THE STAR

PRELIMINARY GEOTECHNICAL ASSESSMENT

PROPOSED RITZ CARLTON TOWER

THE STAR
80 PYRMONT STREET
PYRMONT, NSW

PREPARED BY

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical assessment for the proposed Ritz Carlton tower at The Star, 80 Pyrmont Street, Pyrmont, NSW.

Based on the relevant supplied architectural drawings prepared by FJMT Studio (Project Code: SM13, Sheet Nos. AF100, AF101, AF102, AF200 to AF204, AF2000, AF5001 & AF5102, Revision DA01, dated 1 September 2017), we understand that the top of the proposed tower will be at approximately RL237.0mAHD; approximately 234m above the Pirrama Road footpath level. The maximum plan dimensions of the proposed tower are approximately 60m x 30m. We have been advised by Taylor Thomson Whitting (TTW) that the column working loads will typically be in the order of 8,000kN to 25,000kN.

Based on the supplied survey plan prepared by Hard & Forester, titled 'Casino Site, Site Perimeter, Jones Bay Rd, Foreshore Rd & Edward Street' (Drawing No. 80980030, Revision 01, dated 21/7/95), surrounding footpath surface levels range between RL110.0m (ie. RL10.0mAHD) along the western Jones Bay Road boundary and RL103.0m (ie. RL3.0mAHD) along the lower eastern Pirrama Road boundary. The site specific survey datum is 100m above AHD.

The proposed floor level of the hotel entry lobby (Level B2) will be constructed at RL103.3m (RL3.3mAHD). The proposed lowest basement level (Level B6) will be constructed at RL88.6m (RL-11.4mAHD) and will require a minimum excavation depth of about 15m below the eastern Pirrama Road boundary. Proposed Level B6 partially underlies the proposed tower and accommodates the core. The higher lying proposed Level B4 essentially extends below the entire tower and extends to the Pirrama Road and Jones Bay Road boundaries. Proposed Level B4 will be constructed at RL94.9m (RL-5.1mAHD) and will require excavation to depths between approximately 8.5m and 15.5m below the surrounding footpath levels.

On the south-eastern side of the tower, an underground car stacker is proposed. The proposed tower structure will partially overlie the proposed car stacker footprint. The lowest level of the proposed car stacker will be constructed at RL74.1m (RL-25.9mAHD); that is, at least 29m below the eastern Pirrama Road boundary. The pertinent details of the proposed architectural design are presented in Plates 1 to 4 below.

We have completed a desktop study of the previous geotechnical investigations and inspections completed by Jeffery and Katauskas Pty Ltd (now trading as JK Geotechnics) and to a much lesser extent by DJ Douglas & Partners (now trading as Douglas Partners) between 1992 & 1995 for the 'Sydney Casino Project'. Based on the results of our desktop study we provide our preliminary comments and recommendations on footing design, excavation retention, rock face support, excavation induced ground movements, lateral restraint, and additional geotechnical investigations.

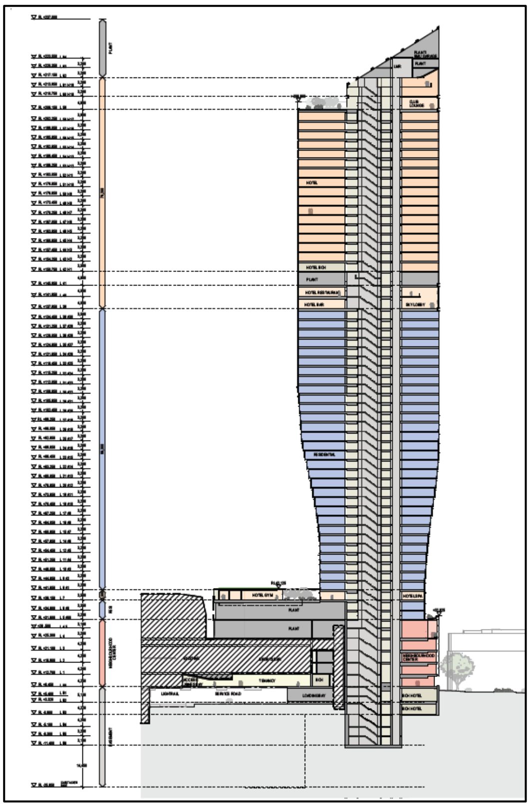

Plate 1: Section 01 through Proposed Tower (FJMT Studio, Sheet No. AF5001)

