Structural
Civil
Traffic
Façade

Wilkinson House

Structural Options

Retention of Existing Facade

Prepared for SCEGGS Darlinghurst / 26 SEPTEMBER 2019 / Rev D

181375 - SAAB

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1.0 Executive Summary

Taylor Thomson Whitting (TTW) have reviewed the structural implications of retaining the existing façade and balcony structures of Wilkinson House at SCEGGS Darlinghurst. This is known as 'Option B' in the Wilkinson House Options Analysis prepared by TKDA Architects, dated November 2018. The recommendations of this report advise that the façade should not be retained due to:

- Constructability of the temporary façade retention system
- High-risk structural requirements of removing the lateral support to an entire building façade, and supporting this temporarily from within a relatively small and congested site
- Safety concerns to the public and workers during construction where the façade stability may be unknown in numerous locations and/or left unstable, particularly during vibration work
- The inability to certify this façade before construction due to very little existing information being available

This report considers and discusses the relevant structural and geotechnical aspects of the proposed retention of the existing façade of Wilkinson House (Option B). It is supported by technical documents provided by construction professionals, geotechnical engineers, and traffic engineers.

As illustrated in the construction management section of this report (refer to Appendix D), the clearances between the existing floors and the façade of the existing building are insufficient for the installation of temporary steelwork required to restrain the façade during construction. Significant demolition of the load bearing structure is required at Ground floor and Basement levels to enable the construction of the foundations needed for the temporary façade retention steelwork. This significant demolition is not possible without this temporary steelwork as it will result in risking the lateral stability of the building, resulting in a high probability of collapse.

The existing structure is largely unknown, with remediation work required to ensure the structures capacity within the current Australian Standards. This unknown structure places risk on the workers and public during construction where the building may be found to be different to expected and left unstable. The working site area is very small due to these temporary structures, placing further risk on the safety of the workers and on the public.

Due to the concerns outlined in this report, TTW recommends that the proposal presented in Option B to retain the façade during construction can not be achieved in a safe and viable manner.

2.0 Wilkinson House Rebuild and Demolition Options

Taylor Thomson Whitting (TTW) have reviewed the structural implications of retaining the existing façade and balcony structures of Wilkinson House at SCEGGS Darlinghurst. This is known as 'Option B' in the Wilkinson House Options Analysis prepared by TKDA Architects, dated November 2018.

Retaining the existing façade will require the installation of significant additional temporary structures that must be coordinated with viable construction sequencing and the proposed development. The types of temporary structures recommended in this report have been investigated and researched to a sufficient extent to enable preliminary structural stabilisation design to be undertaken, and based on the existing building information, the proposed development and TTW's previous experience with similar types of construction methodologies.

Further geotechnical information is recommended to be undertaken to develop and compare the design for the retention systems. This report incorporates the assumption that a retention system is required to excavate the proposed ground floor, and to minimise the risk of collapse of the existing façade by avoiding underpinning to the existing and potentially unstable foundations.

2.1 Supporting Documentation

This report is supported by:

- Review of Constructability of the Façade Retention System for Wilkinson house at SCEGGS, prepared by Innovative Construction Methodologies, Management & Planning (ICMP Group)
- Wilkinson house Options Analysis, drawings prepared by Tanner Kibble Denton Architects
- Turning circle analysis drawings, prepared by Taylor Thomson Whitting
- Wilkinson House Temporary Façade Retention Drawings, prepared by Taylor Thomson Whitting
- Construction and Operational Noise Report, prepared by Wilkinson Murray, report number 18180

3.0 Existing Structure

There is currently very little information on the existing structure at Wilkinson House. The design team have undertaken site inspections in order to understand the existing structure. It consists of timber floor joists and floorboards, supported by masonry walls. The masonry walls are providing the overall lateral stability to the building.

The existing façade is a double brick masonry structure that is supporting the internal floor joists. The existing façade is provided with lateral restraint with the floor joists pocketed into the brick wall. This era of construction would have typically relied on friction between the timber joist and the masonry wall to prevent the wall from buckling outwards, rather than any mechanical fixings. Any removal of the internal bracing walls removes lateral restraint to the façade, and increases the potential for the façade to move independently of the existing timber floors.

The figures below illustrate a typical floor junction into existing masonry walls

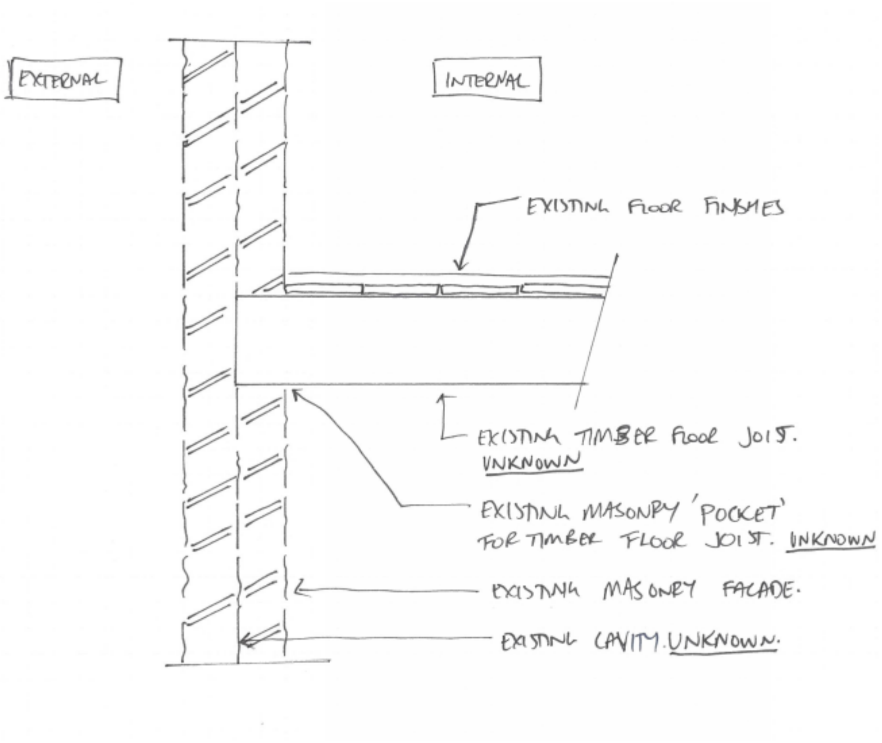

Figure 1: Typical Masonry Façade Junction

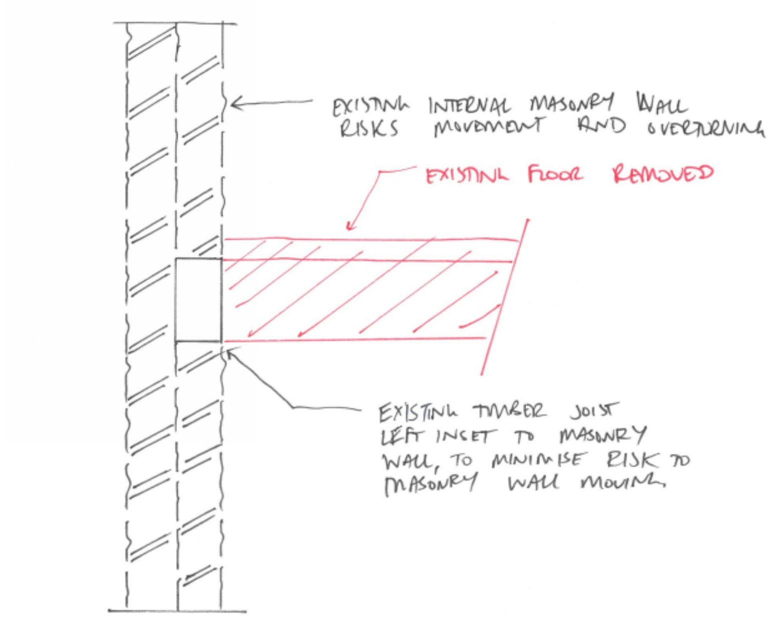

Figure 2: Typical Masonry Façade Junction, During Demolition

Without documentation of this structure, we are unable to determine the level of connection between the floor and the external walls. The timber joists could be subject to rot and decay from water egress through the masonry façade, placing more risk on the longevity and stability of this masonry façade.

3.1 Certification of the Existing Structure

Where existing structure is proposed to be retained, investigations and remediation work must be capable of being completed if the existing structure is to be capable of certification to current codes complying with the National Construction Code. These site investigations will involve the removal of ceilings and floor finishes, to view the condition and connections of the floor joists and walls.

Te existing masonry façade will need significant structural rectification work to meet the requirement for earthquake resistance specified under AS3600. The requirements to the current code are more onerous than when this structure was originally built. Non-compliances include, but is not limited too:

- Mechanical connections of the timber floor joists to the masonry façade, such as screws or steel angles
- Replacement of all internal floor structures which are damaged or can not be certified as compliant
- Adequate connection of the proposed concrete floor slab to the existing masonry façade
- Adequate fire rating of all permanent connections

4.0 Proposed Development

Option B in the Wilkinson House Options Analysis proposes to retain the existing North and East facades, facing Forbes Street and St Peters Street; and to remove the internal load bearing walls and demolish the floor structures.

The proposed re-build will provide with more flexible learning spaces, in line with the clients needs and must provide a new structural system which can be constructed using known techniques, in a safe manner, and that is capable of satisfying all applicable codes and standards.

4.1 Retaining the Façade: Considerations

In order to retain the façade and balcony structures to Wilkinson House, the following critical aspects need to be considered and fully resolved;

- Adequate personal safety for the public outside the construction site, construction workers undertaking the works and the school community utilising adjacent buildings.

- The façade itself needs to be protected in the temporary and permanent case.
  Significant temporary structural steel, including clamping structures and additional foundations are required to be installed to brace the façade during demolition of the internal floor structures.
  The final concrete floor structure will permanently restrain the existing façade at a similar floor levels.

- The existing windows must be removed to clamp the existing façade to temporary steel support members.

- The existing floor to floor dimension would be limited to existing non compliant ceiling heights in order to tie into the existing façade lateral restraint locations

- The educational requirement for larger column free spaces results in a deeper floor and beam structure, compared to the existing. In order to align the new structure with the existing floor levels, the proposed structure depth and a zone for services above the ceiling will result in a ceiling height that is lower than the minimum NCC required 2400mm, and well below the minimum NSW Department of Education required 2700mm ceiling height

4.2 Demolition Methodology

A demolition methodology which protects the existing façade must adequately address and remove the following risks;

- No or minimal underpinning to the façade. Underpinning poses large risks to the existing structure, and is ideally avoided by reducing basement extents
- Construction access through the existing façade for demolition and excavation equipment, workers and materials. Adequate access to the site and the workzones for structural stabilisation will require an agreed path of travel into the site from the street, and will introduce some demolition to the facade to facilitate construction access.
- Hoarding to the footpath to protect the public during construction and to protect the existing façade
- Excavation using equipment that will avoid large vibrations, in order to minimize any damage to the existing façade, for example limiting the use of jackhammers. The site is likely to be underlaid with rock, and the excavation will involve some heavy machinery to construct the basement.

Refer to Appendix D for more detailed analysis for demolition methodology and related risks and considerations.

5.0 Existing Street Conditions

Wilkinson House is located on the corner of St Peters Street and Forbes Street in Darlinghurst. The SCEGGS Darlinghurst Campus is located in the site adjacent to Forbes Street and St Peters Street. The site plan below indicates the key elements surrounding Wilkinson House.

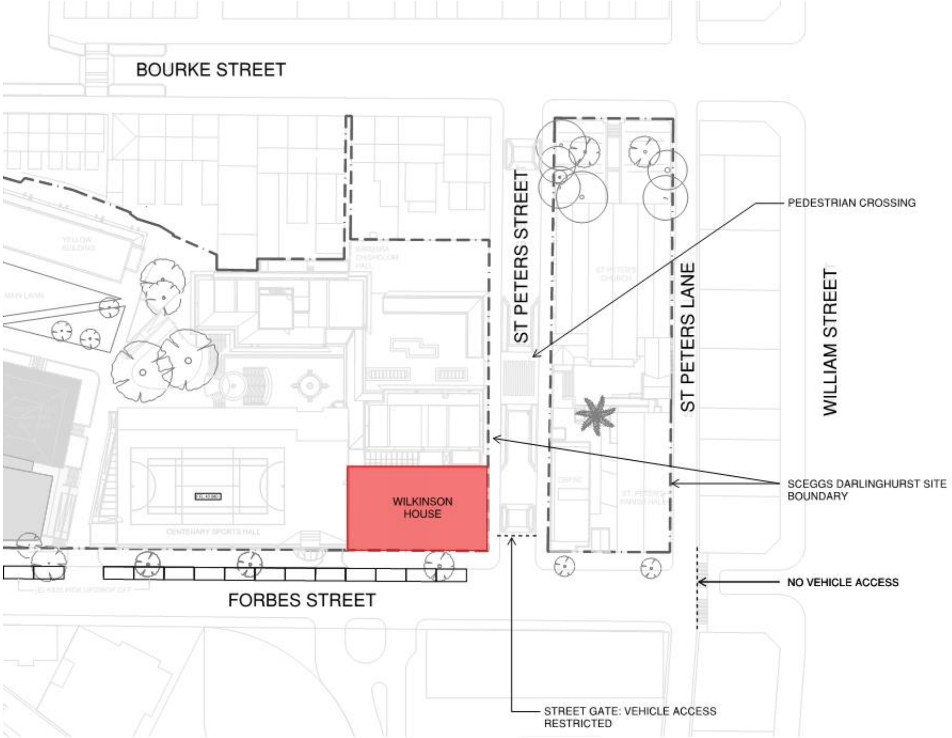

Figure 3: Wilkinson House Site Plan (Image courtesy of TKDA Architects)

During school hours, St Peters Street is closed to local traffic using a street gate. This gate opens at peak drop off and pick up times for the school. During the gates opening hours, St Peters Street also acts as a thoroughfare for public and school buses to cross over to Bourke Street. There is high pedestrian traffic in St Peters Street during the gate opening hours with school children crossing over to and from William Street, Forbes Street and Bourke Street on their commutes.

5.1 Street Construction Management

During the construction of Wilkinson House, SCEGGS Darlinghurst has highlighted the operational need to retain pedestrian access on Forbes St and St Peters St, to maintain existing pick-up and drop-off activities, and to retain the bus turning circle as it currently operates.

A temporary construction zone will need to be established on the street. This is required at 22 metres long, following the advice of the ICMP group in order to provide adequate dimension for delivery vehicles, cranage and concrete pouring. The construction zone is proposed on St Peters Street to avoid;

- Significant disruption to the school operations and bus parking on Forbes St
- Disruption to the local residents on the Eastern side of Forbes St using the local parking and accessing their properties.

This zone would be confirmed by the appointed contractor following design development, and a formal Traffic Control Plan being completed and approved.

For this construction zone to be established, St Peters Street will require re-levelling to facilitate truck access. The existing street islands must be removed. It is likely the vegetation on St Peters Street will be impacted by this construction zone.

Please refer to **Appendix A** for the sweep path movements of the existing bus turning circles adjacent to a proposed construction works zone on St Peters Street. The existing bus movement is difficult on Forbes St, however the introduction of the construction works zone on St Peters Street will not impede this existing bus movement.

In order to achieve this bus movement, the construction zone is required to be adjacent to the existing pedestrian crossing. The crossing is likely to require manned traffic control to ensure pedestrian safety. The existing street garden islands, East of the crossing on St Peters Street will need to be removed.

All existing pedestrian footpaths have been maintained. The footpath to the South of St Peters Street may need further hoarding during construction.

A construction management plan must be developed and approved to ensure these clearances can be achieved with the specific construction equipment proposed by the contractor following design development.

5.2 Truck Movements Around Site

If retention of the two existing street facades to Wilkinson House is considered, there are minimal existing openings that will facilitate truck access into the site. The offset of the existing façade on Forbes Street to the adjacent Sports hall is 2670mm clear, refer to Figure below. This dimension will not provide adequate truck access into the site. A larger portion of the façade must be demolished in order to gain adequate construction access into the site.

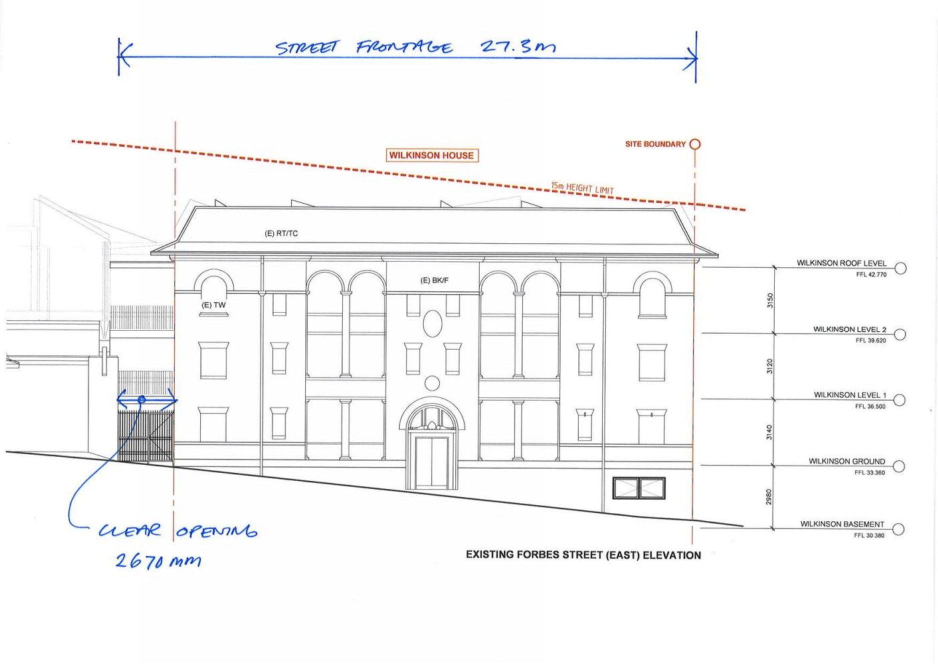

Figure 4: Wilkinson House, Forbes Street Elevation, Clear Opening Offset from Adjacent Sports Hall

In order to comply with typical Council conditions of loading and unloading truck inside the site boundaries, significant demolition of the existing façade is required.

Please refer to **Appendix B** for the possible demolition options to the façade to gain access into the site. The openings indicated on the St Peters Street façade can be created without significant structural risk to the overall building. However these will not provide sufficient space for a Small Rigid Vehicle (SRV) to access into site. The clear opening required for an SRV truck to reverse into the site is 3 metres wide, as per the drawings in Appendix B.

To avoid this on site truck movements, the relevant authority will need to approve the removal of spoil and excavation material by loading trucks parked on the street using conveyor belts or similar. This will require the closing of the pedestrian footpath South of St Peters Street at a minimum to mitigate pedestrian risk.

6.0 Option B-1: Temporary Façade Retention Within Site Boundary

Typically, temporary façade retention works are required to be located entirely inside the site boundary. This is generally a standard Council requirement. The temporary structural façade retention scheme will need to be coordinated with a viable construction management plan which resolves all of the concerns outlined in Section 3.0.

Refer to the constraints identified in **Appendix C** for drawings indicating a possible temporary façade retention scheme. This involves a series of vertical trusses installed between each floor to laterally restrain the existing façade, prior to demolition of the building.

To complete this work, the following is required:

- Local demolition and excavation work to prepare the foundations of this temporary structure
- Installation of piles to support the temporary façade retention, and allow for the proposed excavation of the extended ground floor
- Mobile crane to be setup on the street frontages (including likely road closures in both Forbes Street and St Peters Street) to lift and place structural steel frames and/or temporary propping to support the facade

- Installation of a tower crane (using the mobile crane from point above) in an acceptable location to facilitate all required materials handling
- Undertake major demolition of the existing structure, with construction activity to be coordinated around protection of the existing façade and protection of the temporary façade retention scheme

Please refer to **Appendix D** for the report prepared by the ICMP Group outlining the constructability restrictions of this façade retention system. This report concludes that construction of the temporary façade retention system will involve significant demolition of the existing building, and therefore introducing significant risks to the stability of the existing façade.

The piling involved with the temporary façade retention system and the shoring to achieve the ground floor excavation can not be completed without:

- Significant demolition to the existing façade to gain access for this machinery
- And/or significant demolition to the internal load bearing structure, likely to affect the overall stability of the existing facade

This report also identifies the vibration concerns associated with using this type of machinery adjacent to the existing façade. The equipment will need to meet the low vibration criteria setout in the Construction and Operational Noise Report, prepared by Wilkinson Murray. The equipment to form the shoring wall and piling for the façade retention will need to be trialed onsite to comply with these vibration limits to ensure no damage is created to the existing façade structure. Should the trails fail then this scenario poses an unacceptable level of structural risk to the existing façade due to the potential damage caused by vibration.

The report concludes that the installation of piles for temporary façade support columns is fundamental to the façade retention option. The report further identifies that the drilling of these piles is not possible without causing damage to the façade and imposing undue safety risks to the construction personnel and members of the public.

7.0 Option B-2: Temporary Façade Retention Outside of Site Boundary

An alternative scenario has been investigated whereby the temporary façade retention structures can be founded outside of the site boundary in the footpath and roadway. TTW does not recommend this option due to the following concerns:

- A Dial Before You Dig search has indicated significant existing inground services under Forbes St and St Peters Street, particularly Sydney Water assets. These require clearance zones and approvals to build over to protect and maintain these assets.
  Foundations for the temporary structures will need to be found within the pedestrian footpath and roadway, which will introduce significant risk to these existing inground services
- Street access is already tight and with the temporary structures outside the boundary, it is likely no bus route can be realised through Forbes St and St Peters Street. These roadways will be closed to vehicles, with construction access and local access only for the duration of the project.
- This will introduce significant pedestrian risks, and it is recommended the general public be restricted to the Northern side of St Peters Street.
- The challenges with construction movements in and out of the site, as outlined in Section 3.2 will still need to be realised.

Please refer to **Appendix E** for an overlay of the existing services on the streets adjacent to Wilkinson House.

8.0 Conclusions

In order to retain the existing façade and re-construct the building behind these street frontages, the following minimum requirements will need to be approved relating to general construction management:

- Partial demolition of the façade to gain access into the site for construction equipment (Refer section 3.2)
- Road closures of both Forbes Street and St Peters Street for the duration of the project, or for significant periods of the project.
- Excavation and piling equipment to be trailed onsite to understand vibration limits before the elimination of and potential damage to the existing structure can be guaranteed

Even if the items above are approved, the process of demolishing and retaining the existing façade will impose an unacceptable level of construction safety risks for the duration of the project. The required piling, excavation and retaining procedures are likely to cause damage to the existing façade.

Alternatively, where the proposed development is allowed to remove the existing façade, and to demolish and re-build the building (i.e. Option C in the Wilkinson House Analysis Study) the construction methodology will enable activity to be retained inside the site boundary. This method will provide a safer general public works zone adjacent to the site and will not impose significant risk to pedestrians nearby, whether they be related to school or general public activities.

