LEVEL 11 R.L. 82.310

LEVEL 10 R.L. 78.980

LEVEL 9 R.L. 75.650

LEVEL 8 R.L. 72.320

LEVEL 7 R.L. 68.990

LEVEL 6 R.L. 65.660 MEZZININE

LEVEL 5 R.L. 59.000

LEVEL 3 R.L. 55.050

LEVEL 2 R.L. 51.900

PLATFORM R.L. 46.740

RAILWAY LINES R.L. 45.680

DATE	ISSUE	AMENDMENT
23.10.81	E	LIFT MOTOR ROOM RAISED
27.4.81	D	PARAPETS & PLANT ROOM ADDED
6.3.81	C	ROOF DRAWN, WALL ADDED
16.1.81	B	LIFT MOTOR ROOM FLOOR ADDED
7.1.81	A	LIFTS & TOP RM ADDED

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COMMERCIAL DEVELOPMENT
NORTH SYDNEY STATION

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DRAWING TITLE
SECTION A-A

SCALE	DATE	DRAWN	CHKED
1:100	JULY '80		

JOB No. 80/18 DRAWING No. 13