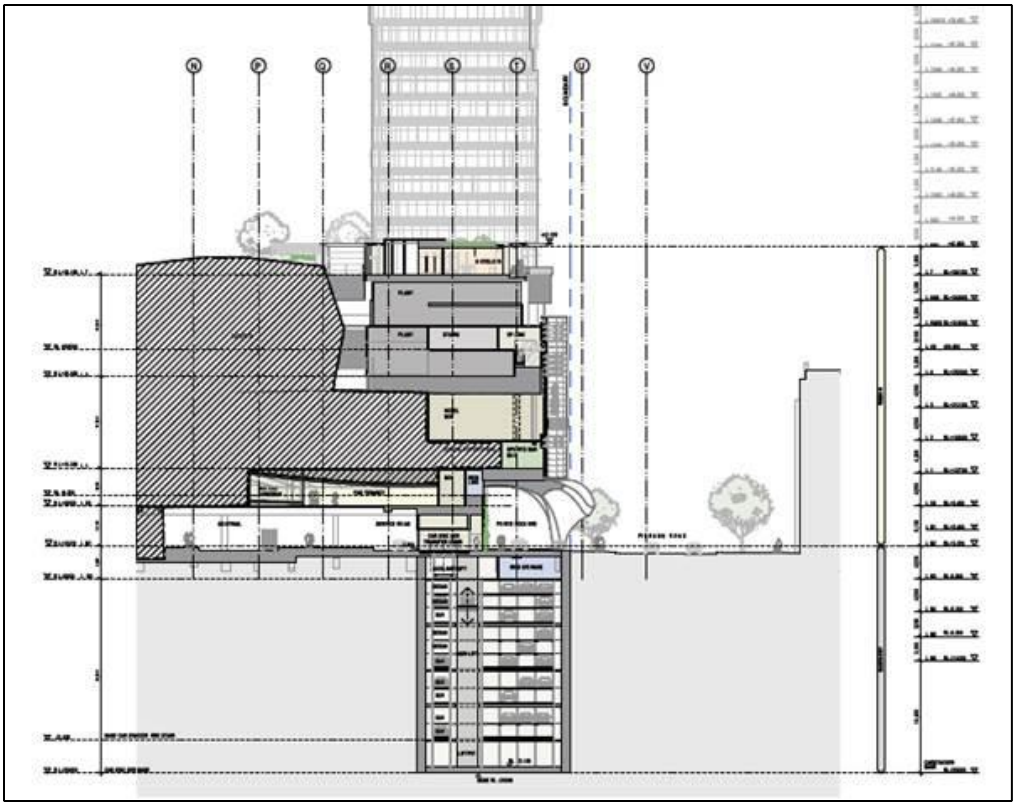

Plate 2: Section D02 through Proposed Car Stacker (FJMT Studio, Sheet No. AF5102)