Prepared by

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BARRY YOUNG
Director

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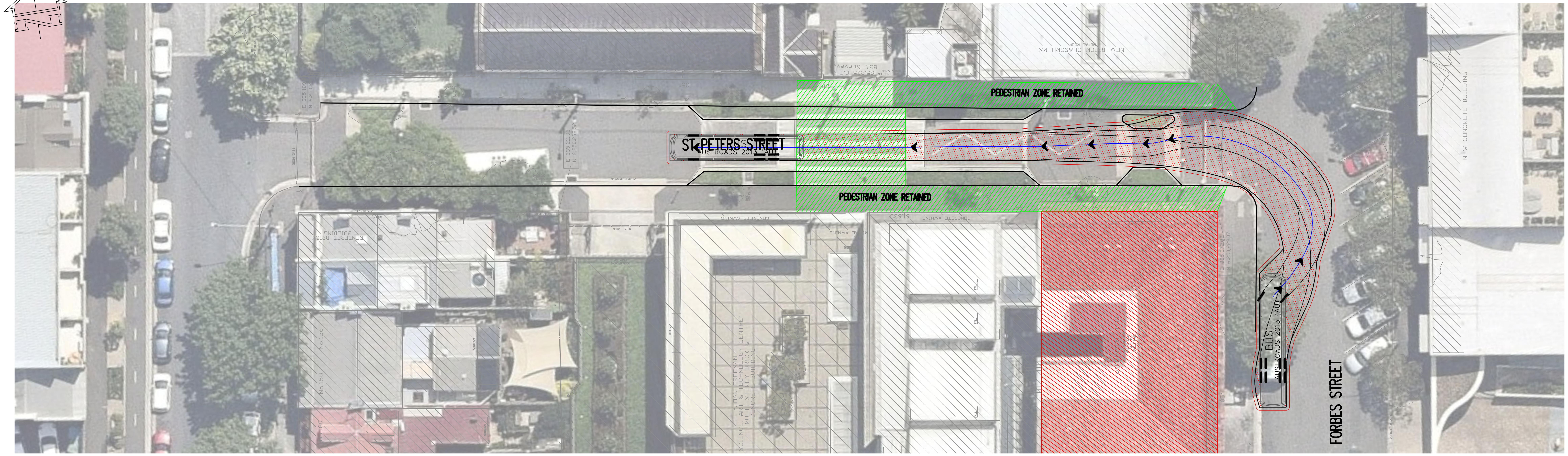
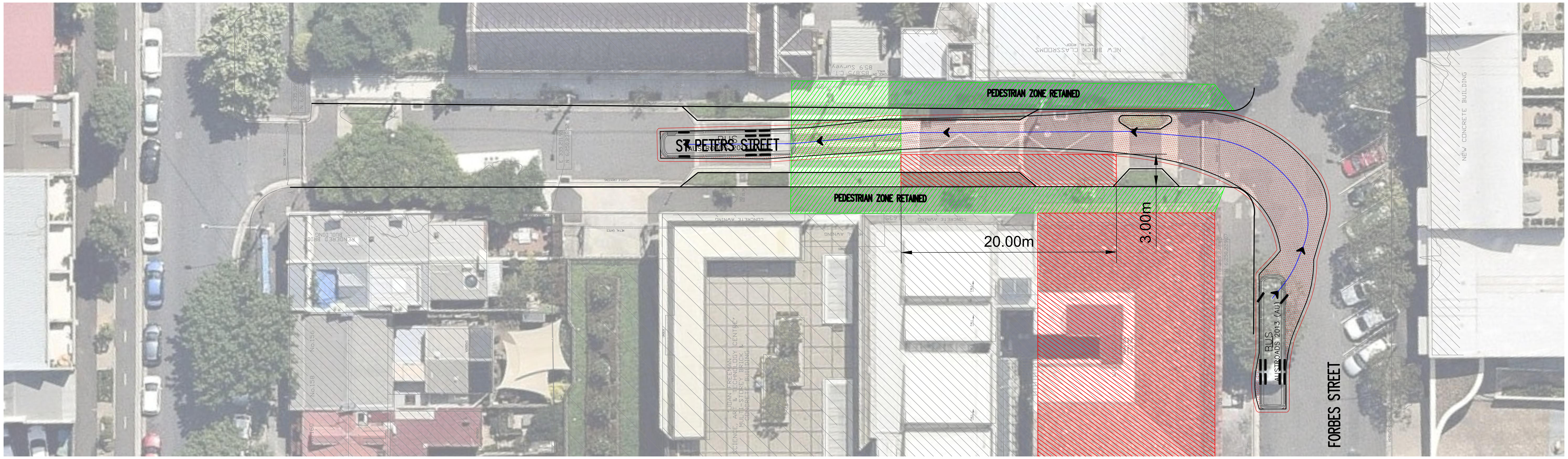
Appendix A

Street Construction Works Zone Drawings

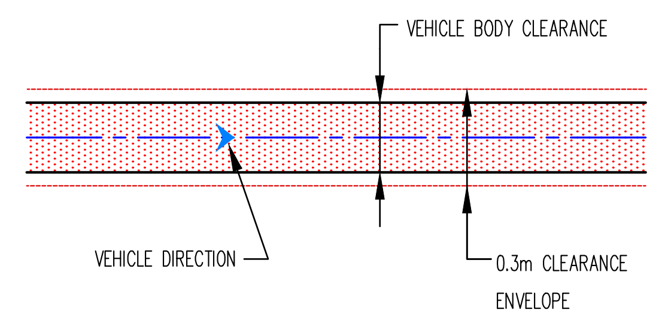


EXISTING BUS MOVEMENTS



FUTURE BUS MOVEMENTS

SWEPT PATH LEGEND:



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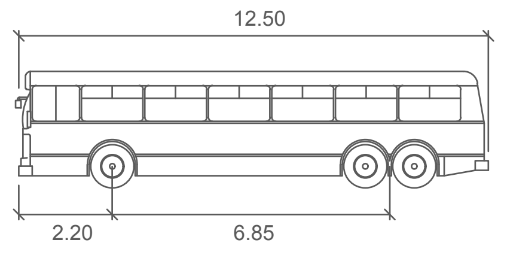
ACCORDANCE WITH AS2890.2



BUS

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Rev	Description	Eng	Draft	Date	Rev	Description	Eng	Draft	Date	Rev	Description	Eng	Draft	Date
P1	PRELIMINARY	MB	MB	05.08.19										

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Project
SCEGGS DARLINGHURST

PRELIMINARY

Sheet Subject
SWEPT PATH ANALYSIS

BUS (12.5m) ACCESS ALONG
ST PETERS STREET

Scale : A1
1:200
Drawn
MB
Authorised
MB

Job No
181375
Drawing No
SK03
Revision
P1

Plot File Created: Aug 05, 2019 - 4:23pm

Appendix B

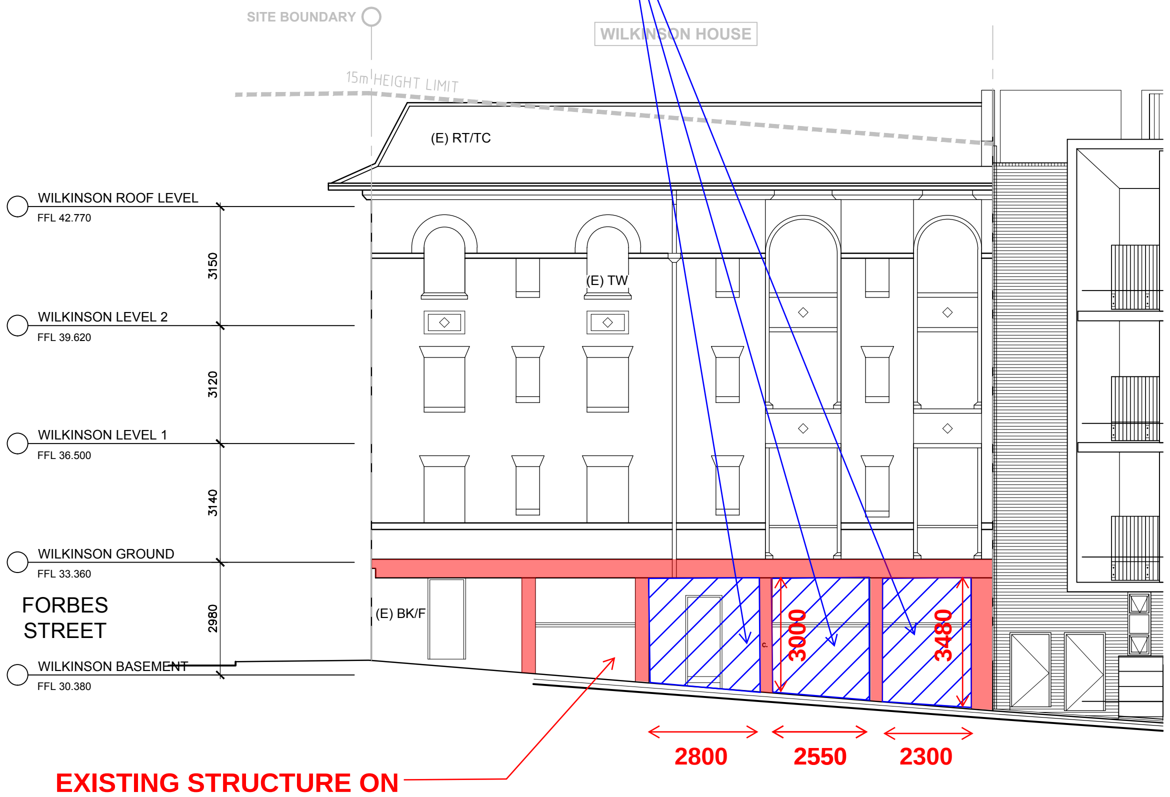
Site Truck Access Drawings

FACILITATING TRUCK ACCESS WITHIN SITE, OPTION 1

- Establish construction zone on St Peters Street
- Removal of existing brick infill panels, refer to zones below, without significant structural work to re-support the facade
- Provide access into the site for trucks to remove demolition spoil and excavation material

THE DEMOLITION ZONES BELOW DO NOT PROVIDE ENOUGH ROOM TO FIT

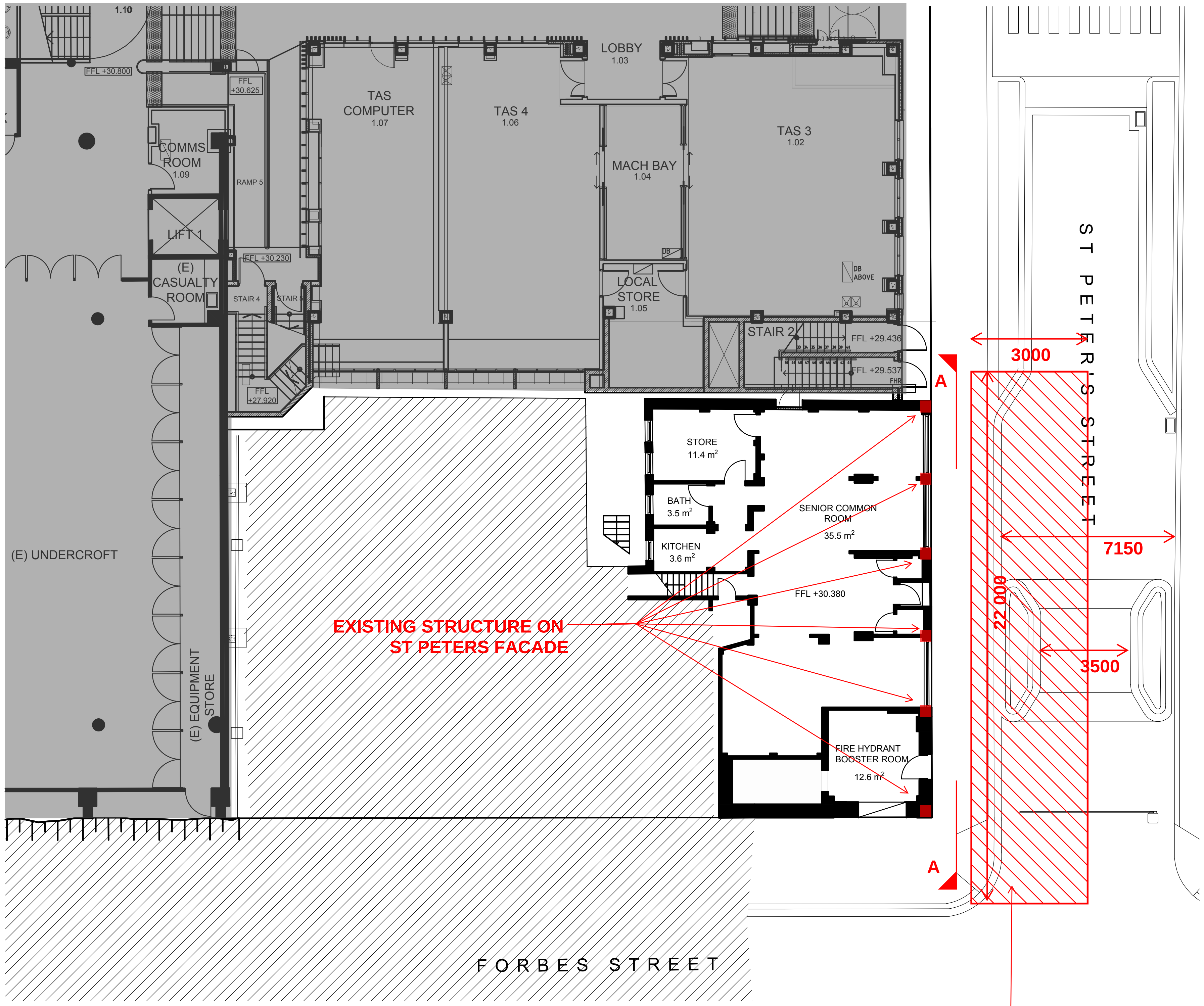
POSSIBLE DEMOLITION ZONE FOR ACCESS WITHOUT SIGNIFICANT STRUCTURAL WORK



EXISTING STRUCTURE ON ST PETERS FACADE

EXISTING WILKINSON HOUSE ELEVATION A-A

SCALE 1:200 @ A3
ALL DIMENSIONS LABELLED ARE APPROXIMATE, AND HAVE BEEN MEASURED ON SITE

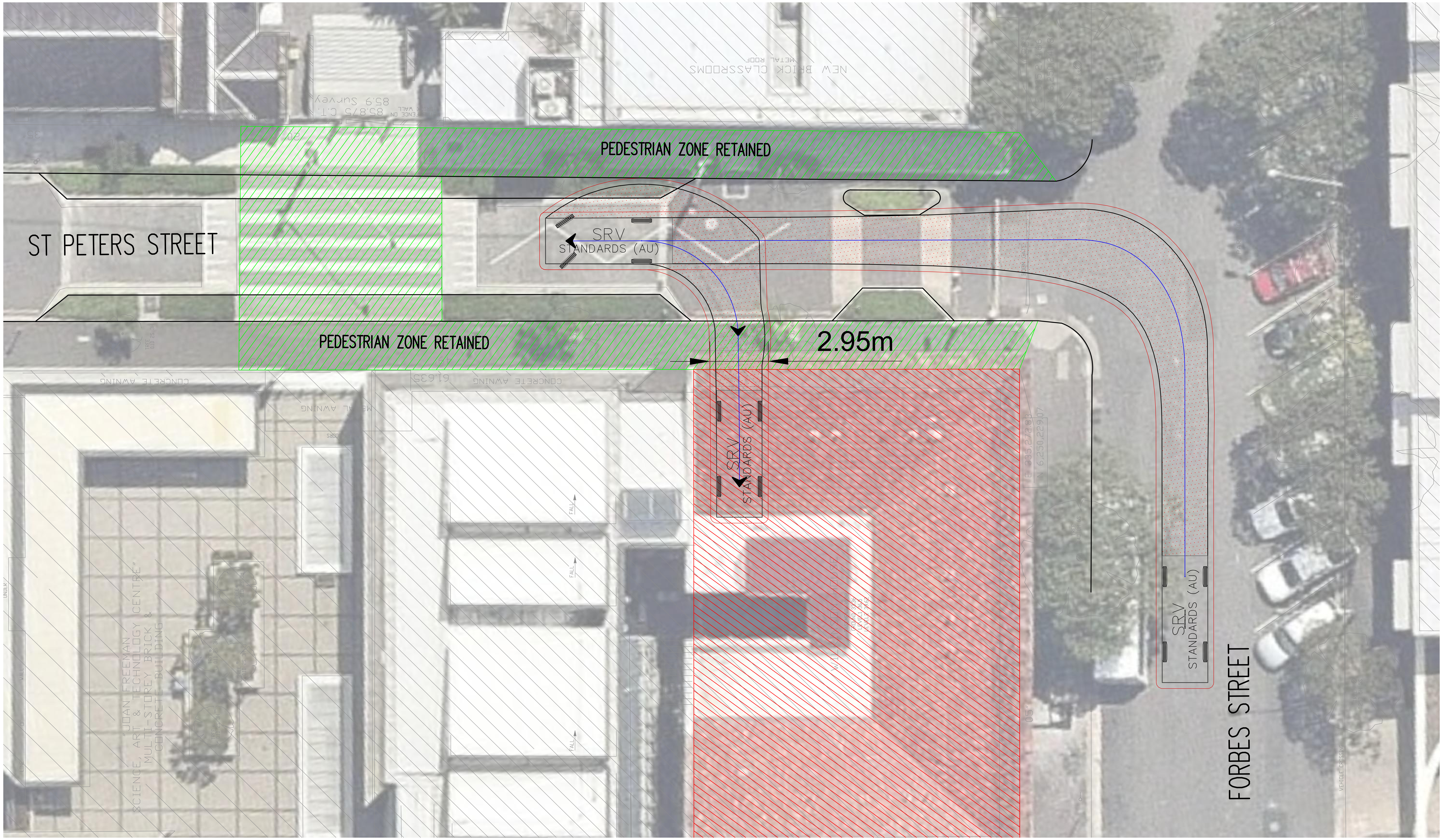


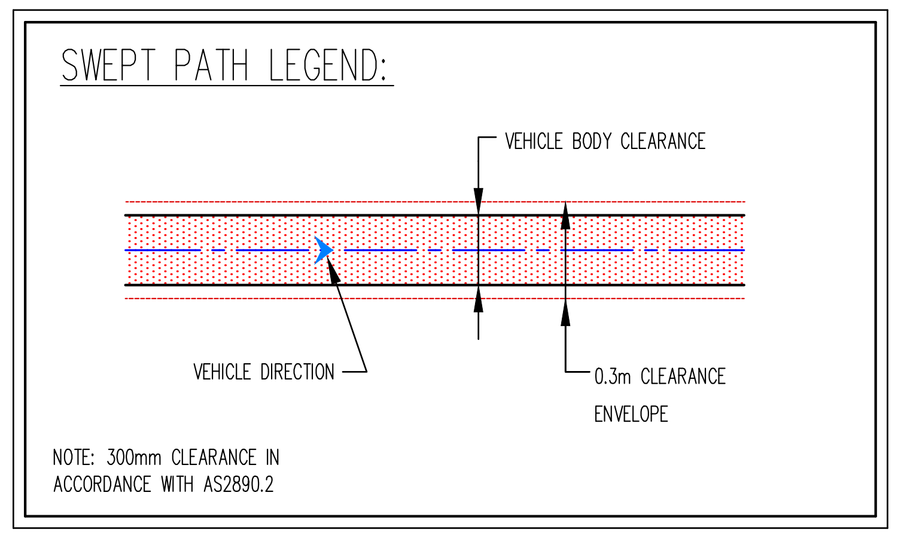
EXISTING STRUCTURE ON ST PETERS FACADE

CONSTRUCTION ZONE, POSITION UNDER REVIEW IN TURNING CIRCLE DRAWINGS

EXISTING BASEMENT FLOOR PLAN

SCALE 1:200 @ A3
ALL DIMENSIONS LABELLED ARE APPROXIMATE, AND HAVE BEEN MEASURED ON SITE





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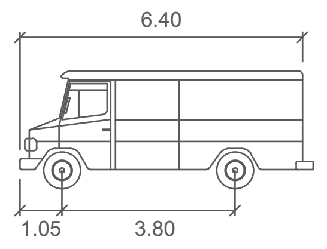

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PRELIMINARY

Rev	Description	Eng	Draft	Date
P1	PRELIMINARY	MB	MB	05.08.19

Architect
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Project
SCEGGS DARLINGHURST

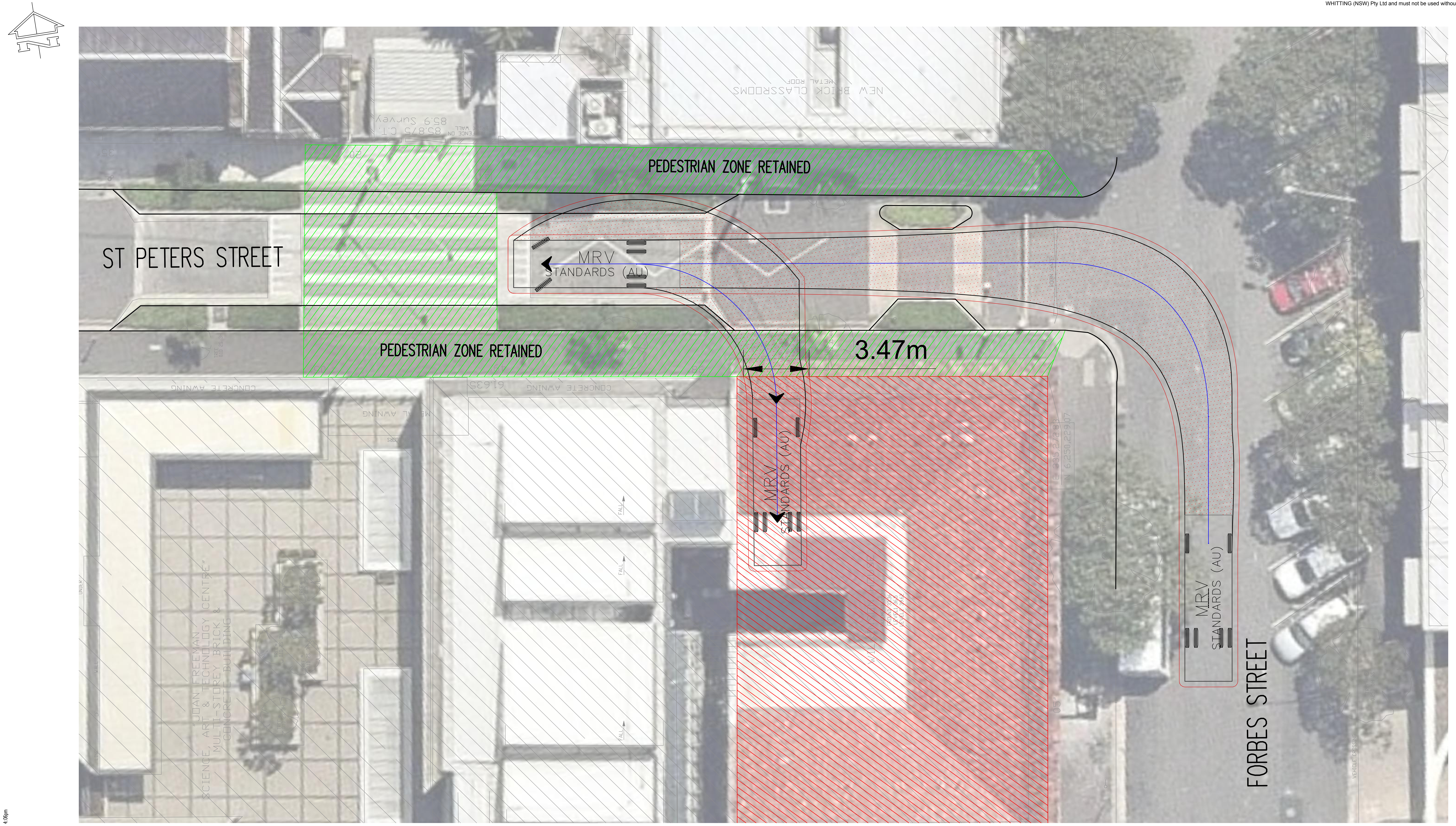
Sheet Subject
SWEPT PATH ANALYSIS

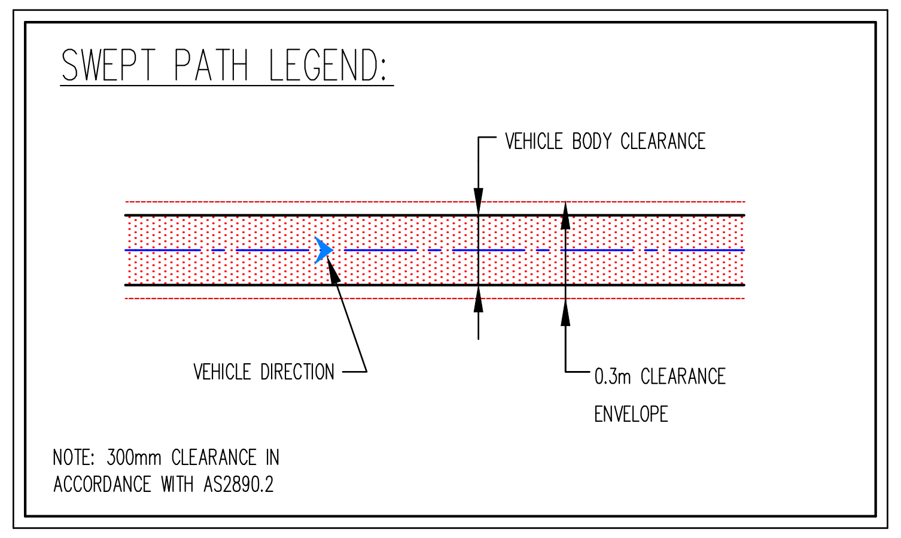
SRV (6.4m) ACCESS INTO SITE

Scale : A1	Drawn	Authorised
1:100	MB	MB

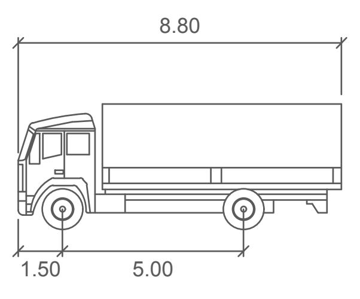
Job No	Drawing No	Revision
181375	SK01	P1

Plot File Created: Aug 05, 2019 - 4:03pm





NOTE: 300mm CLEARANCE IN ACCORDANCE WITH AS2890.2



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PRELIMINARY

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P1	PRELIMINARY	MB	MB	05.08.19