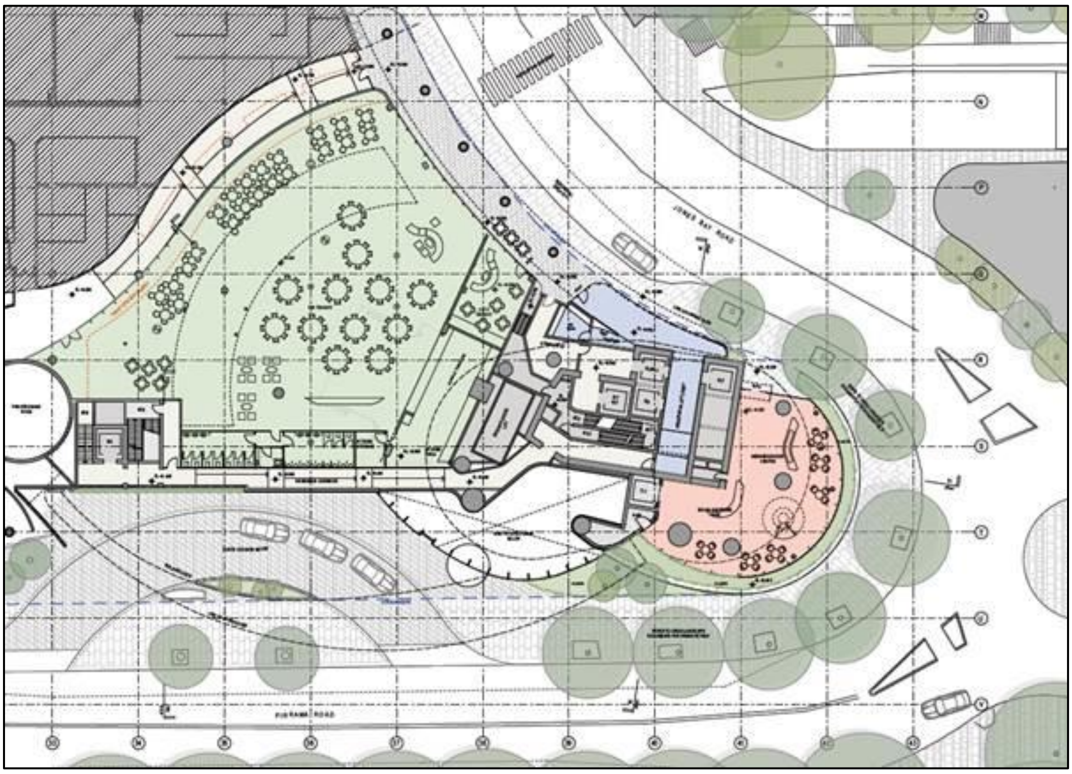

Plate 3: Level 00 Residential Entry Ground Floor Plan (FJMT Studio, Sheet No. AF2000)

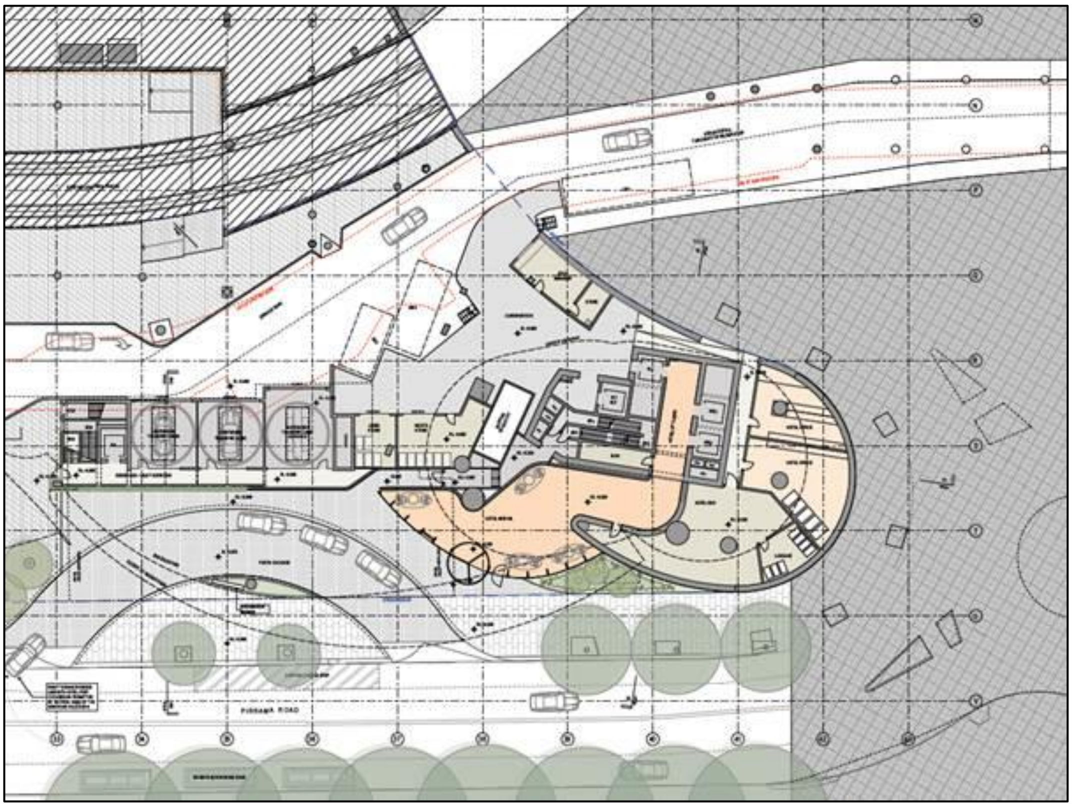

Plate 4: Level B2 Floor Plan (FJMT Studio, Sheet No. AF204)

2 PRELIMINARY GEOTECHNICAL MODEL

The 1:100,000 series geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Hawkesbury Sandstone.

Five previous boreholes were drilled within the proposed tower footprint; one by Douglas Partners (DP) and the remainder by JK Geotechnics (JK). A summary of the pertinent details encountered in the previous boreholes is tabulated below:

Borehole	Surface RL (m) [100m above AHD]	Approximate Grid Location	Top of Sandstone Bedrock		Borehole Termination		Groundwater RL (m) during Drilling [100m below AHD]
			Depth (m)	RL (m)	Depth (m)	RL (m)	
DP BH6	104.2	B/W T & U B/W 40 & 41	1.6	102.6	1.65	102.55	Not encountered
JK T/U 41/42	105.0	B/W T & U B/W 41 & 42	3.3	101.7	4.5	100.5	101.2
JK T39	102.9	T/39	2.8	100.1	6.7	96.2	Not encountered
JK T42A	106.9	T/42	4.0	102.9	7.2	99.7	Not encountered
JK R41	108.0	R/41	5.0	103.0	7.6	100.4	Not encountered

The soil profile in these five previous boreholes typically comprised sandy fill with sandstone gravel, cobbles and boulders. Sandstone bedrock was encountered between RL100.1m & RL103.0m. Only boreholes T39, T42A and R41 were diamond core drilled. An indicative engineering classification of the sandstone bedrock has been carried out for boreholes T39, T42A and R41 (in accordance with 'Foundations on Sandstone and Shale in the Sydney Region' by Pells et al., Australian Geomechanics, December 1998) and is tabulated below:

Borehole	Surface RL (m) [100m above AHD]	Indicative Engineering Classification of Sandstone Bedrock Depths (m) [RL(m) at Top of Stratum]				
		Class V	Class IV	Class III	Class II	Class I
JK T39	102.9	2.8-3.7 [100.1]	3.7-6.7 [99.2]	-	-	-
JK T42A	106.9	4.0-5.6 [102.9]	-	5.6-7.2 [101.3]	-	-
JK R41	108.0	-	5.0-6.5 [103.0]	6.5-7.6 [101.5]	-	-

3 PRELIMINARY COMMENTS AND RECOMMENDATIONS

3.1 FOOTING DESIGN

Proposed basement levels B4 and B6, which underlie the tower structure, will be constructed at RL94.9m (RL-5.1mAHD) and RL88.6m (RL-11.4mAHD), respectively. For preliminary design purposes, footings at these levels should be tentatively designed to be founded in Class IV or better quality sandstone for a maximum allowable bearing pressure of 2,000kPa or an ultimate bearing pressure of 10MPa.

The proposed car stacker will be constructed at RL74.1m (RL-25.9mAHD); well below the above mentioned borehole termination depths. Based on our experience on the nature and quality of the bedrock at depth, we expect that the foundation material will likely comprise Class II or better quality sandstone. For preliminary design purposes, pad footings in this area can be tentatively designed to be founded in Class II sandstone for a maximum allowable bearing pressure of 10,000kPa or an ultimate bearing pressure of 100MPa.

Typical elastic parameters for the Class IV and Class II sandstone are provided in the table below:

Sandstone Class	Typical Elastic Modulus (MPa)	Typical Poisson's Ratio
IV	400	0.25
II	1500	0.15

The provided allowable bearing pressures are based upon serviceability criteria of deflections at the footing base of less than 1% of the minimum footing dimension/pile diameter. We note that these footing settlements will be of an elastic nature and are expected to occur as construction proceeds.