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Project
SCEGGS DARLINGHURST

Sheet Subject
SWEPT PATH ANALYSIS
MRV (8.8m) ACCESS INTO SITE

Scale : A1	Drawn	Authorised
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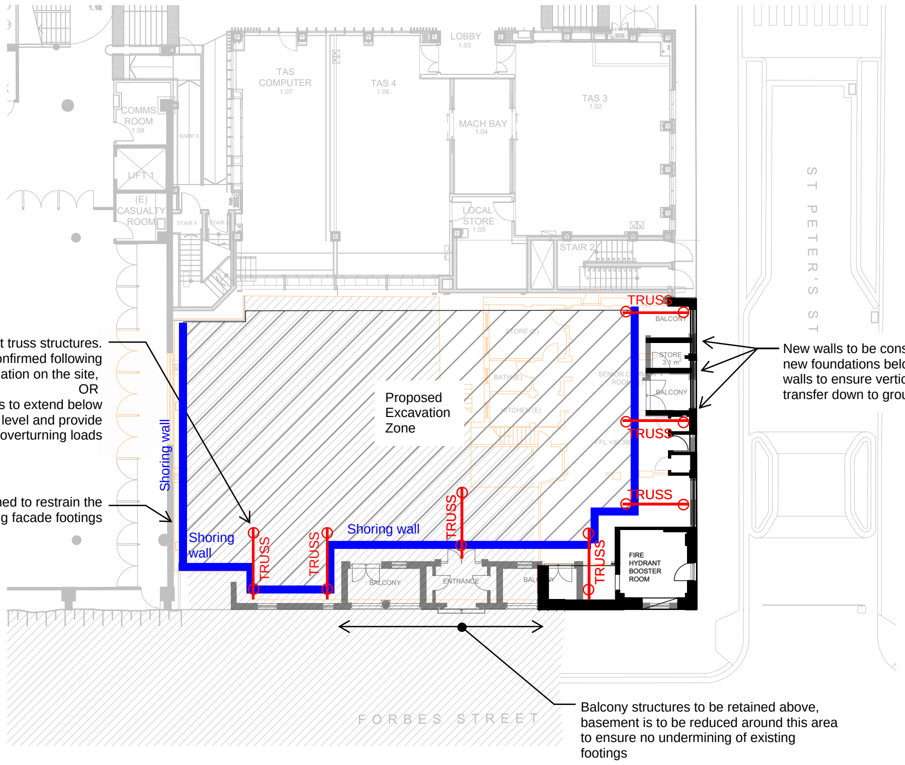
Appendix C

Wilkinson House Temporary Façade Retention Scheme – Option B-1



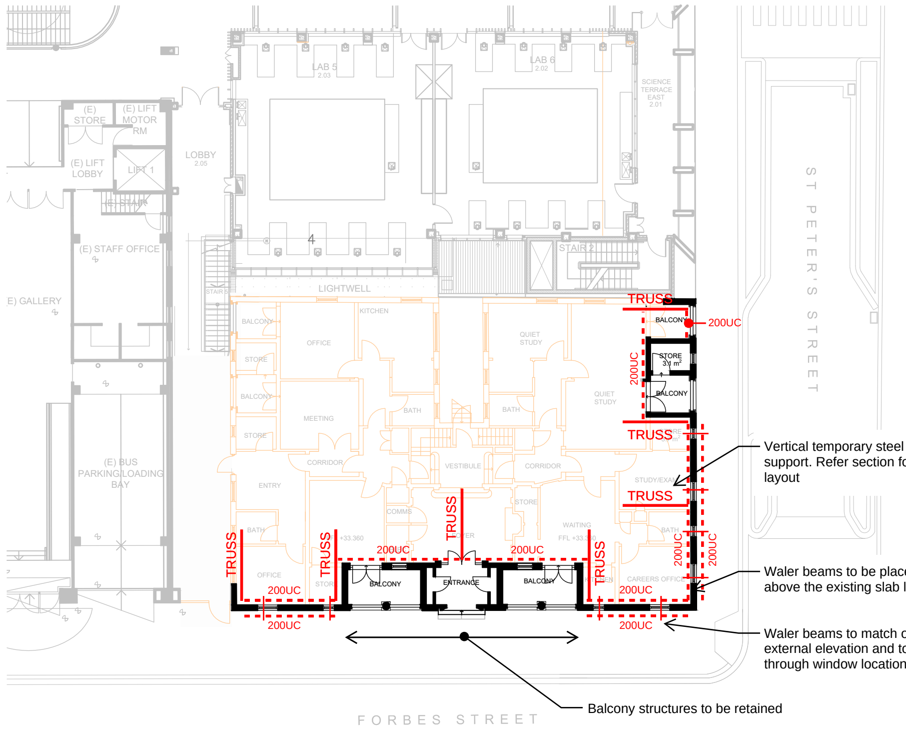
Temporary Piles to support truss structures. Depth to be confirmed following geotechnical investigation on the site, OR
Large pad footings to extend below proposed excavation level and provide sufficient weight for overturning loads

Shoring wall to be designed to restrain the toe of the existing facade footings

New walls to be constructed, and new foundations below balcony walls to ensure vertical load transfer down to ground level

Balcony structures to be retained above, basement is to be reduced around this area to ensure no undermining of existing footings

BASEMENT PLAN



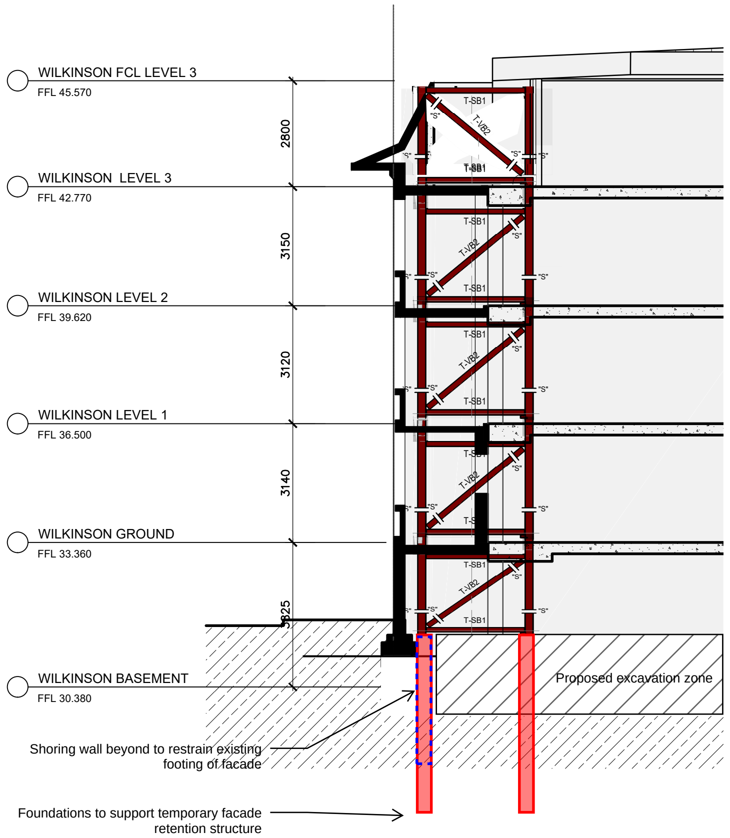
Vertical temporary steel truss support. Refer section for typical layout

Waler beams to be placed just above the existing slab level

Waler beams to match on external elevation and to clamp through window locations

Balcony structures to be retained

GROUND PLAN

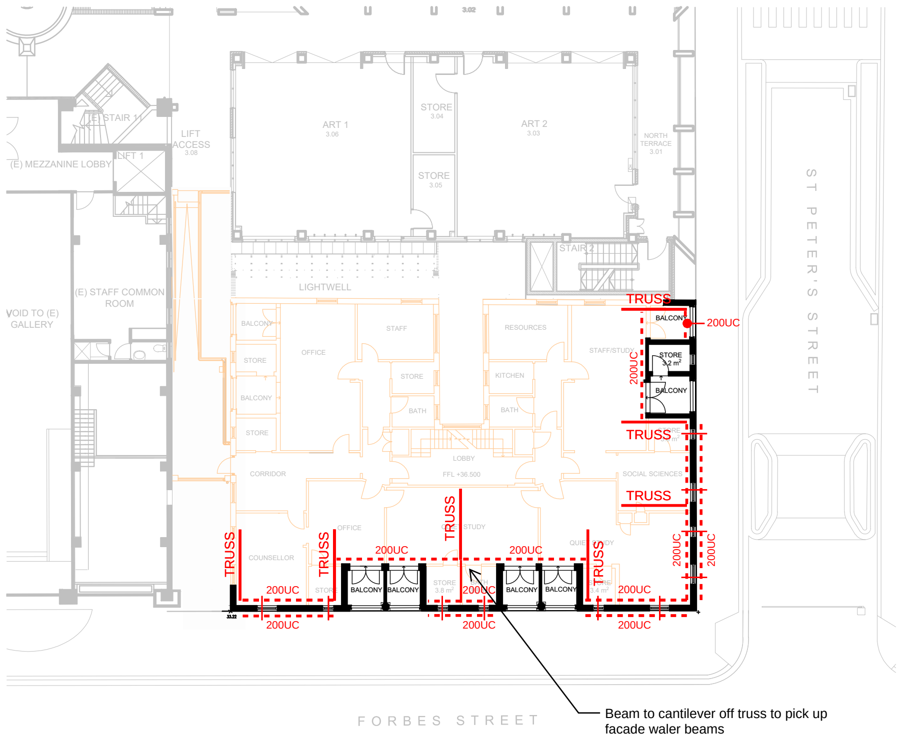


Shoring wall beyond to restrain existing footing of facade

Foundations to support temporary facade retention structure

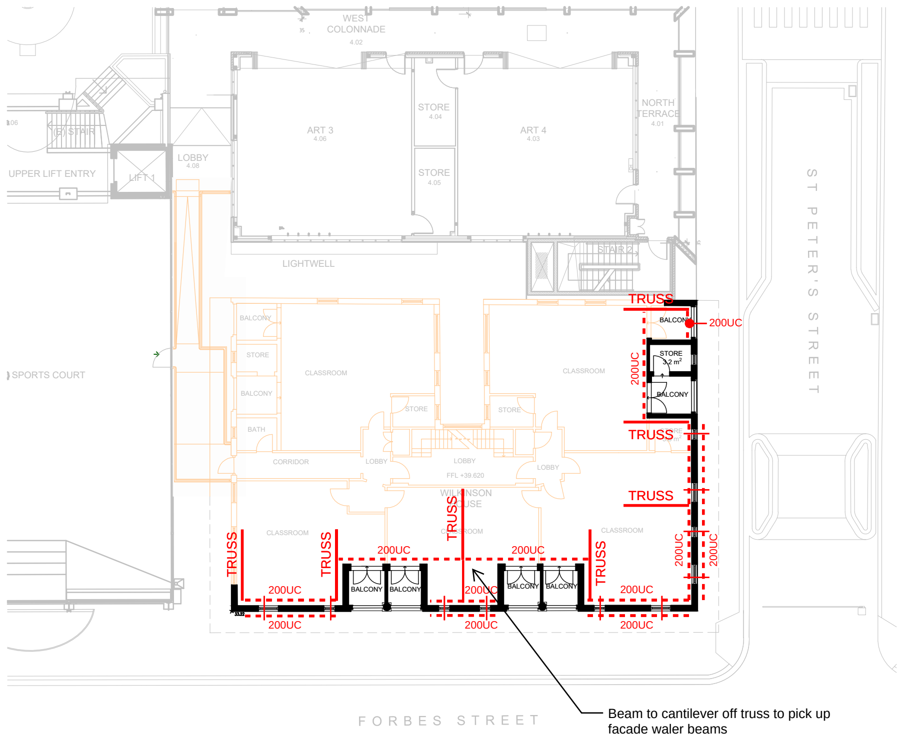
TYPICAL SECTION

TRUSS	MARK	SIZE
	T-SB1	310UC137
	T-VB2	250 X 9 SHS



Beam to cantilever off truss to pick up facade waler beams

LEVEL 1 PLAN



Beam to cantilever off truss to pick up facade waler beams

LEVEL 2 PLAN

STRUCTURAL DEMOLITION METHODOLOGY

1. INSTALL TEMPORARY PILES IN LOCATIONS INDICATED, FOR EXISTING GROUND LEVEL
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5. DEMOLISH EXISTING FLOOR STRUCTURES AROUND THE INSTALLED TEMPORARY STRUCTURES

Taylor Thomson Whitting

Appendix D

Report on Constructability of Retention System to Wilkinson House

Prepared for TTW

Review of Constructability of the Façade Retention System for Wilkinson House at SCEGGS

ICMP Group Pty Ltd
19-8-2019

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Introduction

This report will review the constructability of the façade retention system proposed for the Wilkinson House Project at SCEGGS.

In the review, the following has been considered: -

- Vibration Criteria set for heritage structures to limit the risk of vibration induced damage.
- Types of equipment and access and working space

Proposed Façade Retention System.

A façade retention scheme has been prepared by TTW and is attached in Appendix 1 for ease of reference.

The scheme requires piles to be drilled to base the temporary steel framed retention system.

The temporary façade retention will need to be completed prior to demolition of load bearing and façade restraining walls to maintain stability of the façade and structure. It is likely that the existing floors also provide some restraint to the stability of the existing façade.

The existing floor to floor height is approximately 3m – this includes the supporting structure of the floor. The clear structural height of each floor is considerably less than 3m.

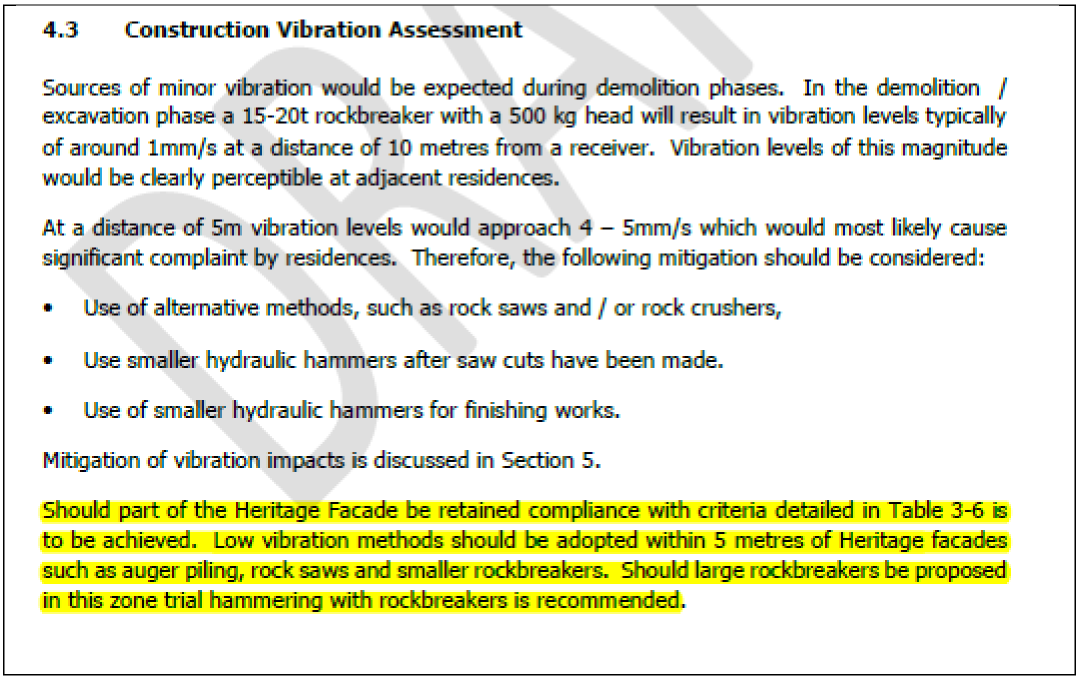
The existing building is founded on rock as seen by a limited visual inspection in the existing Fire Brigade Booster value room. Bore logs where conducted to nearby structures in 1994. These bore logs give an indication of the type and strength of the expected rock which is likely to be high strength slight fractures and weathering. This rock was found at RL29m in BH6 which is adjacent to the Wilkinson House. See Douglas Partners report in Appendix 2.

With the piles likely to be around 600mm diameter and of depth in excess of 6m, a substantially sized and powered piling rig will be required to drill these piles. It is also noted that these rigs require “swing” room of around 30° to work efficiently and ensure the integrity of the drilled pile prior to casting.

The project also requires other perimeter piles to be drilled to act as permeant shoring to the existing façade and building footings.

Vibration Criteria

Wilkinson Murray have prepared a report titled “SCEGGS Darlinghurst Masterplan & Stage 1 Project Application – Construction & Operational Noise Report” Report No 18180 Version B dated July 2019” (W/M Report) which highlights the limits to be set for construction induced vibration to minimize structural damage to the façade. An extract is



4.3 Construction Vibration Assessment

Sources of minor vibration would be expected during demolition phases. In the demolition / excavation phase a 15-20t rockbreaker with a 500 kg head will result in vibration levels typically of around 1mm/s at a distance of 10 metres from a receiver. Vibration levels of this magnitude would be clearly perceptible at adjacent residences.

At a distance of 5m vibration levels would approach 4 – 5mm/s which would most likely cause significant complaint by residences. Therefore, the following mitigation should be considered:

- Use of alternative methods, such as rock saws and / or rock crushers,
- Use smaller hydraulic hammers after saw cuts have been made.
- Use of smaller hydraulic hammers for finishing works.

Mitigation of vibration impacts is discussed in Section 5.

Should part of the Heritage Facade be retained compliance with criteria detailed in Table 3-6 is to be achieved. Low vibration methods should be adopted within 5 metres of Heritage facades such as auger piling, rock saws and smaller rockbreakers. Should large rockbreakers be proposed in this zone trial hammering with rockbreakers is recommended.

Figure 1 – Extract from W/M Report

The peak particle velocity (PPV) is set at 3mm/s for heritage structures. This is industry practice.

Given the types of piles that need to be drilled within the rock and the proximity to the existing façade being around 1m to the front pile, tests should be conducted to ascertain the suitability of the rig.

For the purpose of this exercise, we will assume that a rig can be found.

Equipment, Access, Manoeuvring and Working Space

Assuming we have found a rig that can achieve the set PPV criteria, the rig will need to be manoeuvred inside the existing building to be set up to commence operation. Rigs are of specific dimensions and require set up and working space to operate.

Given the loads imposed by rigs whilst traversing and operating, they need to be found on suitably solid substrates – this is around 300kPa but will need to be confirmed for the specific rig.

The existing building is partially excavated to the north with the south half being unexcavated and around 3m higher. We will need to consider both set ups for the rig as it cannot traverse between levels once inside the building.

The existing façade is limited in readily available openings to “walk in” machines. Lowering machines in over the façade and down to the ground level is structural impractical given the existing façade restraint system.

The existing floor to floor height is also limited and the existing floors also provide some façade restraint.

Loading bearing and façade restraint walls also need to be considered.

Equipment

This purpose of this exercise in the section is to ascertain how “low head height piling rigs” could access, manoeuvre and work within the confines of building to drill the façade retention piles and the shoring piles. For the purpose of this exercise we will look at two piling rigs that are known to be available in the Sydney market and dimensional / special data is available.

It is unknown at this stage if the rigs can meet the vibration criteria or that they are of sufficient size / power to drill the piles in the rock. This is to be further investigated.

A selection of “low head height” rigs from different piling contractors will be consider for the exercise. They are: -

- Vermeer HL 2500 Rig – from Franki
- 515 Short Mast Rig – from Arvo Piling
- Komatsu PC 75 excavator base – from MGI Piling – *likely will have insufficient power to drill the required piles. Will need to be further investigated.*
- Mait Baby Drill – from MGI Piling – *very unlikely will have enough power to drill the required piles.*

The data sheets for these rigs are included in appendix 3.

A summary of their dimensions are below.

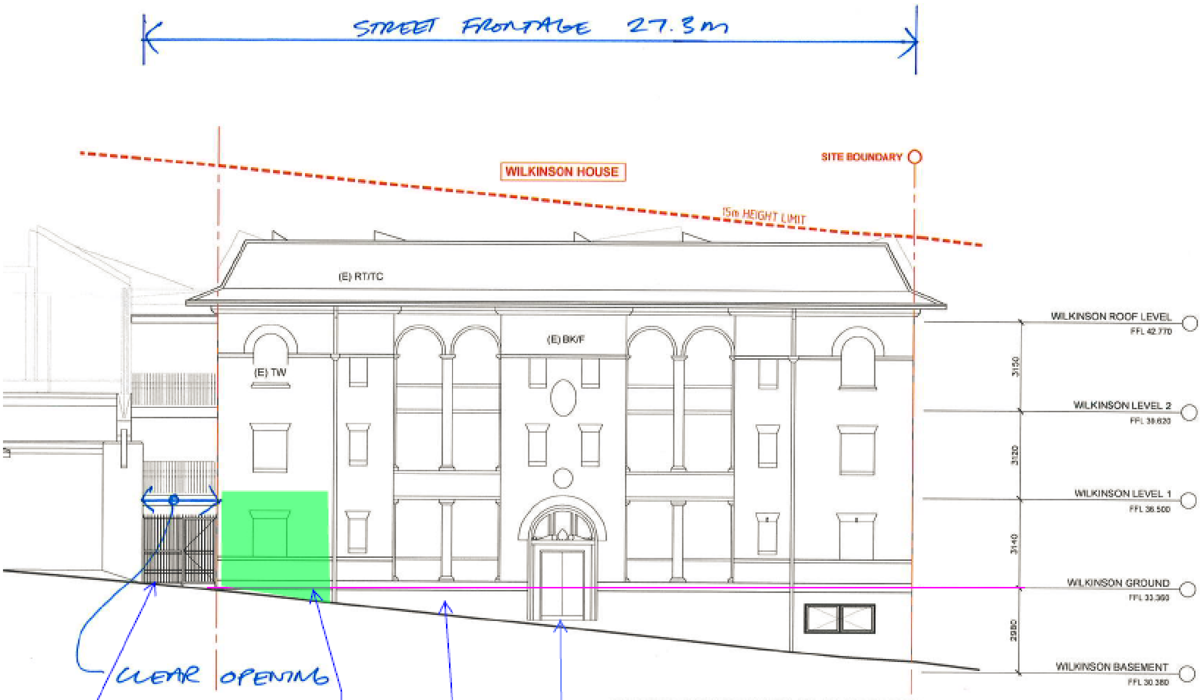
Rig	Travel width	Travel length	Travel Height	Operating Height
Franki	2.0m	9.6m	2.5m	9m?
Arvo Piling	3.2m	10.4m	3.3m	5.7m
MGI Piling Komatsu PC75	*2.1m*	*5.9m*	*2.8m*	*6.5m*
MGI Piling Baby Drill	*1.5m*	*4.2m*	*2.1m*	*3.75m*

Access

The rigs vary in dimensions (length by width and height) and weight.

Two access points will be required to access the two different ground levels. These will be from Forbes St and St Peters St

The existing façade will need to be cut and demolished to allow the rig to access the site. Likely areas to access the site may be as shown below.



STREET FRONTAGE 27.3m

CLEAR OPENING

2670 mm

EXISTING FORBES STREET (EAST) ELEVATION

Due to its length the machine cannot turn right into site - note structural walls inside

Possible opening to be made - BUT unlikely machine can turn within site due to structural walls

Protected area

OPENING (COLUMN DEMO)

3045 mm H x
5275 mm W.