We note that the ultimate limit state design values provided above occur at larger settlements. Notwithstanding, settlement limitations to the proposed building will still need to be satisfied and can be estimated using the above Elastic Modulus values. It should be noted that the ultimate bearing pressures must be used in conjunction with an appropriate "*Basic Geotechnical Strength Reduction Factor*" (ϕ_{gb}), as defined in Clause 4.3.2 of AS2159-2009 ('Piling – Design and Installation'). A specific assessment of the ϕ_{gb} value must be made in accordance with the procedure set out in AS2159-2009. For Hawkesbury Sandstone, a ϕ_{gb} value of 0.75 is quite commonly accepted.

We have carried out preliminary elastic settlement analyses for footings founded on the Class IV and Class II sandstone. The plan areas of the footings have been proportioned based on the above recommended maximum allowable bearing pressures.

Sandstone Class	Expected Elastic Settlement (mm) for Column Working Load		
	8,000kN [proportioned pad footing plan area, m x m]	15,000kN [proportioned pad footing plan area, m x m]	25,000kN [proportioned pad footing plan area, m x m]
IV	10 [2 x 2]	10-15 [2.8 x 2.8]	15-20 [3.6 x 3.6]
II	5-10 [1 x 1]	5-10 [1.3 x 1.3]	10-15 [1.6 x 1.6]

Stability considerations must be given to all new tower footings founded on the higher lying Class IV sandstone immediately behind the proposed car stacker excavation cut face; refer to Section 3.3 below.

During detailed design and following completion of the site investigation recommended in Section 3.6 below, we recommend that finite element analyses be completed to assess total and differential settlements, particularly if footings for the proposed tower are founded at different levels.

3.2 EXCAVATION RETENTION

Based on the results of our preliminary assessment, the geotechnical model comprises sandy fill overlying weathered sandstone bedrock at shallow to moderate depths, then competent sandstone. Sandstone bedrock was encountered between RL100.1m and RL103.0m; that is, to a maximum depth of 6-7m below current grade.

The primary geotechnical consideration for excavation retention is groundwater and likely inflow rates. Detailed site investigations and groundwater modelling will be required, as recommended in Section 3.6 below. DPI Water will likely permit drained basements (ie. for the proposed tower and car stacker) if groundwater inflows into the excavation of less than 3ML/year can be justified. We expect that preferential seepage paths will be at the soil-rock interface, and through defects (eg, open joints, bedding partings, etc.) in the sandstone bedrock, particularly the upper Class V and Class IV profiles. Design and construction of the basement walls will be an iterative design process between the architect, structural engineer, geotechnical engineer, and the basement wall contractor.

In the long-term and depending on seepage inflow rates, the proposed basement walls may need to be tanked. For this scenario, construction of the proposed basement levels will require perimeter cut-off walls into competent sandstone bedrock, internal dewatering, and possibly recharge immediately outside the cut-off walls.

Despite the basements being tanked in the long-term, a temporary dewatering licence from DPI Water may be required during excavation and basement construction. In order to assess whether a licence is needed and whether the basements will need to be tanked, the groundwater inflow volumes and quality will need to be assessed.

The major consideration with the construction of the proposed basement levels is dewatering and the provision of a suitable cut-off wall around the perimeter so that groundwater inflows into the excavation and groundwater drawdown outside the excavation can be controlled. Our preferred retention/cut-off systems to support the vertical cuts through the fill profile as well as through the Class V sandstone profile, and possibly through the Class IV sandstone profile include:

- Diaphragm walls. Due to the limited site access, the diaphragm walls would most likely be constructed using a mini-hydrofraise in order to penetrate the bedrock (refer to Plate 5 below).
- Secant pile walls. Due to the presence of collapsible sandy fill and groundwater at the soil-rock interface, the secant pile wall would need to comprise continuous flight auger (CFA) piles.

During the excavation of each panel of the diaphragm wall using the mini-hydrofraise, the ‘slot’ is supported from collapse using bentonite fluid and/or liquid polymers which are recycled through specialist plant and equipment (ie. silos, a batch plant and a de-sander); refer to Plate 6 below. Consideration would therefore need to be given on where the plant and equipment can be set up. The set-up for the subject project is expected to be slightly smaller than that shown in Plate 6.

If a drained basement can be justified by field testing, monitoring and analyses, then the retention system could comprise contiguous piled walls constructed using CFA piles.

The above retention systems would need to be installed following the demolition works and prior to excavation commencing. The retention systems should be ‘cut-off’ in competent bedrock (ie. Class III or better quality sandstone) either above or below bulk excavation level. The retention systems must be progressively internally anchored or propped as the excavation proceeds (ie. once the restraining point has been uncovered). Prior to the installation of anchors, which will extend outside the site boundaries, permission must be sought from the respective neighbouring property owners.

If the retention system is ‘cut-off’ above bulk excavation level, then the basement floor slabs will need to be designed to provide long-term lateral support to the basement walls. Once the basement floor slabs adequately engage the retention system, then the temporary rock anchors can be de-stressed and/or internal props can be removed. In front of and below the toes of the retention system, the Class III or better quality sandstone bedrock can be cut vertically as discussed in Section 3.3 below.

If retention systems founded above bulk excavation level are required to support the proposed superstructure, then wall sections (ie. barrettes) for the diaphragm wall option will need to be extended down to below bulk excavation level in order to achieve a suitable bearing stratum. Similarly, for the secant pile wall option, select hard piles will need to be extended down to below bulk excavation level in order to achieve a suitable bearing stratum.

We expect that the deeper Class III or better quality sandstone bedrock will be relatively impermeable. If this can be proven, then a drained (lowest) basement slab may be feasible. If not, then a hydrostatic slab will be required.

Due to the close proximity of the site to the harbour foreshore, groundwater at least in the fill profile should be assumed at this stage to be saline. It is possible that the groundwater within the sandstone rock mass is not saline, however, this will need to be proven during site investigations.

Plate 5: Mini-hydrofraise (Photograph courtesy of Menard Oceania)

Plate 6: Typical silo, batch plant and de-sander set-up (Photograph courtesy of Menard Oceania)

3.3 ROCK FACE SUPPORT

With regards to rock face support, we expect that the deeper Class III or better quality sandstone can be cut vertically. However, as defects (particularly inclined joints, extremely weathered seams and clay seams) were encountered in the previous cored boreholes, we recommend that all vertically cut rock faces be incrementally inspected by an experienced geotechnical engineer at no more than 2m depth increments and on completion of excavation to identify features which may require stabilisation (eg. underpinning, rock bolting and/or shotcrete and mesh). As a guide, the following rock face support should be expected for the deeper, competent sandstone:

Sandstone Class	Likely Rock Face Support Measures
III	• Localised pattern bolting in fractured zones. • Mesh, shotcrete and dowels OR fibrecrete and dowels in fractured zone and weathered bands. • Isolated rock bolting of potentially unstable rock wedges.
II & I	• Isolated rock bolting of potentially unstable rock wedges.

Provision must be made in the construction program and budget for the inspections and stabilisation works.

Where the retention system is founded in Class III or better quality sandstone bedrock above bulk excavation level, then the quality of the rock at the base of the retention system must be inspected by an experienced geotechnical engineer once the excavation has extended no more than 1m depth below the base of the wall. This is to assess whether underpinning or rock face support is required.

As the dowels and rock bolts will be permanent, consideration must be given to long-term corrosion. Stainless steel bolts or hot dipped galvanised bolts with protective sheathing may be required.

3.4 EXCAVATION INDUCED GROUND MOVEMENTS

Ground movements outside the basement perimeter are expected to occur due to the proposed excavation. The magnitude of adjacent ground movements within the retained fill profile, Class V and Class IV sandstone profiles will depend on the ground conditions, design lateral pressures, construction sequence and workmanship. Lateral movements of the adjacent ground surface for an anchored or internally propped retention system supporting the fill and weathered sandstone profiles are expected to be in the order of 0.3% to 0.5% of the retained height.