Possible opening for machine access

OPENING

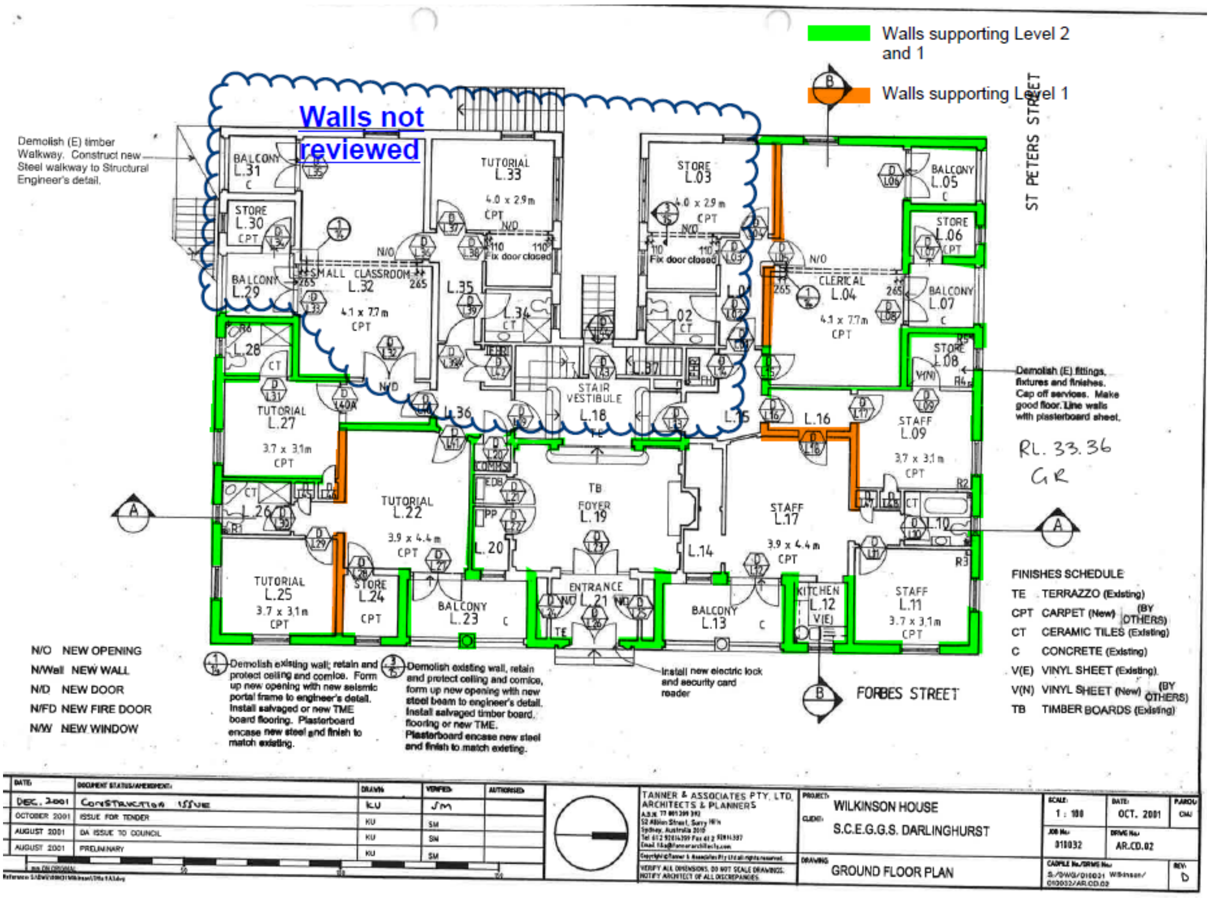
3280 mm H x
2360 mm W

Manoeuvring and Working Space

Once the machines have tracked through the newly created façade openings, they need to manoeuvre into the pile position.

A review of the existing Wilkinson House drawings shows a considerable amount of load bearing walls that will need to be respected.

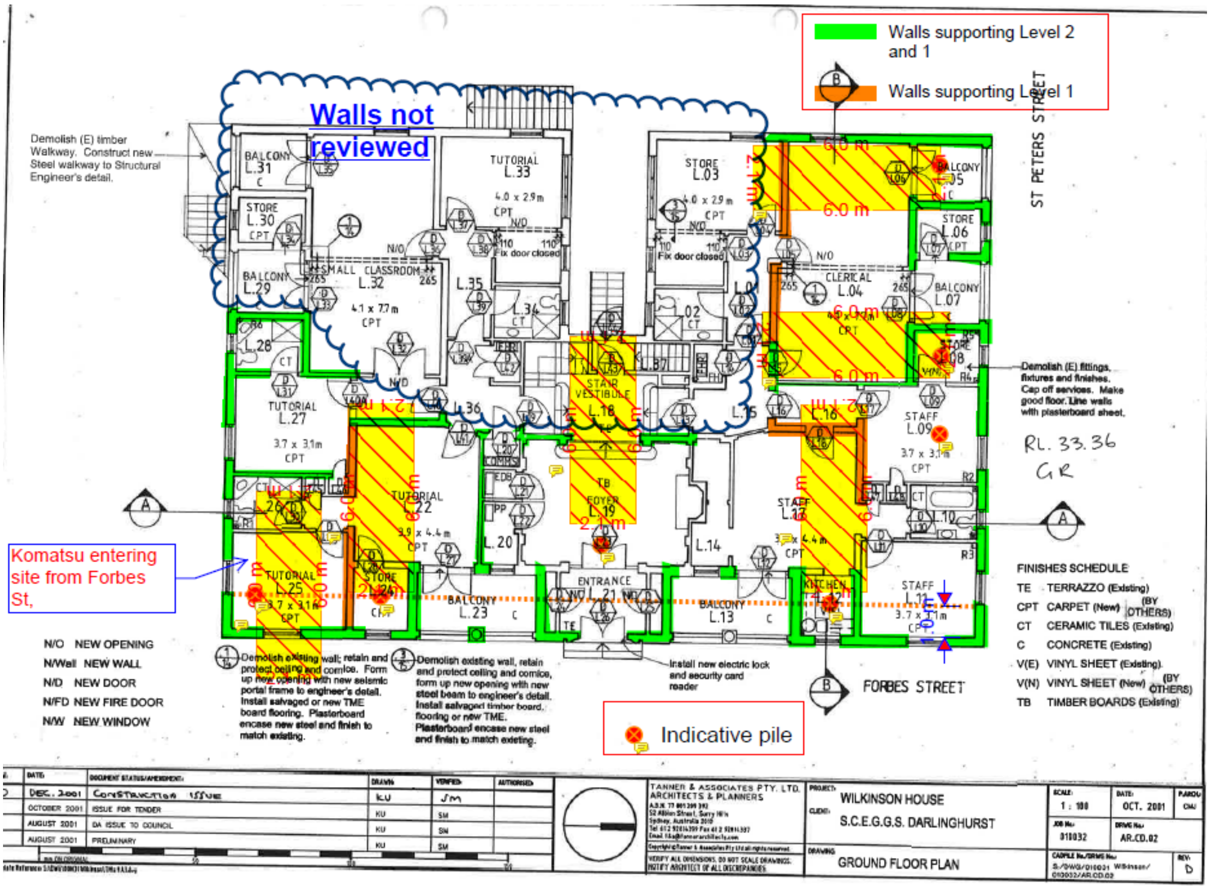
The figure below shows the considered load bearing walls.



It is evident from the mark up below which has the indicative position of the front façade retention piles and a template of the Komatsu that the machine cannot enter the site and manoeuvre to a pile position without major demolition of load bearing walls.

The analysis of positioning the rig over the required pile position and providing working space, will require further demolition of load bearing walls as well as the existing floors to provide height for the rig to set up and operate over the pile position. The consideration of lifting in reinforcement cages into the newly drilled piles prior to casting concrete has not been reviewed given the considerable challenges with getting the rig into position.

Also not considered is the requirement of a temporary retaining wall and back filling between the high level and low-level ground levels, that can take the piling rig surcharge loads for the rig to work safely.



Conclusion

This exercise was to review the constructability of the façade retention scheme.

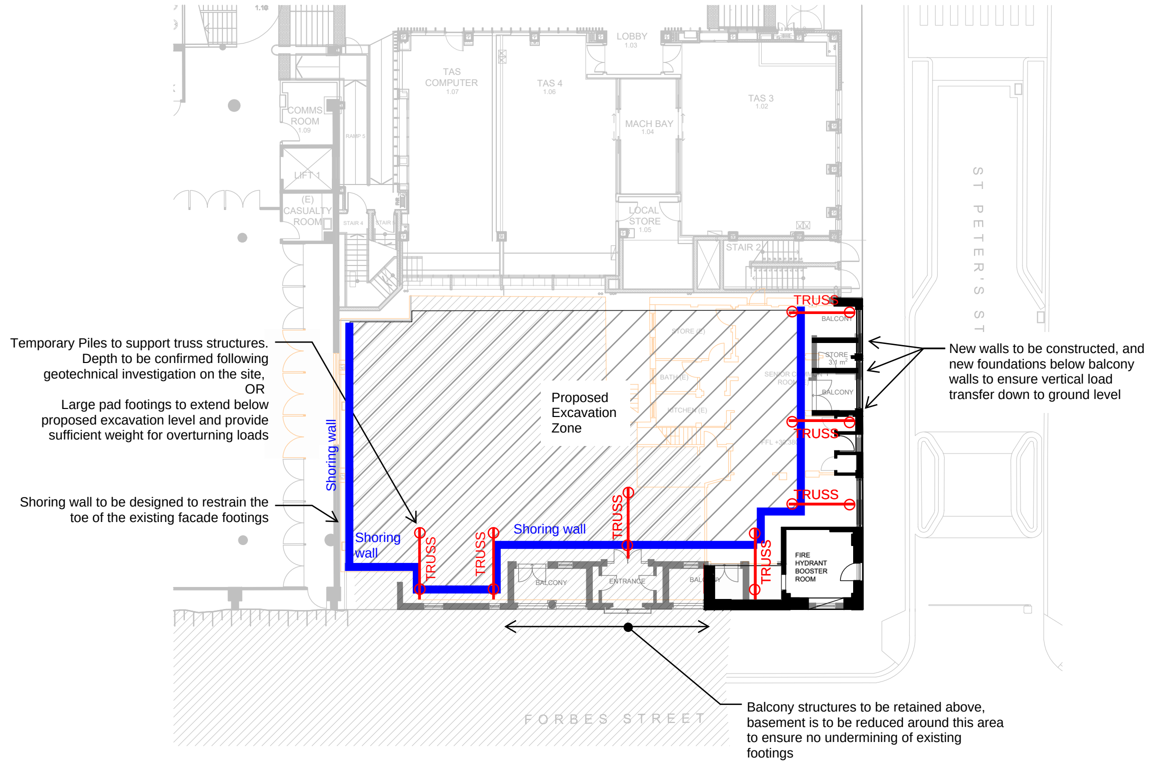
Fundamental to the scheme is the installation of the piles prior to any major demolition which may cause damage to the facade.

A review of low head height rigs from various piling contractors has shown that drilling these piles is not possible without: -

- Creating substantial openings in the existing façade to allow the machine to enter and exit the site. The openings are through the load bearing façade
- Substantial demolition of internal load bearing walls and existing slabs to allow the rig to manoeuvre and work. The removal of these walls and floor will likely affect the stability of the existing façade.

The process required to implement a façade retention system to this project will most likely cause damage to the façade and impose undue risk during construction to workers and public.

Appendix 1



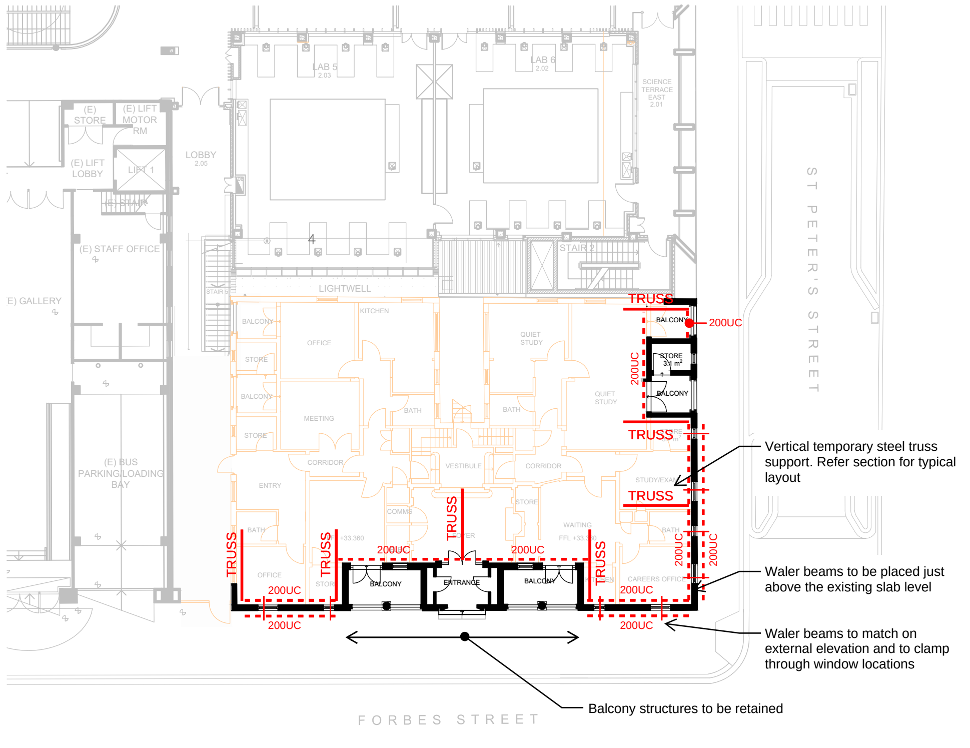
Temporary Piles to support truss structures. Depth to be confirmed following geotechnical investigation on the site, OR Large pad footings to extend below proposed excavation level and provide sufficient weight for overturning loads

Shoring wall to be designed to restrain the toe of the existing facade footings

New walls to be constructed, and new foundations below balcony walls to ensure vertical load transfer down to ground level

Balcony structures to be retained above, basement is to be reduced around this area to ensure no undermining of existing footings

BASEMENT PLAN



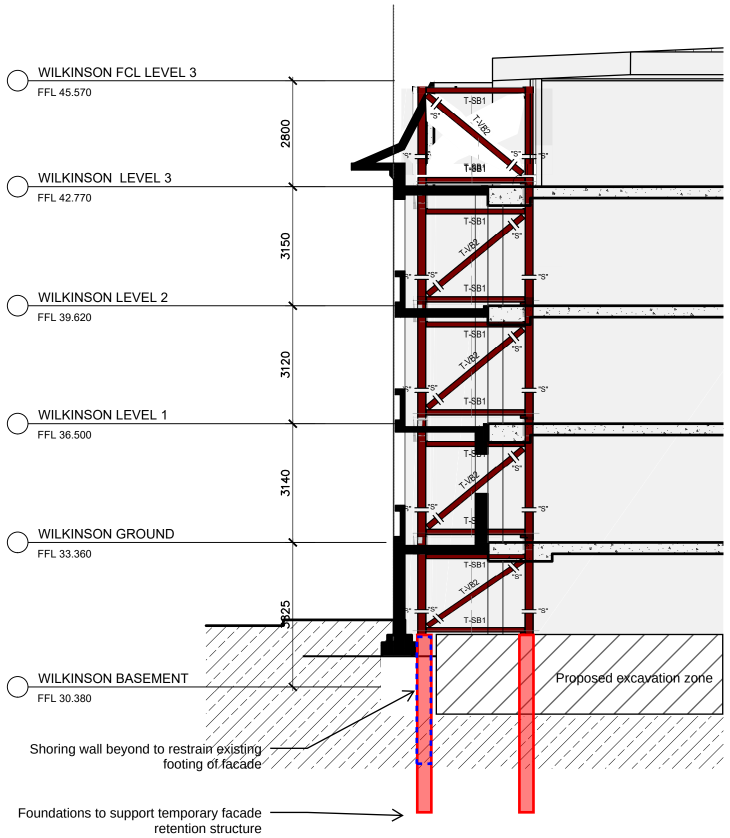
Vertical temporary steel truss support. Refer section for typical layout

Waler beams to be placed just above the existing slab level

Waler beams to match on external elevation and to clamp through window locations

Balcony structures to be retained

GROUND PLAN



Shoring wall beyond to restrain existing footing of facade

Foundations to support temporary facade retention structure

TYPICAL SECTION

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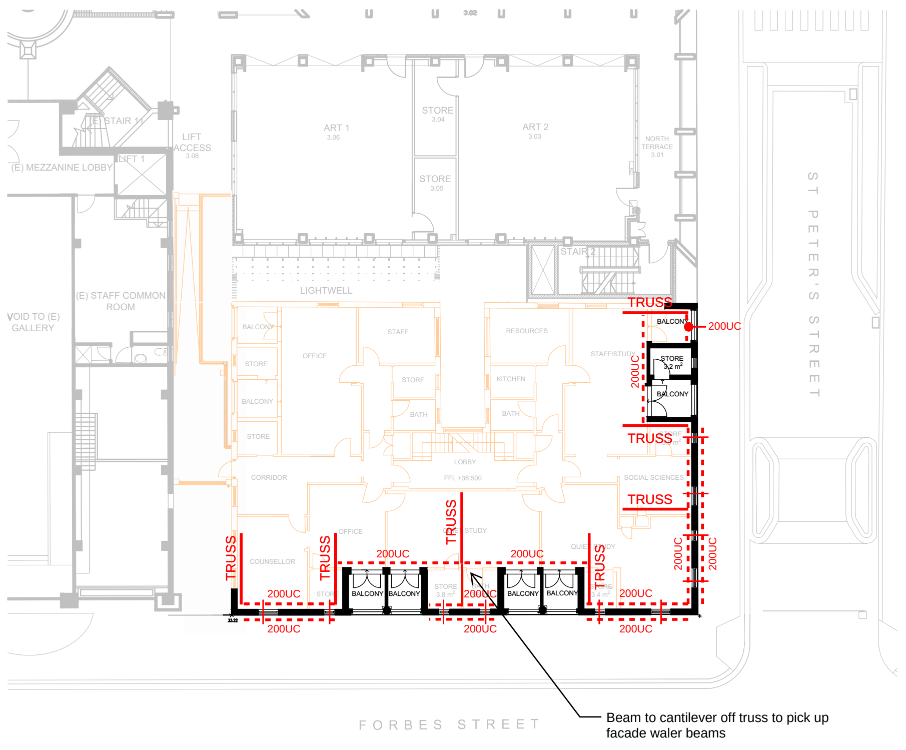
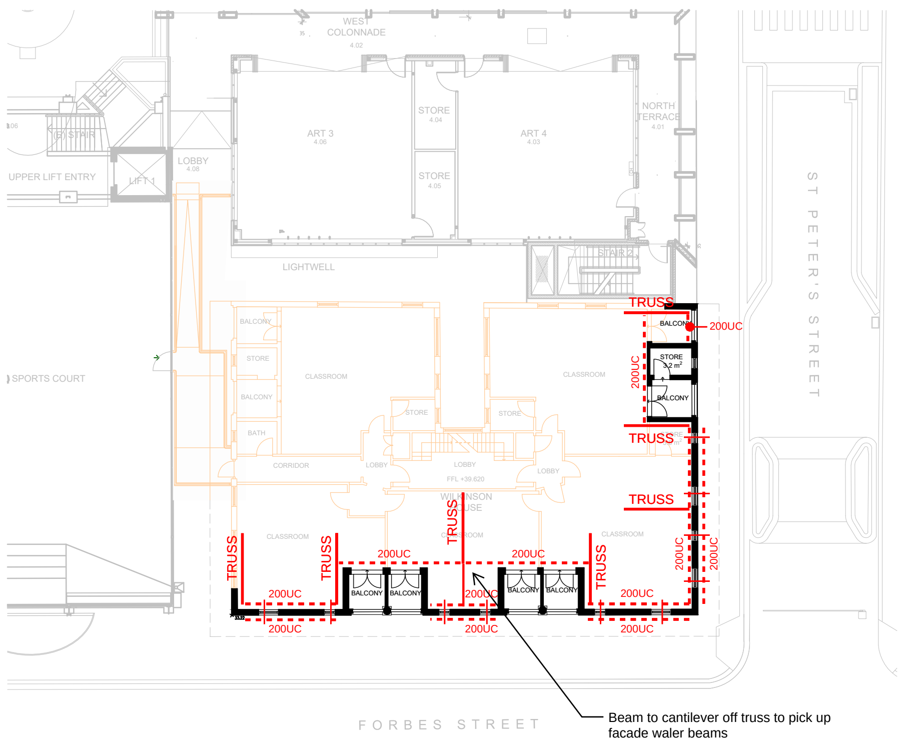


Beam to cantilever off truss to pick up facade waler beams

LEVEL 1 PLAN



Beam to cantilever off truss to pick up facade waler beams

LEVEL 2 PLAN

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TTW Taylor Thomson Whitting

Appendix 2

Douglas Partners
Geotechnics • Environment • Groundwater

Integrated Practical Solutions

REPORT
on
GEOTECHNICAL INVESTIGATION

PROPOSED SCIENCE AND TECHNOLOGY BUILDING
SYDNEY CHURCH OF ENGLAND
GIRLS GRAMMAR SCHOOL
DARLINGHURST

Prepared for
SCEGGS Darlinghurst

Project 45427
April 2008

Douglas Partners
Geotechnics • Environment • Groundwater

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ACEA
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Appendix A: Locality and Borehole Plans
Appendix B: Test Bore and Test Pit Logs and Notes Relating to this Report
B1: Previous Investigations
B2: Current Investigation

MJT:jlb
Project 45427
5 April 2008

GEOTECHNICAL INVESTIGATION
PROPOSED SCIENCE AND TECHNOLOGY BUILDING
SCEGGS DARLINGHURST

1. INTRODUCTION

This report details the results of a geotechnical investigation carried out on the site of the proposed Science and Technology Building to be constructed within the grounds of Sydney Church of England Girls Grammar School in Forbes Street (SCEGGS), Darlinghurst. The work was undertaken for SCEGGS and was carried out in consultation with Hughes Trueman, Consulting Engineers for the project.

The project is for the construction of a new six storey school teaching building which will include two levels of below ground parking. Site investigation was carried out to determine the subsurface conditions and subsequently provide advice on:

- Suitable foundation types and design bearing pressures;
- Stability of permanent and temporary excavation slopes;
- The depth of the water table and potential seasonal fluctuations;
- Seismic design parameters for earthquake loading;
- Recommendations on methods of underpinning existing structures;
- Recommended vehicle and pedestrian pavement profiles.

The investigation comprised test bore drilling and test pit excavation followed by engineering evaluation and geotechnical analysis where appropriate. Details are given in the report together with comments on design and construction practice.

The geotechnical investigation was carried out simultaneously with a contamination assessment and waste classification for material to be disposed of off-site as a result of the basement excavation. Laboratory testing for the contamination assessment is currently in progress and the results of this investigation will be reported separately.

2. PREVIOUS INVESTIGATION

In June 1994, Douglas Partners (DP) carried out a geotechnical investigation for the sports building which is located immediately to the south of the proposed structure. This investigation comprised six bores drilled to depths of up to 8.5 m below the existing surface level to obtain detailed information on the soil and rock stratigraphy. The results of the investigation were provided in a Report No 20080 for Tierney & Partners, Consulting Engineers for the sports building design.

The investigation indicated that sandstone bedrock was generally located less than 0.5 m below surface level. Furthermore, the sandstone was medium or high strength from near the surface and DP assessment was that the vertical excavations could be made in the sandstone but that rock bolts would be required to stabilise areas where steeply dipping joints intersect the excavation faces at unfavourable orientations. It was also determined that localised shotcreting of low and very low strength bands would be required to minimise weathering and deterioration of these beds.

The investigation also comprised mapping of the sandstone cliff-face on the site which was eventually excavated for the sports complex. This mapping identified a weak zone in the Hawkesbury Sandstone and contended that this weak zone could have been caused by a fault in the sandstone or by an igneous dyke intersecting the excavation at about right angles to the Forbes Street frontage. If the weak zone was a dyke, it is possible that similar geological features could intersect the site of the current proposed building.

3. SITE DESCRIPTION AND GEOLOGY

SCEGGS is located in an intensely developed residential and commercial area of Darlinghurst about 1 km from the Sydney Central Business District. Overall the site occupies an irregular shaped area measuring about 150 m x 60 m and is currently occupied by many school buildings which range in age from relatively recent to in excess of 50 years old. The school itself is located on the corner of Forbes Street and St Peters Street with the particular site being currently developed having a frontage of approximately 40 m on St Peters Street and an overall depth of some 30 m. Most of the area being considered for development is currently used as car parking with the eastern portion presently occupied by an elevated demountable classroom over car parking. Immediately adjacent to the development site, to the east, is Wilkinson House which is a multi-storey brick building of some considerable age. On the southern boundary of the development site is Barbara Chisholm Hall and on the western boundary is the backyard of private residences.

SCEGGS is located in undulating country with gentle slopes to the north towards Woolloomooloo Bay estimated to be about 5%. The area being considered for development is relatively level with an overall fall in a north-westerly direction diagonally across the site estimated to be of the order of 2 m.

The 1:100 000 Series Geological Sheet for Sydney indicates that the site is underlain by Hawkesbury Sandstone. This geological formation usually comprises medium to coarse grained quartz sandstone with minor shale lenses. Previous investigations on the site confirm the geological mapping with Hawkesbury Sandstone at shallow depths below the surface. Sandstone was also exposed in a cutting along Forbes Street prior to excavation for the existing sports hall.

An extract from the aerial photograph of the area is given on Drawing 1 in Appendix A and shows SCEGGS school on the corner of Forbes Street and St Peters Street, Darlinghurst. The proposed development site is the open car parking area and adjoining white roof area facing St Peters Street.

4. FIELD INVESTIGATION

4.1 Methods

The field investigation for the geotechnical assessment was carried out simultaneously with the contamination investigation and comprised five bores and three test pits conducted at the locations shown on the site plan in Appendix A.

Bores 101, 102 and 103 were drilled to depths of about 10 m below existing surface level using a truck mounted auger/rotary drilling rig. Initially the bores were advanced using spiral flight augers to the point of refusal on bedrock. Thereafter the bores were continued using rotary drilling techniques to obtain 50 mm diameter cores of the bedrock strata. Bores 104 and 105 were drilled using spiral flight augers through the soils to collect samples for contamination testing and were terminated at refusal.

Test Pits 202 and 203 were excavated by hand to determine bedrock levels and possibly expose the foundations of adjacent buildings. Originally it was intended to excavate a Test Pit 201 adjacent to Wilkinson House but inspections indicated that the footings of Wilkinson House are supported by Sandstone Bedrock above the level of the car parking beneath the demountable building and tennis court. Consequently it was considered sufficient to map the sandstone outcrop and photograph the footings to provide information on the need to underpin footings of Wilkinson House.