Within an unsupported rock excavation through Class III or better quality sandstone, additional lateral movements will likely occur due to relief of insitu horizontal stresses. Such insitu horizontal stresses are typically in the order of 1MPa or greater. Based on our experience with deep basements in Hawkesbury Sandstone in the Sydney CBD, typical lateral movements due to stress relief are in the order of 0.5mm to 2mm per metre depth of excavation, depending on rock quality and presence of joints and bedding partings. The maximum lateral movements are likely to occur towards mid-height of the unsupported rock faces.

3.5 LATERAL RESTRAINT

Lateral stiffness parameters for the sandstone profile will most likely be required to assist the structural engineers with their analysis of the proposed structures. Based on the preliminary geotechnical model provided in Section 2 above and our previous deeper boreholes within The Star basement excavation further to the south, we tentatively expect the geotechnical model down to RL74.1m to comprise the following:

Stratum	Indicative RL(m) at Top of Stratum [100m above AHD]	Typical Elastic Modulus (MPa)	Typical Poisson's Ratio
Existing Surface Level/Granular Fill	103 (eastern side) to 110 (western side)	NA	NA
Class V & IV Sandstone	100 (eastern side) to 103 (western side)	100 - 400	0.25 - 0.3
Class III Sandstone	101	800	0.2
Class II Sandstone	95	1500	0.15

We have provided a range in the typical elastic parameters for the Class V and Class IV sandstone to account for the expected variability in the upper weathered sandstone profile across the proposed building footprint. Sensitivity analyses should be carried out for these ranges in values.

The above typical range in elastic parameters should be adopted in the development of stiffness values for use in a spring model. Essentially, the edge of each basement floor slab bearing laterally against the excavated rock faces will behave as strip footings. From the outcomes of the preliminary spring model analyses, further iterations of the stiffness values may be required depending on the extent of movement. Applied bearing pressures on the cut rock faces from the edges of the basement floor slabs should not exceed the allowable values recommended in Section 3.1 above. Edge thickenings may be required for greater load distribution onto the cut rock faces.

3.6 ADDITIONAL INVESTIGATIONS

Following demolition of the existing building, we recommend that deep cored boreholes be completed at each heavily loaded column for the proposed tower in order to optimise the design bearing pressures.

In addition and for preliminary design purposes, we recommend that at least two cored boreholes be completed around the existing building to assess the subsurface conditions at depth as well as the groundwater characteristics. The Star is planning to commence the preliminary borehole investigations after formal lodgment of the development application.

Within the soil profile, Standard Penetration Tests (SPT) should be completed at no more than 1.0-1.5m depth intervals to assess the relative compaction/strength of the subsoil profile. Bedrock must be diamond core drilled to at least 6m below bulk excavation level. The depths of the cored boreholes below bulk excavation level may need to be significantly increased if tie-down anchors are proposed.

Insitu Packer testing should be completed in the preliminary cored boreholes to assess the permeability of the bedrock at various levels. In these boreholes, monitoring wells should be installed and sealed at different levels within the sandstone bedrock to monitor groundwater levels and quality, and also for the purpose of completing pump-out tests to assess inflow rates. Adjacent to each of the preliminary boreholes, an adjacent borehole should be drilled to the bedrock surface for the purpose of installing a second groundwater monitoring well. Groundwater sampling and analysis from these six boreholes will be required to satisfy DPI Water, and also to assess salinity.

Groundwater level monitoring of all wells must be carried out to assess the design head of water for a tanked basement. The monitoring period should include heavy and prolonged rainfall events. Groundwater modelling and seepage analyses should then be completed to assess inflow rates into the proposed excavation.

The geotechnical investigation report would need to contain detailed advice on retention systems, groundwater quality and dewatering, excavation, excavation induced ground movements, footings, including total and differential settlements, and the lowest basement floor slab. We recommend that 3D Plaxis (finite element) analyses be carried out to model the rock-structure interaction.

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. The waste classification assessment should be carried out concurrently with the detailed geotechnical investigation (post-demolition) so that the appropriate samples are obtained from the geotechnical boreholes.

4 GENERAL

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Should you require any further information regarding the above, please do not hesitate to contact the undersigned.

Yours faithfully

For and on behalf of
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THE STAR