4.2 Results

Details of the conditions encountered in the test bores and test pits are given on the respective logs in Appendix B which also contains notes on the standard terms used to classify the strata and on the strength classification of bedrock.

The current investigations indicate a profile comprising about 1 m of filling overlying weathered sandstone and then medium and high strength sandstone from depths of 0.35 m to 1.0 m. The medium and high strength sandstone had some moderately weathered zones in the upper 2 m as indicated by some core loss and clay filled joints. Below depths of about 3 m, however, sandstone was generally medium and high strength fresh unbroken rock with few defects.

Table 1 below summarises the results of the current series of bores together relevant bores conducted in the vicinity during the previous investigation.

Table 1 – Summary of Bore Data

Data	**RL of Interface of Strata (m)**					
Description	**BH4**	**BH5**	**BH6**	**BH101**	**BH102**	**BH103**
SL	33.2	30.3	30.1	28.15	27.15	28.28
FILLING/ CLAYEY SAND – Topsoil, Sandstone rubble and weathered residual soil						
	32.95	29.98	29.20	27.15	26.45	28.08
SANDSTONE – Extremely low strength	NE	NE	NE			
	32.95	29.98		26.91	26.15	27.93
SANDSTONE – medium and high strength, moderately weathered						
	32.95	29.98	29.20	25.68	23.00	25.58
SANDSTONE – medium or high strength fresh, slightly fractured						
	29.7	26.3	28.1	18.15	17.15	18.28
	BD	BD	BD	BD	BD	BD

SL = Surface level
NE = Not encountered
BD = Bore Discontinued

The results indicate relatively uniform conditions with approximately 1 m of filling and weathered material overlying medium or high strength bedrock. The bedrock surface appears to be dipping generally in a north-westerly direction, following the natural surface contours. The overall level difference diagonally across the proposed development site of the surface of the medium or high strength rock appears to be of the order of 4 m from the previously drilled Bore 5 to Bore 102 in the north-western corner of the site.

No free groundwater was observed during the current investigation phase and this is consistent with observations made during the previous investigation in 1994. It is expected, however, that after periods of heavy rainfall some seepage will occur along the bedrock surface. The permanent water table within the intact bedrock is expected to be at many tens of metres below the current site level with groundwater flow along bedding planes and through vertical joints being extremely low. Conditions encountered at the three proposed test pit locations are also described on the test pit logs in Appendix B. The conditions are to be confirmed at a later date but bedrock is above the surface level at TP 201.

5. PROPOSED DEVELOPMENT

It is understood that the proposed development will comprise a new six storey school teaching building with two levels of below ground parking. The building will be a reinforced concrete frame structure with column working loads indicated by the consulting engineers to be in the range of 4,000 to 5,000 kN. The building abuts existing buildings which means that underpinning of adjacent footings will be required if they are not founded on sandstone bedrock.

The excavation for the construction of the underground car park is expected to extend to depths of 6 – 7 m below existing road level. As the car park will extend to the edges of the site, vertical excavations will be required for the car park construction.

6. ENGINEERING EVALUATION

6.1 Ground Conditions

The investigation indicates that the site is underlain by approximately 1 m of filling and weathered material over generally medium or high strength bedrock. Whilst these conditions are favourable insofar as vertical excavations are generally feasible (as indicated below) and relatively high bearing pressures can be adopted, on a small site such as this excavation can be difficult and vibration issues become critical in carrying out the bulk excavation works. These aspects of the development are discussed in the following subsections of the report.

6.2 Excavation

The excavation for the proposed basement construction will be up to about 7 m below existing surface level and will therefore be mostly in medium to high strength sandstone with some minor low or very low strength bands. Generally, it is considered that the excavation of sites of this size underlain by medium and high strength sandstone, would be difficult and under normal circumstances would need a heavy bulldozer, such as a D10 (or larger), and also the use of rock breakers to break some of the stronger layers and to trim the final excavation faces. However, due to the proximity of buildings which are sensitive to vibrations it may be necessary to utilise a rock saw around the perimeter of the excavation and to use small rock breakers to assist in the excavation so as to limit the vibration of adjacent structures. For this reason it is suggested that potential excavation contractors be provided with the test bore logs and core photographs and be required to make their own assessment of the equipment needed to carry out the excavation, being mindful of the need to limit vibration so as to not cause damage to nearby residences and other school buildings. It should also be mandatory for an excavation trial to be carried out using the equipment proposed for the work before the main excavation work commences to establish whether the vibration limits given below can be achieved. If the trial proves satisfactory, then bulk excavation works could commence but if the vibration levels are too high it may be necessary to get the contractor to either adjust their excavation techniques or to utilise smaller equipment.

6.3 Excavation Support

Whilst it should be generally possible to excavate medium and high strength sandstone vertically it is considered likely that rock bolts or pins may be required to stabilise areas where steeply dipping joints intersect the excavation faces at unfavourable orientations. Localised shotcreting of low and very low strength bands may also be required to reduce weathering and deterioration of these bands.

In order to determine the requirement for rock bolts or pins it is recommended that inspections of the excavation faces be undertaken by an experienced geotechnical engineer or engineering geologist at regular intervals during construction. For a depth of excavation of 7 m it will probably be necessary for the inspections to take place on each excavation face at least twice so that remedial measures on the upper part of the excavation can be implemented before excavation continues to a point where stabilisation measures might be difficult to install and safety of the workers in the site is compromised.

While the overburden soils and filling are likely to be relatively thin (ie generally less than 1 m) these materials should be battered back at a maximum slope of 2H :1V. Alternatively, they could be supported by a retaining structure.

6.4 Underpinning of Adjoining Structures

The investigation indicates that the site is underlain by a relatively shallow depth of filling and soil overlying medium strength bedrock. The foundations of the adjacent structures also appear likely to be supported on medium strength bedrock so there appears to be no need for underpinning of adjoining structures. It is, however, recommended that the entire footings of all buildings near the excavation face be inspected when excavation commences to ensure that the conditions observed on the site during the investigation are representative of all conditions beneath the footings of existing buildings. If underpinning is required it will have to be done in short panel no greater than about 1.5 m lengths by excavating under the footings and providing temporary support until concrete blade walls can be installed from the underside of the footing down to competent bedrock.

6.5 Foundations

The foundation material underlying the proposed science and technology building will mostly comprise medium to high strength sandstone with the possibility of some minor low or very low strength bands. Pad or strip footings founded on this material are considered to be suitable footing types. For these conditions it is considered that the building footings could be designed on the basis of an ultimate bearing pressure of 20 MPa. Even with a very low geotechnical strength reduction factor it is still possible that excessive settlements could occur so it is recommended that a maximum allowable bearing pressure of 3.5 MPa be adopted without the need for any spoon testing in the foundation excavation or 6 MPa if spoon testing is undertaken in at least half of the footing excavations during construction.

During the previous investigations weak zones were identified in the rock face along Forbes Street. Although the area where the weak zones occurred is located some distance from the present site, it was contended that the weak zones may have been caused by an igneous dyke. If this were correct then the current site may also be intersected by such weak zones. It is therefore imperative that all foundations be inspected to ensure that foundations conditions are not impacted by igneous intrusions.

6.6 Vibrations

Excavation of the medium and high strength rock will cause some vibration but with care this can be maintained at levels which are below the critical levels for major building damage. The sandstone bedrock underlying the proposed development site is expected to extend into adjacent properties and is likely to transmit vibrations generated by the excavation process. Consequently, it will be necessary to adopt appropriate construction methodologies and equipment to limit the vibration at adjacent buildings to acceptable levels.

If hydraulic rock hammering is required it may result in vibrations being transmitted to the surrounding ground and any buildings or structures in the vicinity. It will generally be necessary to use smaller excavation plant or alternatively methods such as rock sawing, line drilling or a milling head when in close proximity to existing structures. It is DP's experience that particular care is warranted when using hydraulic rock hammers within 10 m of adjacent structures and within 15 – 20 m of structures that are old or of heritage significance. To limit the risk of causing vibration induced damage to existing structures it is recommended that monitoring of the vibration be carried out during an initial excavation trial. If acceptable vibrations are recorded using the techniques and equipment proposed for bulk excavation then excavation could continue. If excessive vibrations occur it may be necessary to amend the excavation plan.

The propagation of vibrations at a site depends upon the plant used to carry out the excavation and the prevailing ground conditions together with the type of construction and foundation of the structures receiving the vibrations. The ground conditions such as rock strength and defects are unique to every particular site and therefore it is recommended that excavation trials be subject to vibration monitoring to establish the extent to which vibration are attenuated by the local geological conditions.

The Australian Explosives Code (AS2187.2-1993) recommends a peak particle velocity (PPV) of 10 mm per/sec for residential structures subject to blasting vibration. Ground vibrations arising from excavation plant, however, are continuous and not transient as would be blasting vibrations. Therefore, more stringent vibration limits should apply. On the basis of the above it is considered that the vibrations should be limited to a maximum PPV of 5 mm per/sec at the building line of the existing adjacent structures.

It should also be noted that humans are very sensitive to vibration, even at levels which are considered inconsequential for buildings and utilities. It may therefore be beneficial to give ample notice to local residents that excavation is to commence. It would also be prudent to undertake a dilapidation survey on all adjacent buildings so that any pre-existing damage can be identified and therefore avoid claims that the excavation has caused deterioration in adjacent structures.

6.7 Groundwater

As indicated above it is expected that the groundwater would be located many tens of metres below the existing site. It is, however, probable that some seepage will occur along the sandstone bedrock after periods of heavy rainfall. The quantity of flow should not be significant but provision will need to be made to collect any seepage flows and to dispose of this into the local stormwater drainage system.

6.8 Pavement Design

For areas within the excavation medium or high strength sandstone bedrock will be exposed at subgrade level. For areas restricted to normal passenger vehicles and light delivery vehicles up to 3 tonne it is considered that a flexible pavement thickness of 200 mm would be sufficient. Alternatively, 130 mm of concrete on a 100 mm sub-base layer would be sufficient for the same loads.

In areas where pedestrian traffic only is anticipated a 100 mm concrete pavement should be sufficient. All pavements should be placed on a subgrade compacted to 100% standard density ratio and with a moisture content suitable for achieving the compaction with the plant available.

6.9 Seismic Design

The Earthquake Loading Code (AS1170.4-1993) indicates that general structures in Sydney should be designed using an acceleration coefficient of 0.08. Furthermore, the site is underlain by shallow sandstone and Table 2.4(a) of the code dictates that a site factor of 1.0 is applicable for sites underlain by rock which is at least extremely low strength.

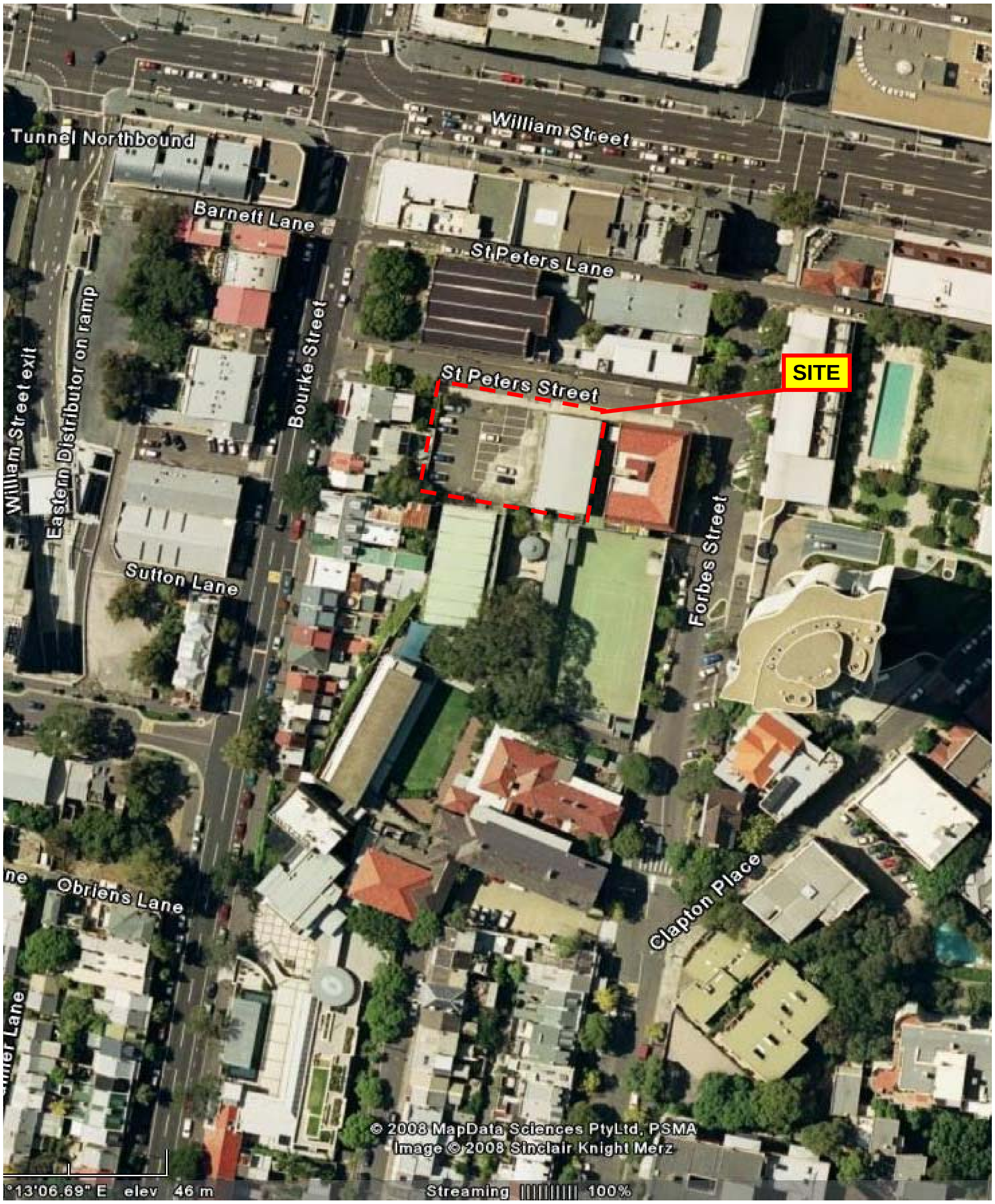
Yours faithfully
DOUGLAS PARTNERS PTY LTD

Reviewed by

Michael J Thom
Principal

Fiona MacGregor
Principal

APPENDIX A
Locality and Borehole Plans



Douglas Partners
Geotechnics · Environment · Groundwater

Sydney, Newcastle, Brisbane, Wollongong, Campbelltown

Melbourne, Perth, Wyong, Townsville, Cairns, Darwin

Title
Locality Map
New Science & Technology Building
Sydney Church of England Girls Grammar School Darlinghurst

Client: SCEGGS

Office: Sydney

Drawn by: MJT

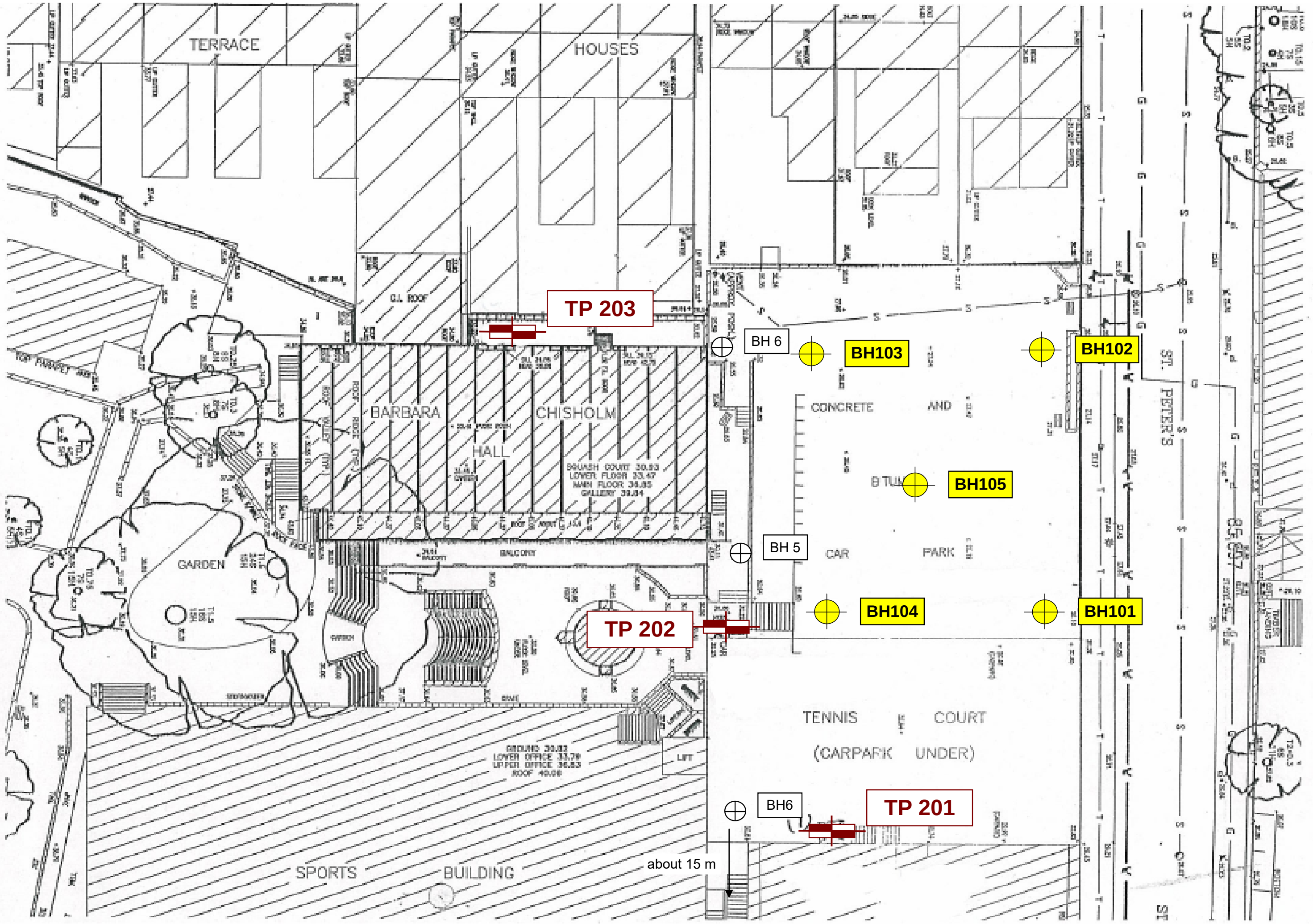
Scale: NTS

Project No: 45427

Drawing No.1

Approved by:

Date: 25 March 2008



Note: TP201 was not excavated as sandstone was above ground level

Client: SCEGGS		**Project Number: 45427** **Date: 25March 2008**	Douglas Partners Geotechnics - Environment - Groundwater	*Sydney, Newcastle, Brisbane, Wollongong* *Melbourne, Perth, Wyong, Townsville, Cairns* *Wollongong, Campbelltown, Darwin*
Drawn by: MJT	**Scale: NTS**	**Office: Sydney**		
Approved by:		**Drawing 2**	**Title**	**Bore and Test Pit Location Plan** **New Science & Technology Building** **Sydney Church of England Girls Grammar School Darlinghurst**

APPENDIX B

Test Bore and Test Pit Logs and Notes Relating to this Report

APPENDIX B1

Previous Investigations

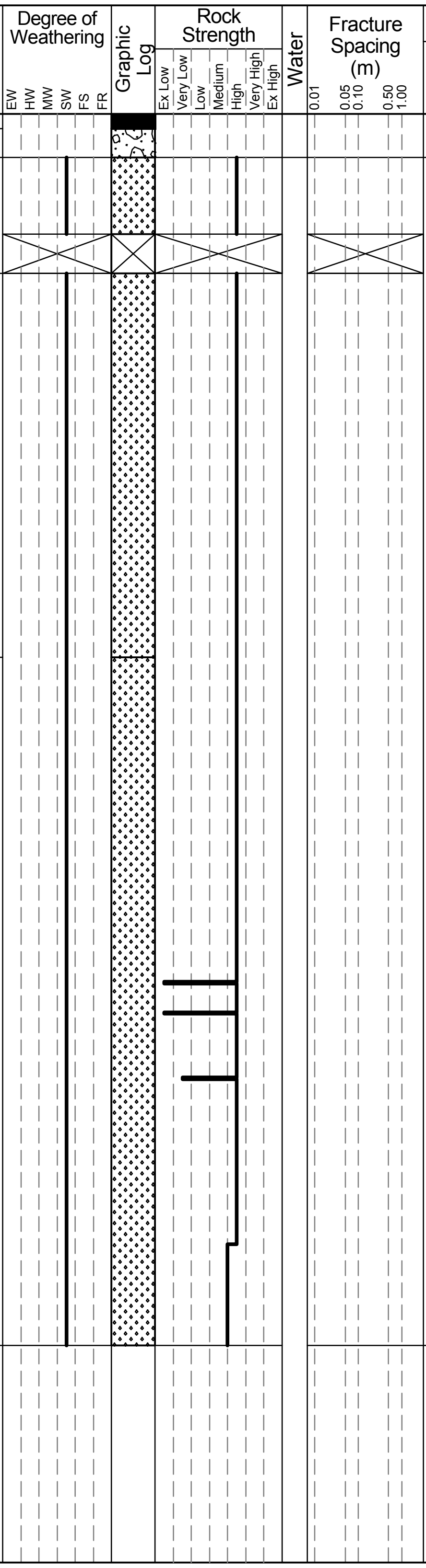
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 38.6
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 45427
DATE: 30 Jun 94
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	BITUMEN					
0.3	ROADBASE - grey gravel					
0.83	SANDSTONE - high strength, slightly weathered, slightly fractured, orange brown and brown medium grained sandstone with a possible extremely low strength band		C	90	89	PL(A) = 2.2MPa
1.1		0.83m: CORE LOSS: 270mm				
3.75	SANDSTONE - high strength, slightly weathered, slightly fractured, purple and brown medium grained sandstone - with some clay seams around 3m					PL(A) = 1.4MPa
8.5	SANDSTONE - high strength, slightly weathered, slightly fractured, light grey and brown medium grained sandstone - with some clay seams from 6m to 6.3m - medium to high strength from 7.85m		C C	100 98	98 72	PL(D) = 1.5MPa PL(A) = 1.2MPa PL(D) = 0.9MPa PL(A) = 1.5MPa PL(A) = 0.7MPa
	Bore discontinued at 8.5m					



B - Bedding J - Joint
S - Shear D - Drill Break

RIG: Scout **DRILLER:** Cooper **LOGGED:** Patel **CASING:** Uncased

TYPE OF BORING: Solid flight auger to 0.30m then HQ3 Coring

WATER OBSERVATIONS: No free groundwater observed

REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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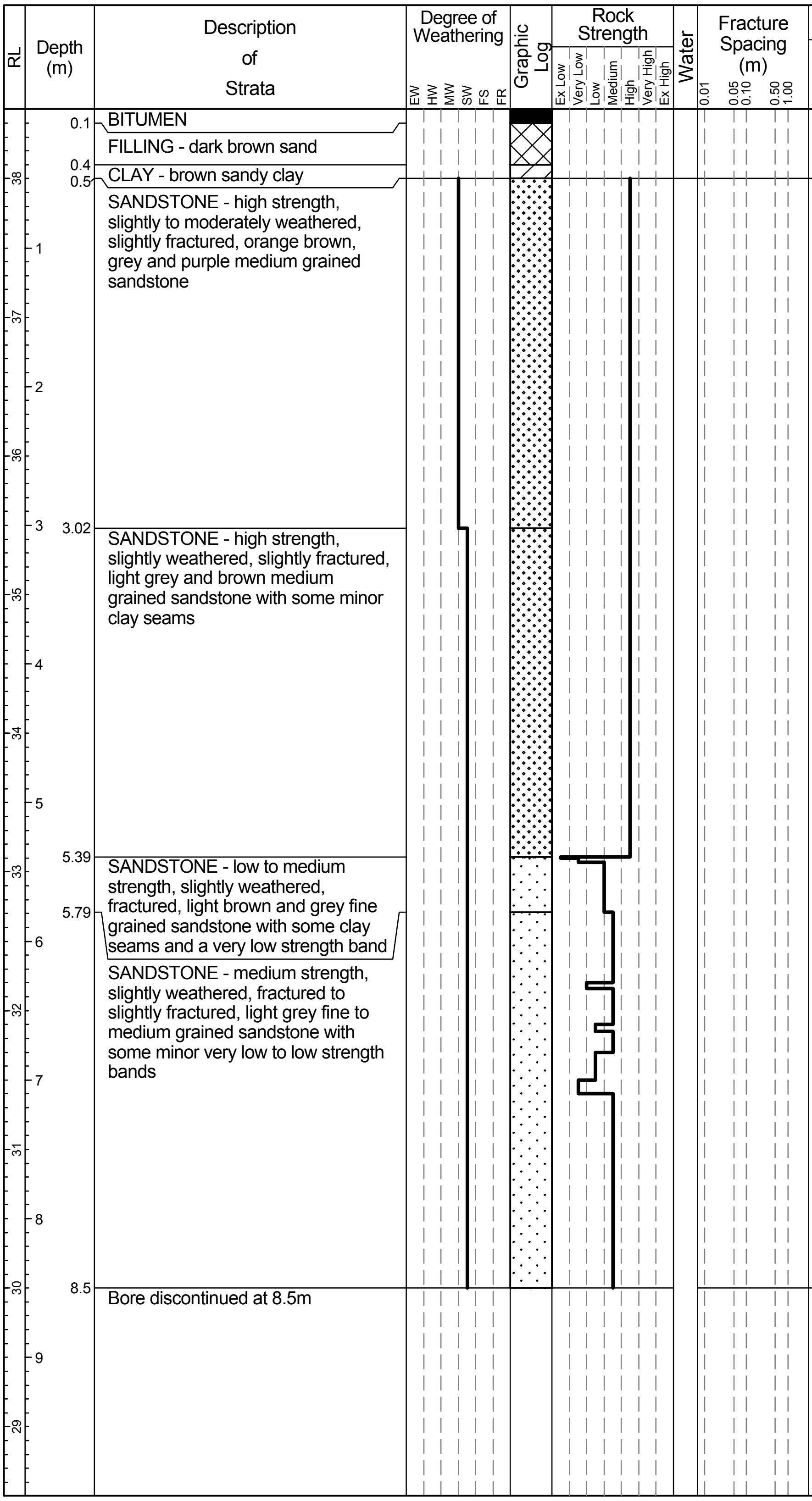
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 38.5
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 45427
DATE: 01 Jul 94
SHEET 1 OF 1

Depth (m)	Description of Strata	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	BITUMEN				
0.4	FILLING - dark brown sand				
0.5	CLAY - brown sandy clay				
3.02	SANDSTONE - high strength, slightly to moderately weathered, slightly fractured, orange brown, grey and purple medium grained sandstone	C	100	97	PL(A) = 1.3MPa PL(D) = 1.9MPa PL(A) = 1.1MPa PL(D) = 1.2MPa
5.39	SANDSTONE - high strength, slightly weathered, slightly fractured, light grey and brown medium grained sandstone with some minor clay seams	C	100	100	PL(D) = 2.1MPa PL(A) = 1.1MPa
5.79	SANDSTONE - low to medium strength, slightly weathered, fractured, light brown and grey fine grained sandstone with some clay seams and a very low strength band				PL(D) = 0.3MPa PL(A) = 0.2MPa
8.5	SANDSTONE - medium strength, slightly weathered, fractured to slightly fractured, light grey fine to medium grained sandstone with some minor very low to low strength bands	C	100	95	PL(D) = 0.9MPa PL(A) = 0.8MPa
	Bore discontinued at 8.5m				



Discontinuities: B - Bedding, J - Joint, S - Shear, D - Drill Break

RIG: Scout **DRILLER:** Cooper **LOGGED:** Patel **CASING:** HW to 0.6m
TYPE OF BORING: Solid flight auger to 0.50m then HQ3 Coring
WATER OBSERVATIONS: No free groundwater observed
REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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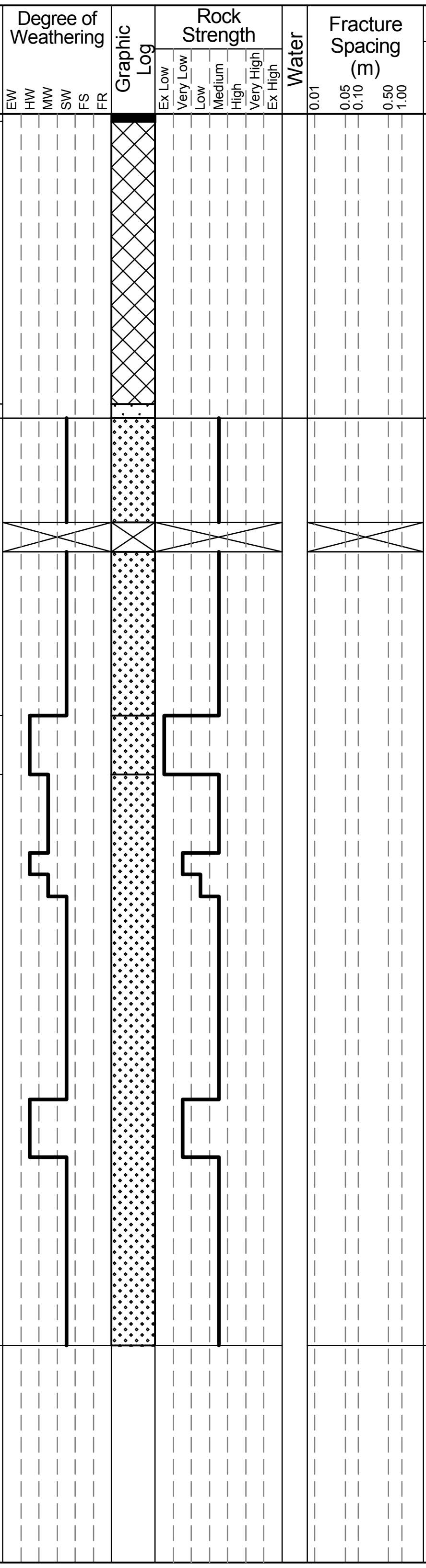
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 37.9
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 45427
DATE: 01 Jul 94
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	BITUMEN					
	FILLING - brown to dark brown sand with gravel and sandstone rubble at base					
2.0	SANDSTONE - very low strength sandstone					
2.1	SANDSTONE - medium strength, slightly weathered, fractured to slightly fractured, orange brown and light grey medium grained sandstone with a possible extremely low strength band		C	85	84	PL(A) = 0.5MPa
2.82		2.82m: CORE LOSS: 200mm				
4.15	SANDSTONE - extremely low strength, highly weathered, grey sandstone					
4.56	SANDSTONE - medium strength, slightly weathered, slightly fractured, light grey medium grained sandstone with some very low strength bands and some clay seams above 7.2m		C	100	73	PL(A) = 0.3MPa PL(A) = 1MPa
			C	100	81	PL(A) = 0.8MPa PL(A) = 0.8MPa
8.5	Bore discontinued at 8.5m					

RIG: Scout **DRILLER:** Cooper **LOGGED:** Patel **CASING:** HW to 0.5m
TYPE OF BORING: Solid flight auger to 0.50m then HQ3 Coring
WATER OBSERVATIONS: No free groundwater observed
REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 33.2
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 45427
DATE: 26 Jun 94
SHEET 1 OF 1

Degree of Weathering: EW HW MW SW FS FR
Rock Strength: Ex Low, Very Low, Low, Medium, High, Very High, Ex High
Fracture Spacing (m): 0.01 0.05 0.10 0.50 1.00
Discontinuities: B - Bedding J - Joint S - Shear D - Drill Break

RL	Depth (m)	Description of Strata	Graphic Log	Water	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
33	0.25	FILLING - topsoil and sandstone rubble							
		SANDSTONE - medium strength, slightly weathered, fractured, grey, medium grained sandstone with some slighty fractured lengths and a minor extremely low strength band							PL(D) = 0.6MPa
	1					C	100	70	
32									
									PL(D) = 0.4MPa
	2					C	100	75	
31									
									PL(D) = 0.9MPa
	3					C	100	70	
30									
		- high strength from 3.2m							PL(D) = 1.5MPa
	3.5	Bore discontinued at 3.5m							
	4								
29									
	5								
28									
	6								
27									
	7								
26									
	8								
25									
	9								
24									

RIG: Portable **DRILLER:** Chittleburgh **LOGGED:** Patel **CASING:** NW to 0.5m
TYPE OF BORING: Hand auger to 0.25m then NMLC Coring
WATER OBSERVATIONS: No free groundwater observed
REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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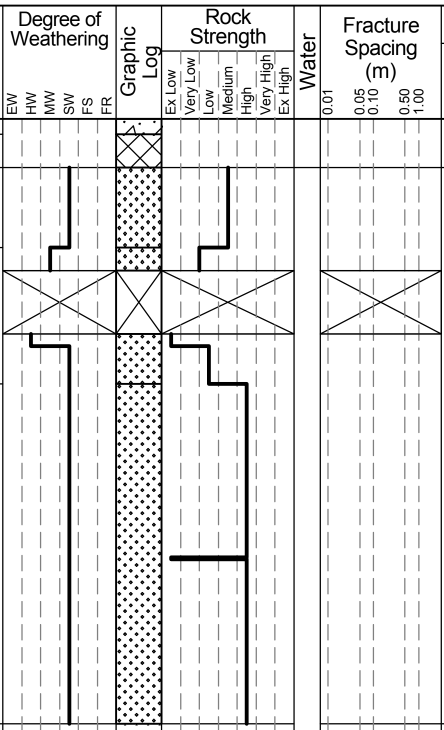
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 30.3
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 45427
DATE: 28 Jun 94
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	CONCRETE					
0.32	FILLING - sandstone rubble and sandy clay					
0.85 1.0	SANDSTONE - medium strength, slightly weathered, fractured to slightly fractured, grey medium grained sandstone		C	78	42	PL(D) = 0.4MPa
	SANDSTONE - extremely low to very low strength, moderately weathered, grey fine to medium to coarse grained sandstone	1m: CORE LOSS: 420mm				
1.75	SANDSTONE - high strength, slightly weathered, fractured, grey medium to coarse grained sandstone with some clay bands to 20mm above 3m					PL(D) = 1.4MPa
			C	100	17	PL(D) = 0.7MPa
			C	100	88	PL(D) = 1.7MPa
4.0	Bore discontinued at 4.0m					

RIG: Portable **DRILLER:** Chittleburgh **LOGGED:** Patel **CASING:** NW to 0.5m
TYPE OF BORING: Hand auger to 0.32m then NMLC Coring
WATER OBSERVATIONS: No free groundwater observed
REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 30.1
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 45427
DATE: 29 Jun 94
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Water	Fracture Spacing (m) (0.01 0.05 0.10 0.50 1.00)	Discontinuities (B - Bedding J - Joint S - Shear D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
30		FILLING - topsoil		☒								
	0.1	FILLING - sandy clay and sandstone rubble		☒								
29	0.9	SANDSTONE - high strength, slightly weathered, fractured to slightly fractured, grey and brown medium grained sandstone	SW	⠿	High				C	100	89	PL(D) = 1.6MPa PL(D) = 1.7MPa
28	2.0	Bore discontinued at 2.0m										

RIG: Portable **DRILLER:** Chittleburgh **LOGGED:** Patel **CASING:** NW to 0.9m
TYPE OF BORING: Hand auger to 0.9m then NMLC Coring
WATER OBSERVATIONS: No free groundwater observed
REMARKS: These bores are a summary of the original bore logs

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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APPENDIX B2

Current Investigation

BOREHOLE LOG

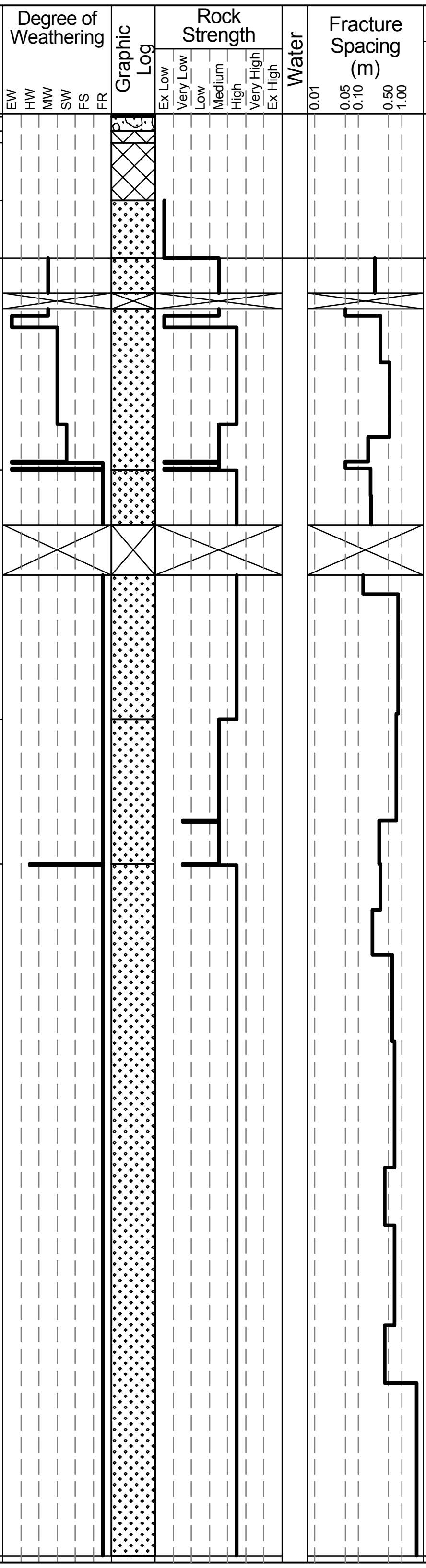
CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 28.15
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 45427
DATE: 05 Mar 08
SHEET 1 OF 1

Depth (m)	Description of Strata
0.02	BITUMINOUS CONCRETE
0.12	ROADBASE GRAVEL
0.2	SLAG FILLING
0.6	FILLING - light grey brown medium grained, sand filling with some gravel
1.0	SANDSTONE - extremely low strength, light grey brown, medium grained sandstone
1.24	SANDSTONE - medium and high strength, moderately weathered and fresh, slightly fractured, light grey and brown medium to coarse grained sandstone
2.47	SANDSTONE - high strength, fresh, slightly fractured, light grey medium to coarse grained sandstone
2.85	
4.2	SANDSTONE - medium strength, fresh, slightly fractured, light grey medium grained sandstone
5.2	SANDSTONE - high strength, fresh, slightly fractured then unbroken, light grey medium to coarse grained sandstone
10.0	

Bore discontinued at 10.0m



Discontinuities

B - Bedding J - Joint
S - Shear D - Drill Break

Note: Unless otherwise stated, rock is fractured along rough, clay veneered, planar bedding or joints dipping 0°- 10°

1.24m: CORE LOSS: 110mm

2.41m: B5°, 5mm

2.85m: CORE LOSS: 350mm

3.2m: J40°

3.46m: B0°, 2mm clay

4.16m: J30°, ironstained

4.9m: B0°, 5mm clayey sand

5.2m: B0°, 3mm clayey sand

5.52m: J30°, ironstained

Sampling & In Situ Testing

Type	Core Rec. %	RQD %
A		
A		
C	69	69
C	100	87
C	88	87
C	100	99
C	100	100

Test Results & Comments:

PL(A) = 1.3MPa

PL(A) = 0.8MPa

PL(A) = 1.7MPa

PL(A) = 0.7MPa

PL(A) = 1.4MPa

PL(A) = 2MPa

PL(A) = 1.9MPa

PL(A) = 1.7MPa

PL(A) = 1.4MPa

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** SI **CASING:** HW to 1.0m
TYPE OF BORING: Solid flight auger to 1.0m; NMLC-Coring to 10.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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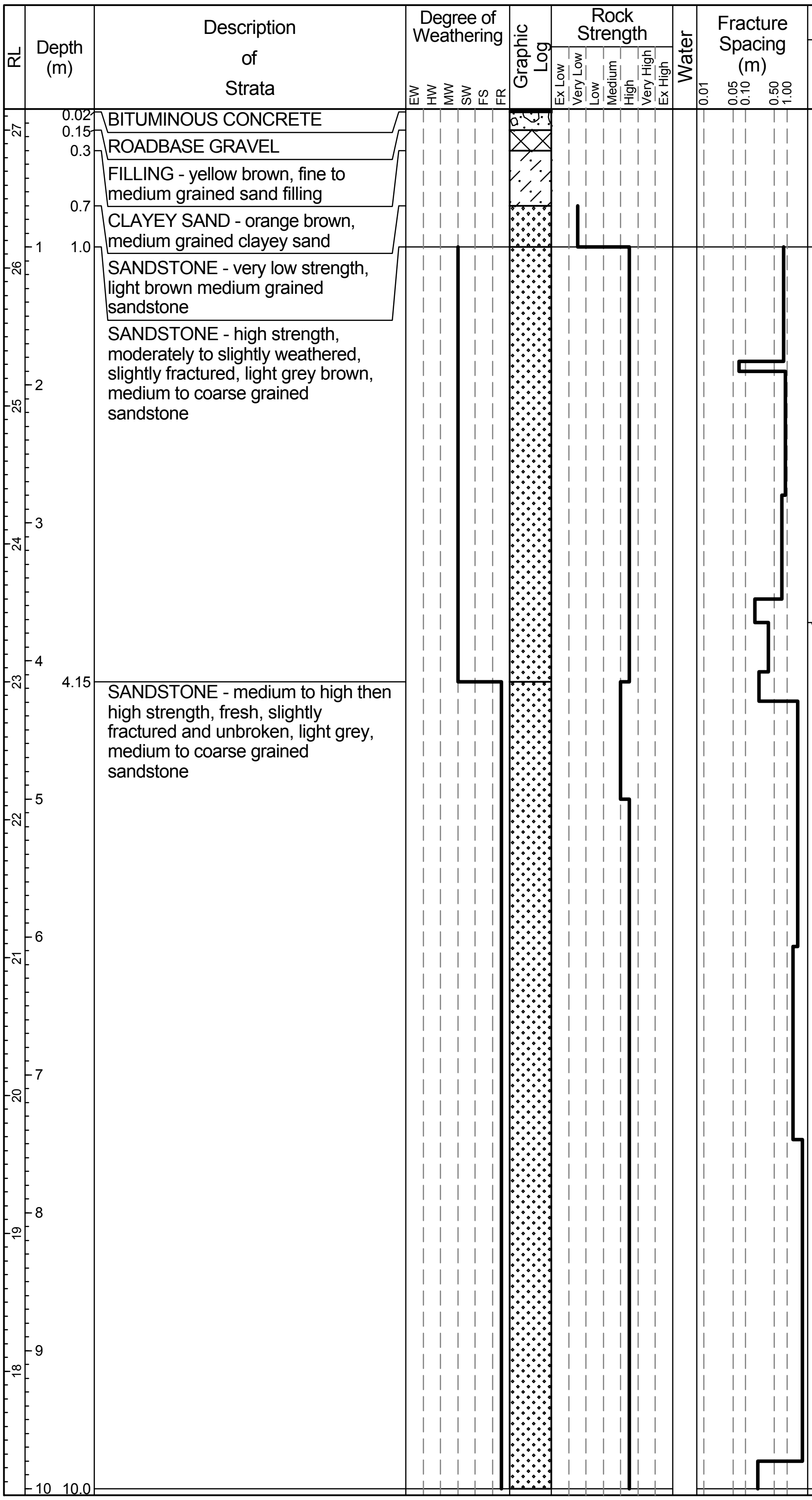
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 27.15
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 45427
DATE: 3-4/3/08
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear D - Drill Break	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	BITUMINOUS CONCRETE	Note: Unless otherwise stated, rock is fractured along rough planar bedding or joints dipping 0°- 10°	A			
0.15	ROADBASE GRAVEL					
0.3	FILLING - yellow brown, fine to medium grained sand filling		A			
0.7	CLAYEY SAND - orange brown, medium grained clayey sand					
1.0	SANDSTONE - very low strength, light brown medium grained sandstone		C	100	100	PL(A) = 1.5MPa
	SANDSTONE - high strength, moderately to slightly weathered, slightly fractured, light grey brown, medium to coarse grained sandstone	3.55m: B15°, 2mm sandy clay 3.72m: B5°, ironstained	C	100	98	PL(A) = 1.7MPa PL(A) = 1.7MPa
4.15	SANDSTONE - medium to high then high strength, fresh, slightly fractured and unbroken, light grey, medium to coarse grained sandstone					PL(A) = 1MPa
		6.07m: B0°, clay smear 7.47m: B15°, clay veneer	C	100	100	PL(A) = 1.7MPa PL(A) = 2.1MPa PL(A) = 2.7MPa
		9.8m: B0°, clayey	C	100	100	PL(A) = 1.7MPa PL(A) = 1.8MPa
10.0	Bore discontinued at 10.0m					



RIG: Bobcat **DRILLER:** E Grima **LOGGED:** SI **CASING:** HW to 1.0m
TYPE OF BORING: Solid flight auger to 1.0m; NMLC-Coring to 10.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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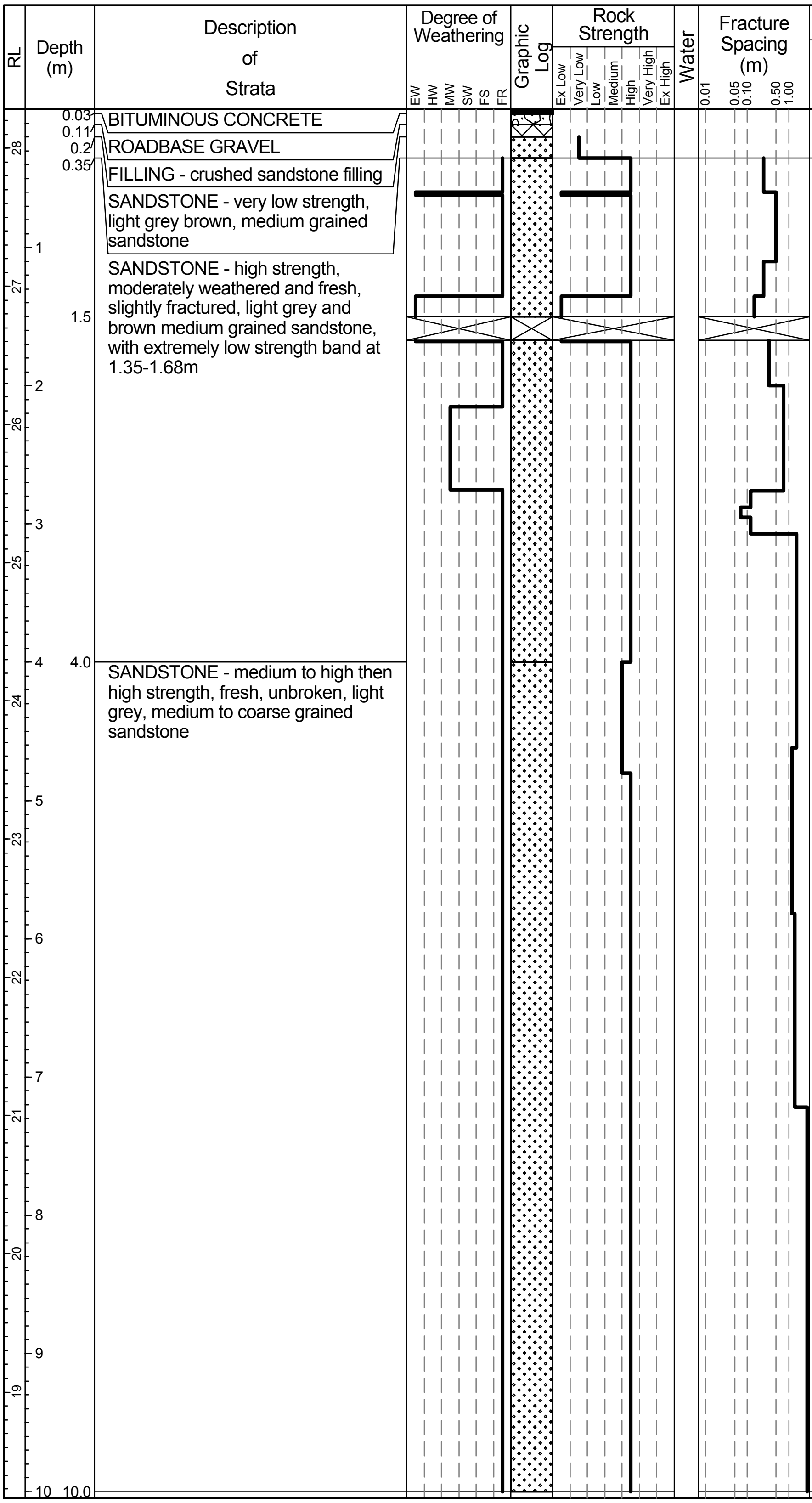
BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 28.28
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 45427
DATE: 04 Mar 08
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Type	Core Rec. %	RQD %	Test Results & Comments
0.03	BITUMINOUS CONCRETE		A			
0.11 0.2	ROADBASE GRAVEL		A			
0.35	FILLING - crushed sandstone filling					
	SANDSTONE - very low strength, light grey brown, medium grained sandstone	0.6m: B0°, 15mm clay	C	100	97	PL(A) = 1.5MPa
	SANDSTONE - high strength, moderately weathered and fresh, slightly fractured, light grey and brown medium grained sandstone, with extremely low strength band at 1.35-1.68m	1.10-1.35m: J85°				
1.5		1.5m: CORE LOSS: 170mm	C	83	67	PL(A) = 1.9MPa
		Note: Unless otherwise stated, rock is fractured along rough planar bedding planes or joints dipping 0°- 10°	C	100	100	PL(A) = 2.4MPa
						PL(A) = 2MPa
			C	100	97	
4.0	SANDSTONE - medium to high then high strength, fresh, unbroken, light grey, medium to coarse grained sandstone					PL(A) = 1MPa
						PL(A) = 1.7MPa
			C	100	100	PL(A) = 2.8MPa
						PL(A) = 2MPa
						PL(A) = 2.1MPa
			C	100	100	
						PL(A) = 1.7MPa
10.0	Bore discontinued at 10.0m					

RIG: Bobcat **DRILLER:** E Grima **LOGGED:** SI **CASING:** HW to 3.5m

TYPE OF BORING: Solid flight auger to 0.35m; NMLC-Coring to 10.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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BOREHOLE LOG

CLIENT:	SCEGGS Darlinghurst	**SURFACE LEVEL:** --	**BORE No:** 104
PROJECT:	Joan Freeman Building	**EASTING:**	**PROJECT No:** 45427
LOCATION:	St Peters Street, Darlinghurst	**NORTHING:**	**DATE:** 03 Mar 08
		DIP/AZIMUTH: 90°/--	**SHEET** 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Well Construction Details
				Type	Depth	Sample	Results & Comments		
		CONCRETE AND BRICK							
	0.17	FILLING - brown silty sand filling, some sandstone gravel		A	0.17 0.2				
	0.3	SANDSTONE - highly weathered, very low strength, white sandstone		A	0.4 0.5				
	0.5	Bore discontinued at 0.5m - refusal on sandstone							
	1								1

RIG: Bobcat **DRILLER:** E Grima/DT **LOGGED:** AHP **CASING:** Uncased

TYPE OF BORING: Concrete core 120mm solid filght auger

WATER OBSERVATIONS: No free groundwater obsesrved

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

Douglas Partners
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BOREHOLE LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: --
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 45427
DATE: 03 Mar 08
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Well Construction Details
				Type	Depth	Sample	Results & Comments		
		ASPHALT	■		0.02				
	0.03	FILLING - dark brown fine gravel filling - strong hydrocarbon odour at 0.2m - brick at 0.15m	▨	A	0.1				
	0.2	FILLING - dark grey brown silty snd filling, fine gravel, strong hydrocarbon odour	▨		0.2		hydrocarbon odour at 0.2m		
			▨	A	0.4				
			▨	A	0.5				
	0.6	FILLING - light to dark grey brown silty clayey sand, strong hydrocarbon odour and some fine gravel	▨		0.6				
			▨	A	0.8				
	0.95	SANDSTONE	⁘	A	0.95				
	1		⁘						1
	1.05	Bore discontinued at 1.05m - refusal on sandstone			1.05				

RIG: Bobcat **DRILLER:** E Grima/DT **LOGGED:** AHP **CASING:** Uncased
TYPE OF BORING: Solid flight auger
WATER OBSERVATIONS: No free groundwater obsesrved
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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TEST PIT LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 30.2
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

PIT No: 202
PROJECT No: 45427
DATE: 28 Mar 08
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Dynamic Penetrometer Test (blows per mm) 5 10 15 20
				Type	Depth	Sample	Results & Comments		
30		FILLING - brown sandy filling, with some sandstone gravel	▨						
	0.6	Pit discontinued at 0.6m - refusal on sandstone							

RIG: Hand tools

LOGGED: F Volbrecht

WATER OBSERVATIONS: No free groundwater observed whilst excavating

REMARKS: Surface level interpreted from Tanner Architects site plan

☐ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ▼ Water level

CHECKED
Initials:
Date:

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TEST PIT LOG

CLIENT: SCEGGS Darlinghurst
PROJECT: Joan Freeman Building
LOCATION: St Peters Street, Darlinghurst

SURFACE LEVEL: 32.0
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/--

PIT No: 203
PROJECT No: 45427
DATE: 28 Apr 08
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing				Water	Dynamic Penetrometer Test (blows per mm) 5 10 15 20
				Type	Depth	Sample	Results & Comments		
32		CONCRETE							
	0.1	FILLING - brown sandy filling, with some sandstone gravel and brick fragments							
	0.7	SANDSTONE - very low to low strength, red brown sandstone							
	0.75	SANDSTONE - low to medium strength, light grey sandstone							
	0.8	Pit discontinued at 0.8m - limit of investigation							
31	1								1
30	2								2
29	3								3
28	4								4

RIG: Hand tools

LOGGED: B O'Kane

WATER OBSERVATIONS: No free groundwater observed whilst excavating

REMARKS: Surface level interpreted from Tanner Architects site plan

☐ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	PID	Photo ionisation detector
B	Bulk sample	S	Standard penetration test
U_x	Tube sample (x mm dia.)	PL	Point load strength Is(50) MPa
W	Water sample	V	Shear Vane (kPa)
C	Core drilling	▷	Water seep ⊻ Water level

CHECKED
Initials:
Date:

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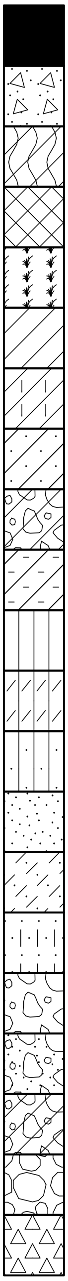
Log!GRAPHIC-SYMBOLS 24/11/2003 4:38:57 PM

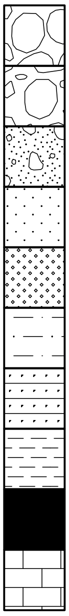
GRAPHIC SYMBOLS FOR SOIL & ROCK

SOIL



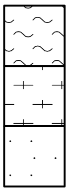
BITUMINOUS CONCRETE
CONCRETE
TOPSOIL
FILLING
PEAT
CLAY
SILTY CLAY
SANDY CLAY
GRAVELLY CLAY
SHALY CLAY
SILT
CLAYEY SILT
SANDY SILT
SAND
CLAYEY SAND
SILTY SAND
GRAVEL
SANDY GRAVEL
CLAYEY GRAVEL
COBBLES/BOULDERS
TALUS

SEDIMENTARY ROCK



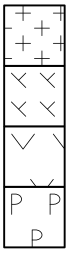
BOULDER CONGLOMERATE
CONGLOMERATE
CONGLOMERATIC SANDSTONE
SANDSTONE FINE GRAINED
SANDSTONE COARSE GRAINED
SILTSTONE
LAMINITE
MUDSTONE, CLAYSTONE, SHALE
COAL
LIMESTONE

METAMORPHIC ROCK



SLATE, PHYLITTE, SCHIST
GNEISS
QUARTZITE

IGNEOUS ROCK



GRANITE
DOLERITE, BASALT
TUFF
PORPHYRY

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring 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.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\, c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 “Methods of Testing Soil for Engineering Purposes”. Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein 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. 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 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, the frequency of sampling and the possibility of other than ‘straight line’ variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it 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.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations 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.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company 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 condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

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

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, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document “Guidelines for the Provision of Geotechnical Information in Tender Documents”, published by the Institution of Engineers, Australia. Where information obtained from this investigation 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. The Company 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 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.

AN ENGINEERING CLASSIFICATION OF SEDIMENTARY ROCKS IN THE SYDNEY AREA

This classification system provides a standardized terminology for the engineering description of the sandstone and shales in the Sydney area, but the terms and definitions may be used elsewhere when applicable.

Under this system rocks are classified by Rock Type, Degree of Weathering, Strength, Stratification Spacing, and Degree of Fracturing. These terms do not cover the full range of engineering properties. Descriptions of rock may also need to refer to other properties (e.g. durability, abrasiveness, etc.) where these are relevant.

ROCK TYPE DEFINITIONS

Rock Type	Definition
Conglomerate:	More than 50% of the rock consists of gravel sized (greater than 2mm) fragments
Sandstone:	More than 50% of the rock consists of sand sized (.06 to 2mm) fragments
Siltstone:	More than 50% of the rock consists of silt-sized (less than 0.06mm) granular particles and the rock is not laminated
Claystone:	More than 50% of the rock consists of clay or sericitic material and the rock is not laminated
Shale:	More than 50% of the rock consists of silt or clay sized particles and the rock is laminated

Rocks possessing characteristics of two groups are described by their predominant particle size with reference also to the minor constituents, e.g. clayey sandstone, sandy shale.

DEGREE OF WEATHERING

Term	Symbol	Definition
Extremely Weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly Weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decreased compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original fresh rock substance is no longer recognisable.
Moderately Weathered	MW	Rock substance affected by weathering to the extent that staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is no longer recognisable.
Slightly Weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh	Fs	Rock substance unaffected by weathering, limonite staining along joints.
Fresh	Fr	Rock substance unaffected by weathering.

STRATIFICATION SPACING

Term	Separation of Stratification Planes
Thinly laminated	<6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	>2 m

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Society of Rock Mechanics (Reference).

Strength Term	Is(50) MPa	Field Guide	Approx. qu MPa*
Extremely Low:	0.03	Easily remoulded by hand to a material with soil properties	0.7
Very Low:	0.1	May be crumbled in the hand. Sandstone is “sugary” and friable.	2.4
Low:	0.3	A piece of core 150 mm long x 50 mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	7
Medium:	1	A piece of core 150 mm long x 50 mm dia. can be broken by hand with considerable difficulty. Readily scored with knife.	24
High:	3	A piece of core 150 mm long x 50 mm dia. cannot be broken by unaided hands, can be slightly scratched or scored with knife.	70
Very High:	10	A piece of core 150 mm long x 50 mm dia. may be broken readily with hand held hammer. Cannot be scratched with pen knife.	240
Extremely High:		A piece of core 150 mm long x 50 mm dia. is difficult to break with hand held hammer. Rings when struck with a hammer.	

* The approximate unconfined compressive strength (qu) shownin the table is based on an assumed ratio to the point load index of 24:1. This ratio may vary widely.

DEGREE OF FRACTURING

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude known artificial fractures such as drilling breaks

Term	Description
Fragmented:	The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than the core diameter.
Highly Fractured:	Core lengths are generally less than 20 mm - 40 mm with occasional fragments.
Fractured:	Core lengths are mainly 30 mm - 100 mm with occasional shorter and longer sections.
Slightly Fractured:	Core lengths are generally 300 mm - 1000 mm with occasional longer sections and occasional sections of 100 mm - 300 mm.
Unbroken:	The core does not contain any fracture.

REFERENCE

International Society of Rock Mechanics, Commission on Standardisation of Laboratory and Field Tests, Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index, Committee on Laboratory Tests Document No. 1 Final Draft October 1972

Appendix 3

FRANKI

SECTION 28

VERMEER HL 2500 RIG


FRANKI
FRANKI
VERMEER MT HL-2500
FM12


Genie Z-60/3
WINCH
FRANKI
1CNE·232


FRANKI
FM12


5 1:09 PM


5 1:09PM

VERMEER HL 2500 PILING MACHINE
PLANT NOS. FM 12
C.F.A. – BORED – FORUM - DRIVEN TUBES.
VERMEER HL 2500 - PLANT NO. FM 12
TECHNICAL SPECIFICATIONS
VERMEER SERIAL NO. - YEAR BUILT 2007

ROTARY DRIVE CAPACITY

MAX. TORQUE	**6 T/M**
DRILL SPEED	**0 – 34 RPM**

MACHINE MAXIMUM AND MINIMUM HEIGHTS

MINIMUM HEIGHT FOR HAMMER WORK	**3.3 METRES**
MAXIMUM HEIGHT	**10.8 METRES**

MAST SECTION LENGTHS : 2740 & 2500 & 1500 & 1500 + CATHEAD 400 MM

DRILLING DATA INFORMATION

MAXIMUM PULL OUT IN CFA MODE IS 9 TONS

MAXIMUM PULL OUT BORED PILE IS 3 TONS

	BORED PILE	C.F.A.
PILE DIAMETERS	900 MM	700 MM
DRILLING DEPTH	9 M	9.5 M
KELLY 3 X 3.7 WEIGHT 880 KG.		
KELLY DRIVE	130 x 130	STD HEX

CROWD SYSTEM RACK AND PINION TYPE

STROKE	**5000 MM**
CROWD AND EXTRACTION FORCE	**4.5 TON**

MAIN WINCH (FREE FALL)

MAX. LINE PULL	**3 TON**
LINE SPEED	**40 - 80 M/MIN**
ROPE DIA	**16 MM**

AUXILIARY WINCH

MAX. LINE PULL	**1.5 TON**
LINE SPEED	**30 M/MIN**
ROPE DIA	**14 MM**

CRAWLER BASE

MAKE	**VERMEER HL2500**
ENGINE	**DEUTZ BF 4M 2011**
POWER	**60 kW**
GROUND BEARING PRESSURE	**70 kPa**

OPERATING WEIGHT

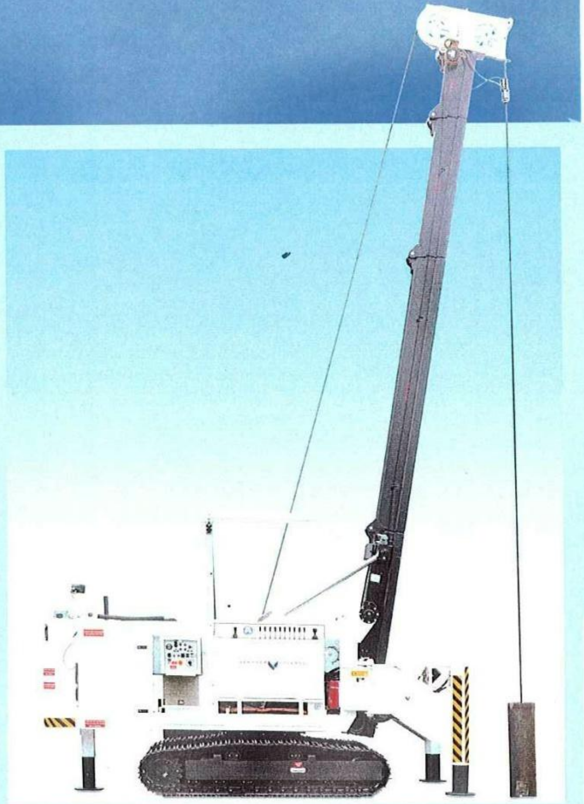
C.F.A. OR BORED PILES	**8 TON**
TUBE OR FORUM PILES	**8 TON**

SECTION 31

VERMEER
MACHINE TECHNIEK

Piling machine

HL-2500

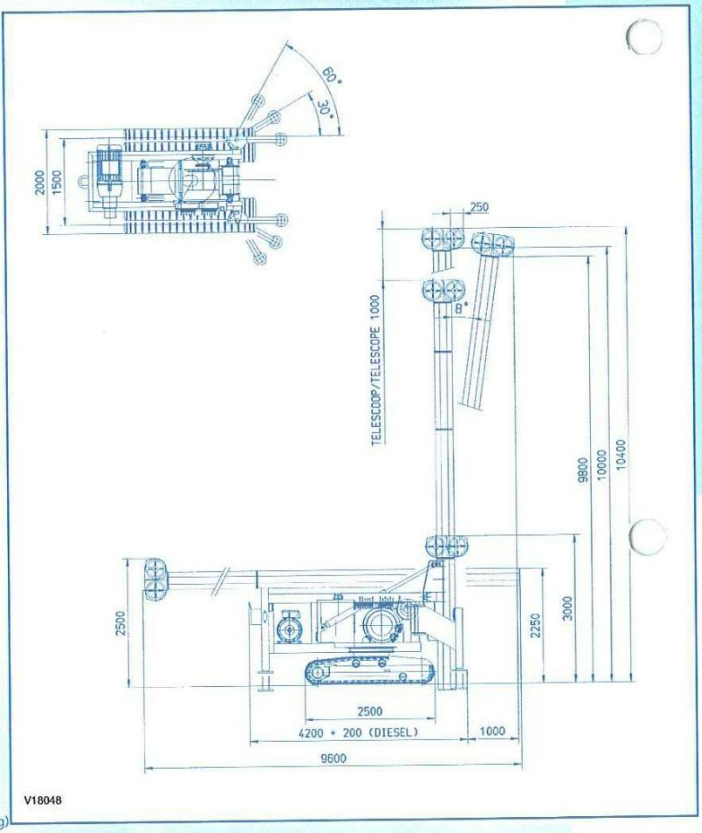


Model HL-2500

TECHNICAL SPECIFICATIONS

Vermeer Machine Techniek piling machines are suitable for various foundation installation techniques. In audition to piling act a free fall drop hammer internally or top drive, casings and augers can be used for piling methods, including Bovi and Vibro. VermeerMT piling machines are manufactured to the highest quality standards. The fully automatic computerised piling system provides a continuous, high frequency free fall resulting in excellent performance. VermeerMT piling machines are available in seven models using 750, 1200, 1800, 2500 and 4000 kgs. weight drop hammer. The machines are of modular construction, allowing some requested specification changes e.g. track width, 360° slewing, and different mast types.

Undercarriages	
Track width (std)	2000 mm
Track length	2500 mm
Track plate width	400 mm
Track plate material	steel
Torque	2 x 5,5 kNm
Main winch (free-fall)	
Drop hammer weight (max./ min)	2500/500 kg
Hoisting speed	80 m/min
Control (piling)	fully automatic
Control (hoisting)	hydr. proportional
Rope diameter/length	16 mm/40 m
Service winch	
Pull (max)	15 kN
Hoisting speed	30 m/min
Hoisting control	hydr. proportional
Rope diameter/length	14 mm/25 m
Mast	square tube with guides
Movement of mast	hydraulic 8° forward, 90° backward
Transport height	2500 mm (ex. sheave head 2250 mm)
Max./min. working height	10400/3000 mm (incl. sheave head)
Hydraulic mast extension	1000 mm
Power unit	
Diesel engine	45kW (24 Volt)
Electric motor (optional)	45 kW - 400/690 V
Stabilisers	
Position	1 behind - 2 turnable in front
Control	hydraulic
Weight of standard machine	7700 kg (without drop hammer)



Options

- 360° slewing ring
- Expanding tracks (1500 - 2000 mm)
- Additional service winch
- Mast positioning frame (forward/backward, tilting, telescoping)
- Hydraulic power outlet (max. 150 ltr. by 175 bar)
- Mast extension with hydr. clamp system
- Auger drill head
- Electric socket (for welding or small electric tools)
- 2 work lights 24 V/70 W
- Painting to customers requirements
- Protection shield/vandal shield for electric control box

Your dealer:

VERMEER
MACHINE TECHNIEK

Graftermeerstraat 29-33
2131 AA Hoofddorp
The Netherlands

P.O. Box 20
2130 AA Hoofddorp
The Netherlands

Telephone: +31 (0)23 - 55 42 300
Fax: +31 (0)23 - 56 37 133

E-mail: info@VermeerMT.nl
Internet: www.VermeerMT.nl

Vermeer-Holland
Piling machine Type HL-2500

VERMEER
MACHINE TECHNIEK

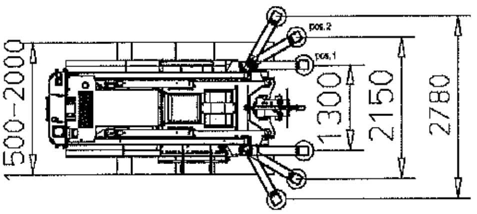
Rotary Table SB-65

Advantages of Displacement Drilling:

- No noise
- No vibrations
- High friction of the pile due to displacement. This results in smaller required diameter casing with higher bearing capacity

CAPACITY CHART PILING MACHINE

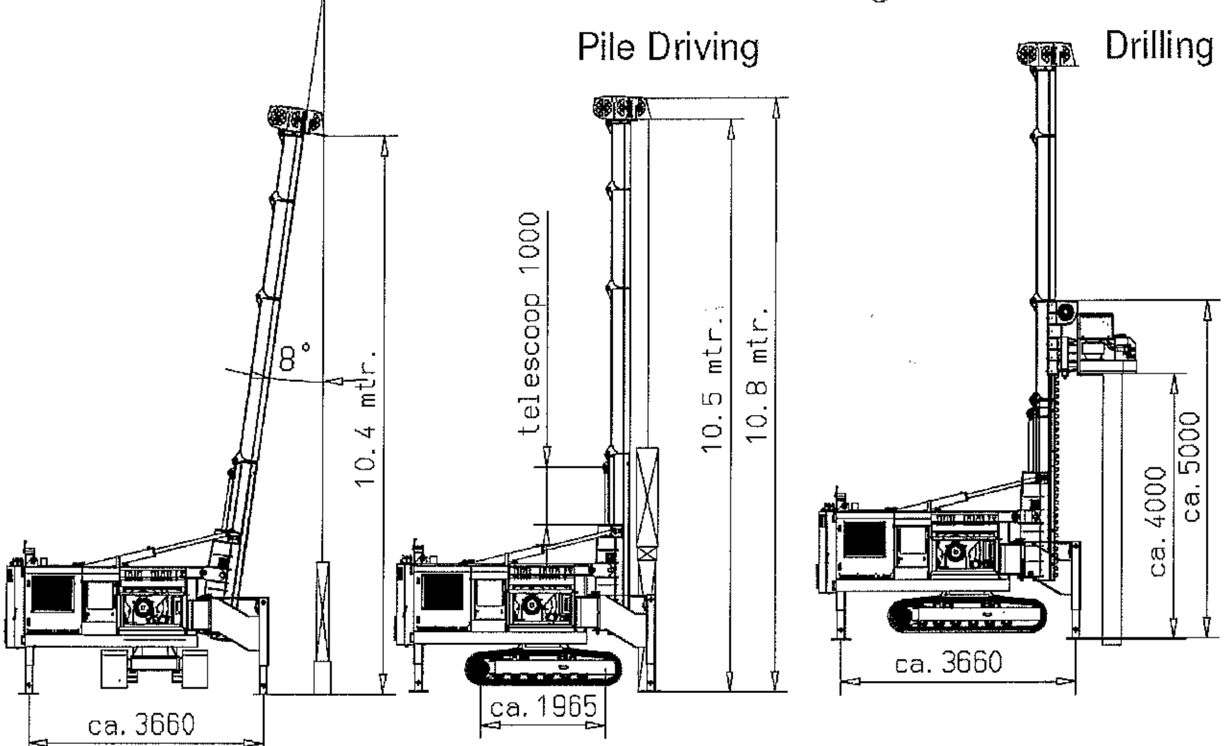
Builder : VermeerMT (The Netherlands)
Type : HL-2500
Serial number : 07.2201
Year of construction : 2007



"Free Fall" Piling Inside Casing

Pile Driving

Drilling

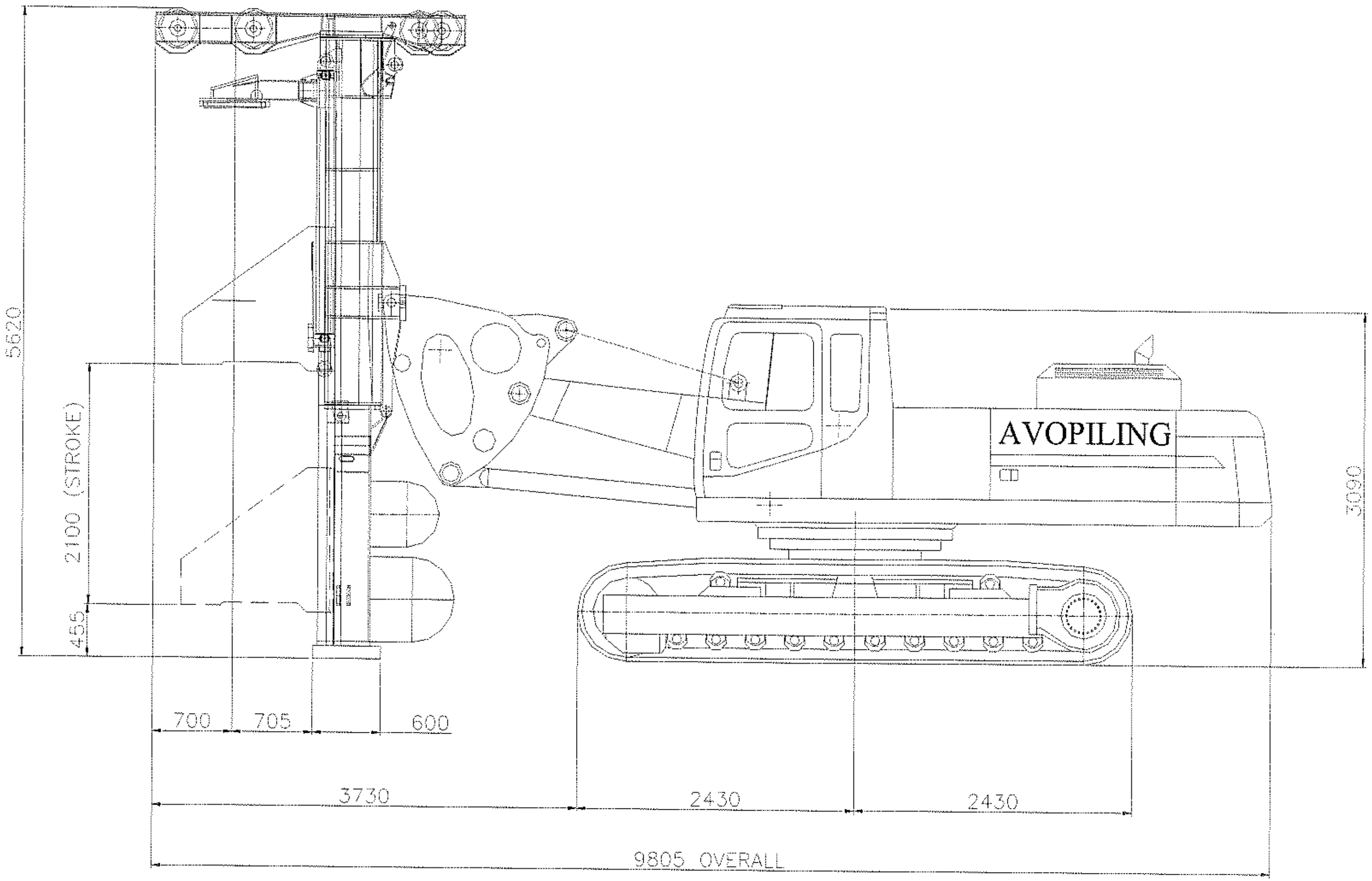


All mentioned capacities are based on ramming with a drop-weight.

Admissible mass of the drop-weight (inside casing)	: 3000	kg.
Admissible mass of the drop-weight (pile driving)	: 2600	kg.
Admissible weight of cap	: 400	kg.
Admissible mass of the pile or casing	: 1500	kg.
Maximum torque	: 65	kNm (max. height 5m.)
Maximum length of leader	: 10,8	m.
Mass leader	: 2200	kg.
Mass of sheavehead	: 130	kg.
Mass of contra-ballast (baseframe under engine)	: --	kg.
Rotation upperstructur	: 360	°
Admissable cable-pull main winch	: 30	kN.
first auxiliary winch	: 15	kN.
second auxiliary winch	: 15	kN
Admissible maximum wind-force	: 20	m/sec.
Groundpressure machine on jacks	0,5	N/mm,
Groundpressure machine on tracks	0,065	N/mm,

The capacity data are based on the directives of the NVAF
(Dutch Association of Piling and Foundation Contractors).
The machine complies with EN 996. The stability complies with NEN 2022.
Crane Directory NEN 2018:D-6.

V36512



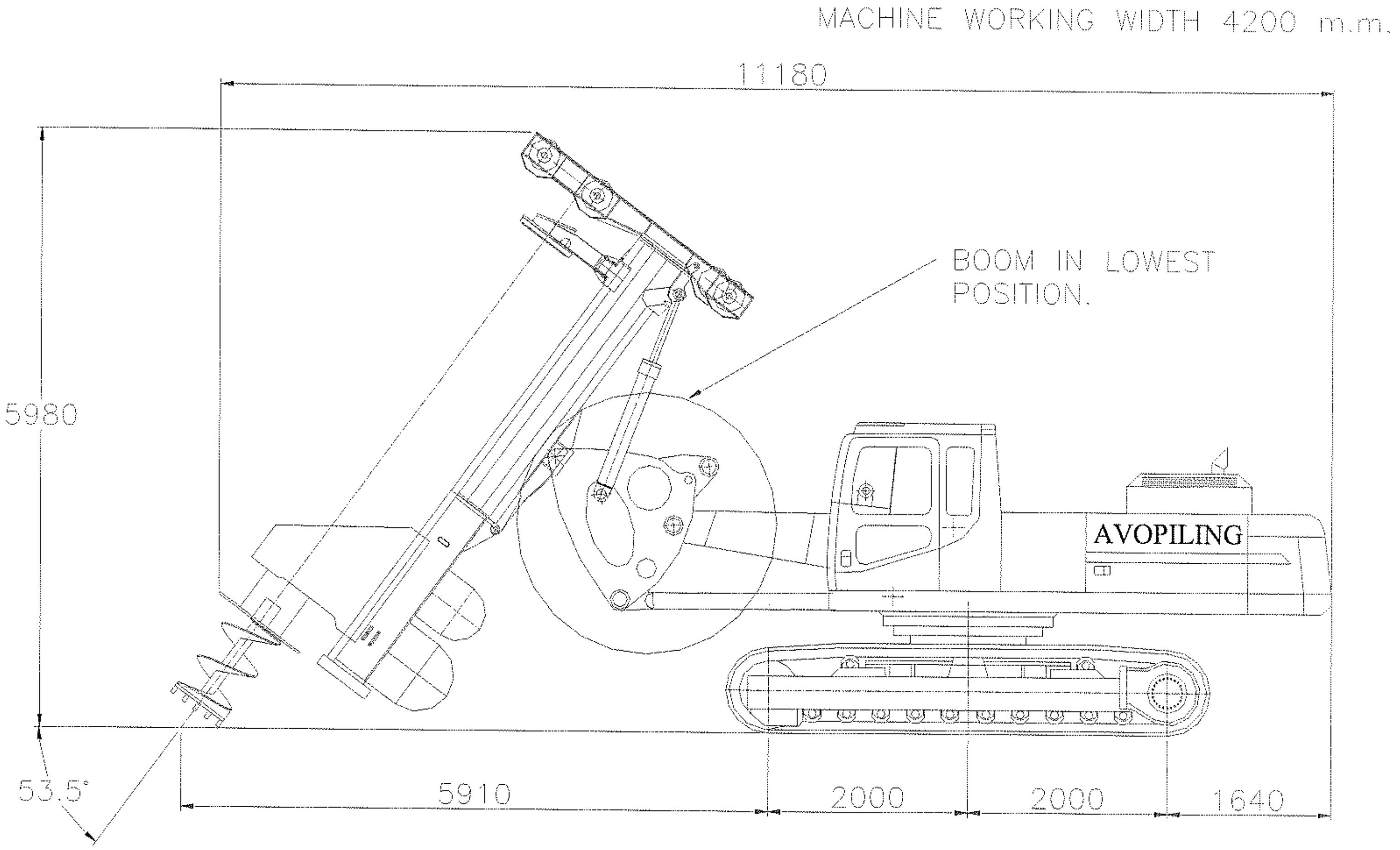
METRIC – DO NOT SCALE
DRAFTING STANDARD
AS 1100

SYSTEM ; 515 SHORT MAST
DESC ; GENERAL DIMENSIONS

DRAWN ; T. SHACKLETON	DATE ; 26-07-04	JOB No	
CHECKED ;	SCALE ;	APPRVD ; A.T.	REVISION ;

A4

AVOPILING



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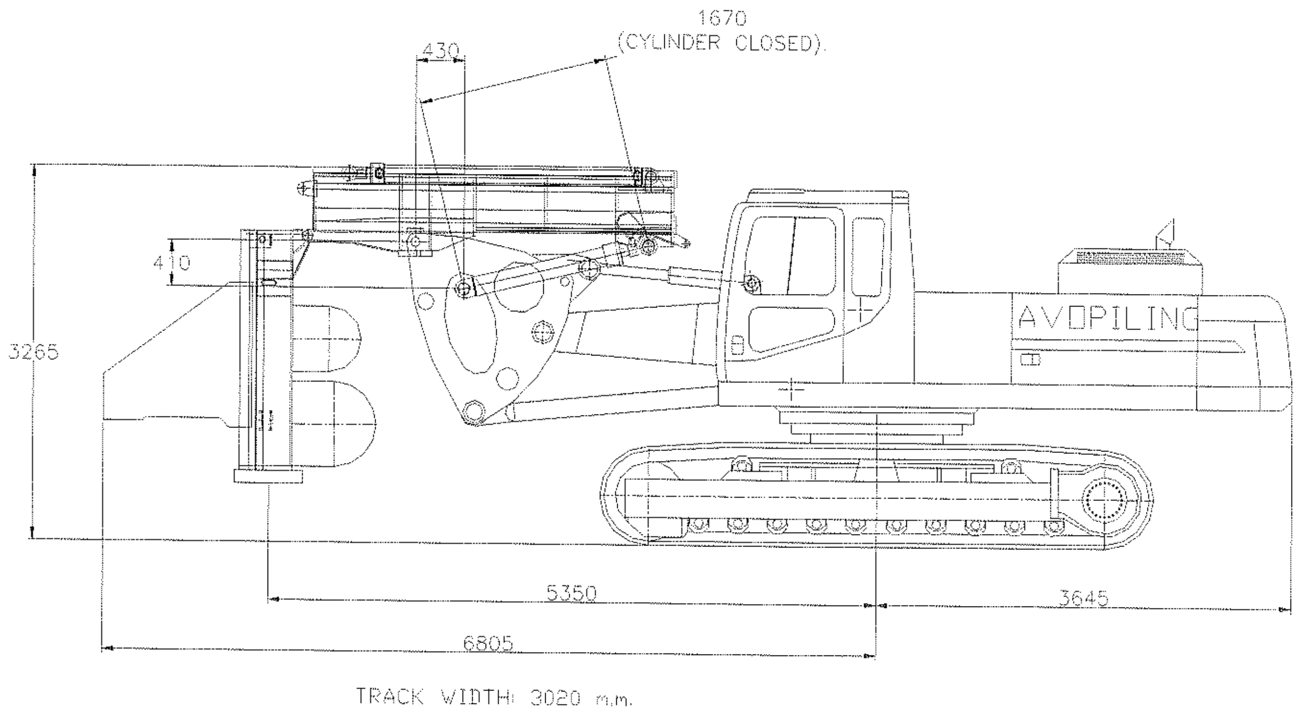
METRIC - DO NOT SCALE
DRAFTING STANDARD
AS 1100

DESCRIPTION SHORT MAST DRILLING MACHINE
36.5 DEG DRILLING ANGLE.

DRAWN : [illegible]
DATE : [illegible]
JOB No
DWG No : [illegible]
CHECKED : A.M.
SCALE : [illegible]
APPROVED : A.T.
REVISION :

A4

AVOPILING



TRACK WIDTH: 3020 m.m.

METRIC – DO NOT SCALE
DRAFTING STANDARD
AS 1100

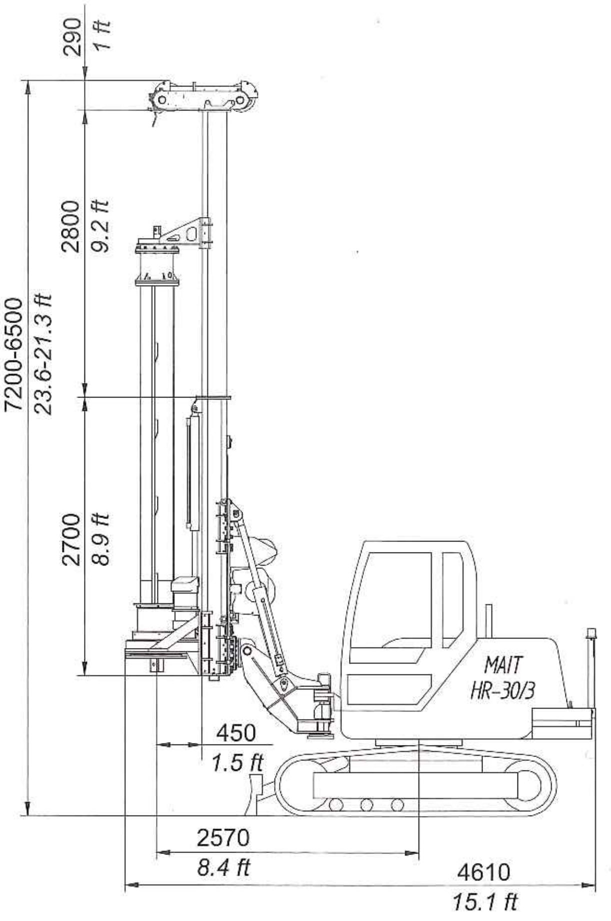
DESCRIPTION: SHORT MAST PROJECT.
MAST DISASSEMBLED & FOLDED FOR TRANSPORT.

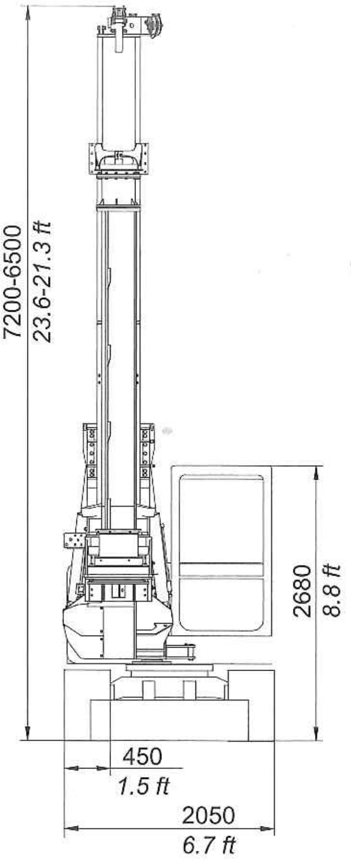
DRAWN: [illegible]	DATE: [illegible]	JOB No:	DRG No: [illegible]
CHECKED: J.M.	SCALE: 1:[illegible]	APPRD: [illegible]	REVISION:

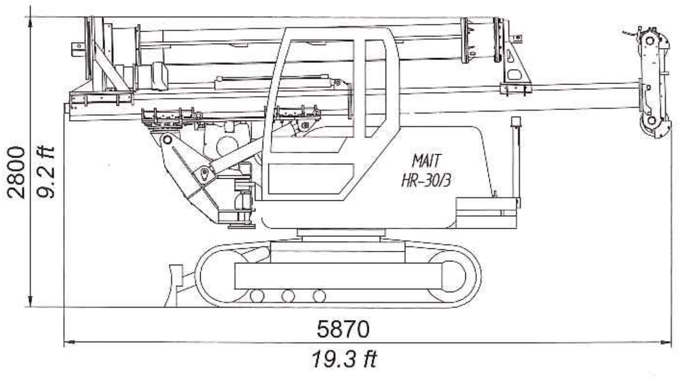
A1

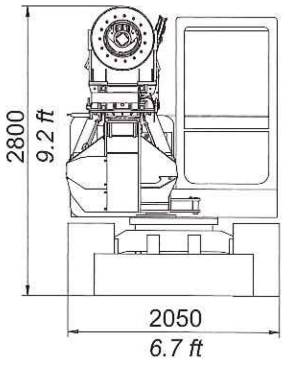
AVOPILING

MAIT









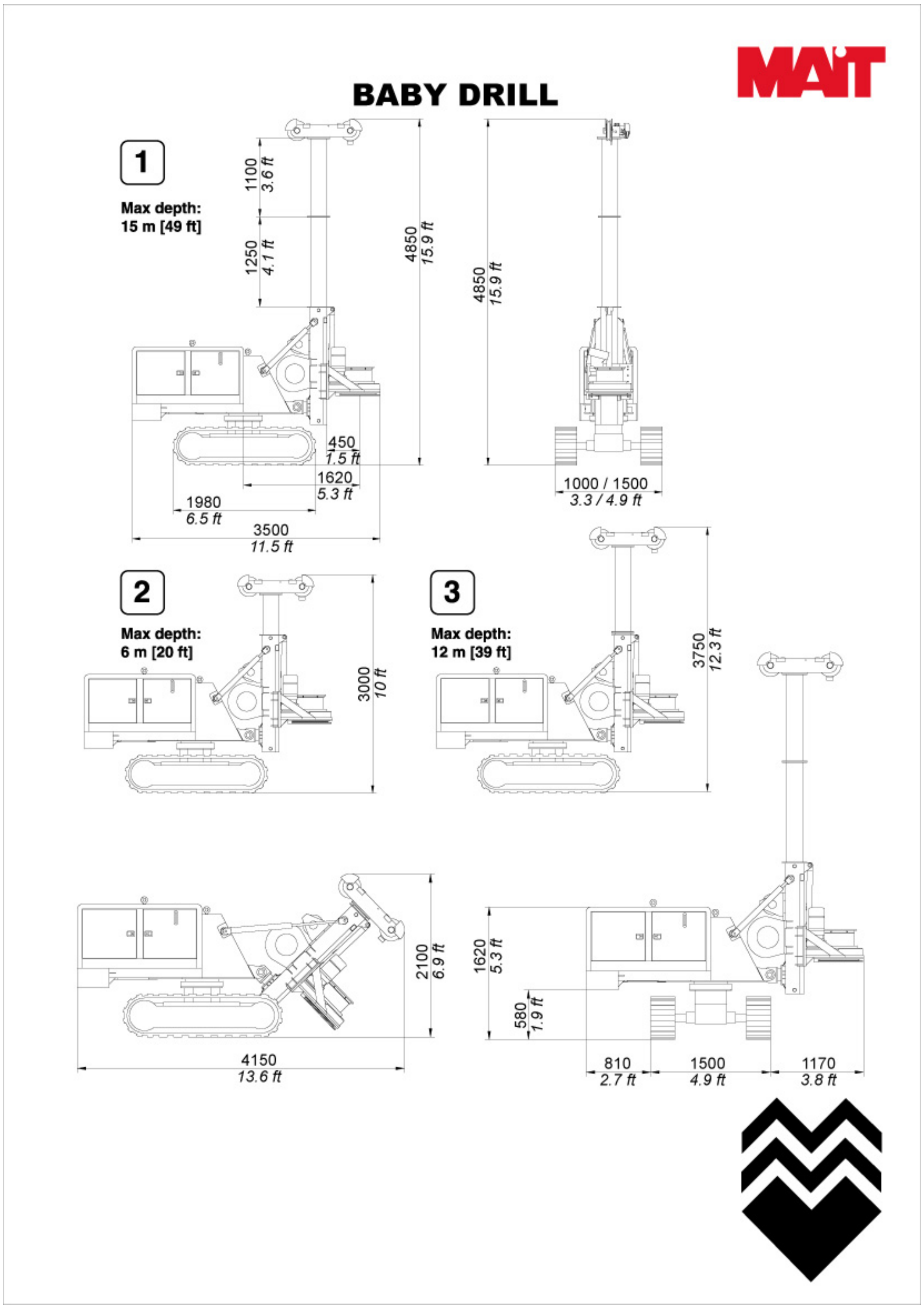
Technical Data

Mounted on KOMATSU PC 75 excavator base

Rotary:		
Nominal torque	35 KNm	25,800 lbsft
Max nominal pulldown	30 KN	6,750 lbs
Max nominal pullback	40 KN	9,000 lbs
Unloading speed	84 rpm	
Rotary stroke	950 mm	3.11 ft
Winches:		
Main winch nominal pullback	36 KN	8,100 lbs
Auxiliary winch nominal pullback	10 KN	2,250 lbs
Diesel Engine:	**KOMATSU 4D98E-2XFB**	
Power	50 KW 2.300 rpm	67 HP @ 2,300 rpm
Hydraulic Installation	2 pumps	
Hydraulic oil tank	70 liters	18.48 gal
Fuel Tank	100 liters	26.4 gal
Crawler:		
Length	2.875 mm	9.5 ft
Width	2.050 mm	6.7 ft
Shoes	450 mm	1.5 ft
Approx. Weight:	11,2 tons	24,685 lbs

MAIT S.p.A. DRILLING RIGS
via Flaminia Seconda, 149/153 - P.O. Box 1040 60027 OSIMO (AN) ITALY
- TEL +39 071 7822186 - FAX +39 071 780535
- web: http://www.mait.it - E-mail: info@mait.it

01/2010 All technical data are to be considered as approximate. MAIT reserves the right to modify them opportunely. All numbers shown in feet and inches have been rounded and are approximate for the simplification of conversion. The nominal values are referred to 100% total efficiency.


MAIT
BABY DRILL
1
Max depth:
15 m [49 ft]
1100
3.6 ft
1250
4.1 ft
4850
15.9 ft
4850
15.9 ft
450
1.5 ft
1620
5.3 ft
1980
6.5 ft
3500
11.5 ft
1000 / 1500
3.3 / 4.9 ft
2
Max depth:
6 m [20 ft]
3000
10 ft
3
Max depth:
12 m [39 ft]
3750
12.3 ft
2100
6.9 ft
4150
13.6 ft
1620
5.3 ft
580
1.9 ft
810
2.7 ft
1500
4.9 ft
1170
3.8 ft

Appendix E

Existing In-Ground Services

Job Name : SCEGGS Darlinghurst

Sketch Title:

In-ground services

Date: 21/08/2019

By: JA

TTW Taylor Thomson Whitting Job Number: 181375

Project

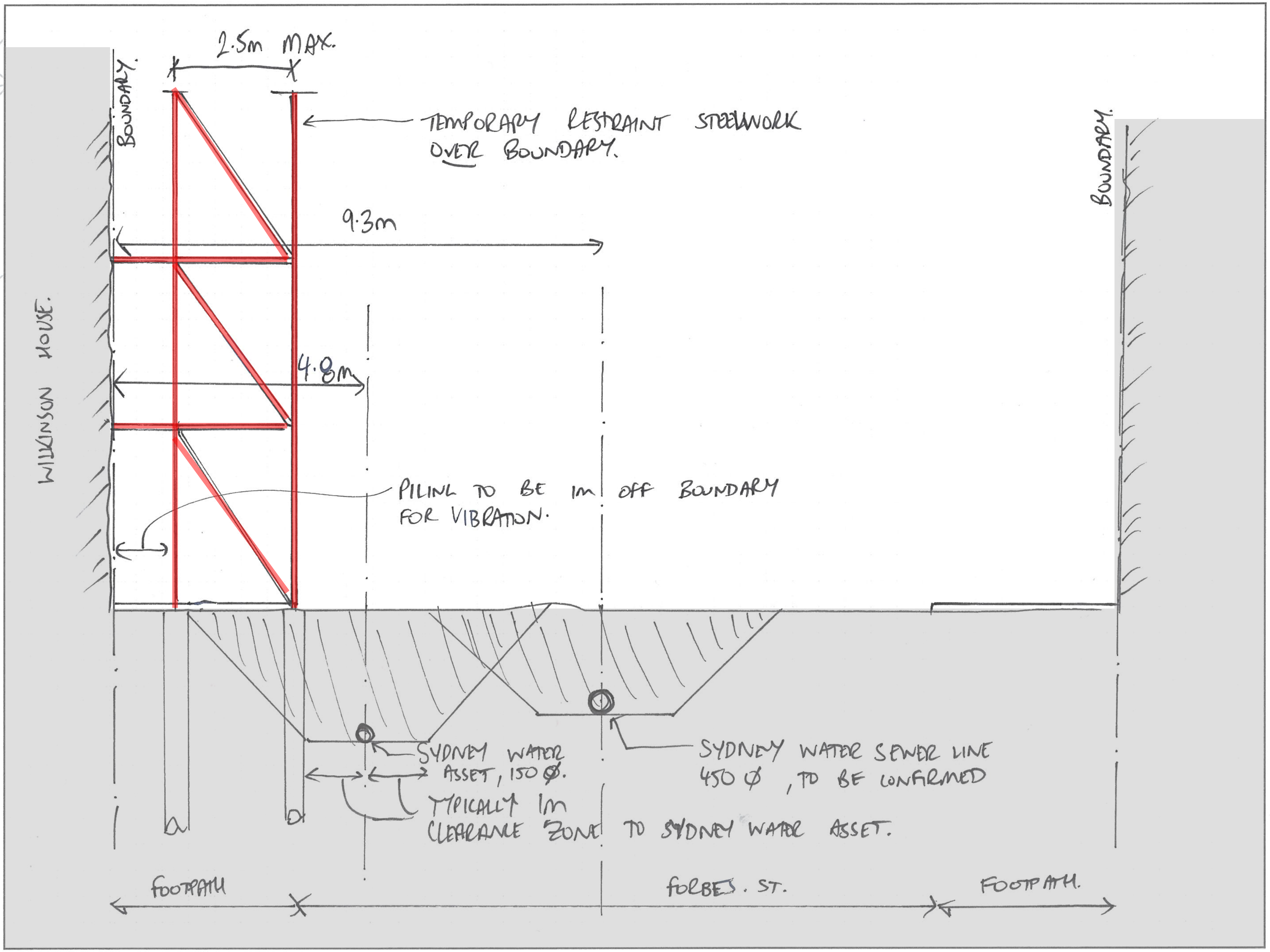
Subject

Job No.

By

Page No.

Date



TEMPORARY RESTRAINT STEELWORK OVER BOUNDARY.

PILING TO BE 1m OFF BOUNDARY FOR VIBRATION.

SYDNEY WATER ASSET, 150 Ø.

SYDNEY WATER SEWER LINE 450 Ø , TO BE CONFIRMED

TYPICALLY 1m CLEARANCE ZONE TO SYDNEY WATER ASSET.