Geotechnical Investigation Report

Proposed Development at
37-39 Loftus Crescent, Homebush NSW

prepared for

Georges Constructions Pty Ltd

Report No. G140

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REFERENCES

1. Australian Standard AS1726-1993 ‘Geotechnical Site Investigation’; and
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1.0 INTRODUCTION

Benviron Group was engaged to undertake a geotechnical investigation at the subject site located at 37-39 Loftus Crescent, Homebush. The purpose of this investigation is to assess the site’s surface and subsurface conditions within accessible areas in order to provide geotechnical recommendations and advice for the design and construction of the proposed residential development in preparation for a development application submission.

The proposed development involves the demolition of all existing site features and the construction of a 9-storeys residential building with a 3-level basement car parking facility.

This report presents and interprets the findings of the geotechnical investigation carried out on the 10th and 11th March 2016 at the subject site, known as number 37-39 Loftus Crescent, Homebush. With the site constraints, the fieldwork was carried out by using a track mounted drilling rig with coring facilities to accessible parts of the site. Two boreholes holes were drilled and cored to provide the following information:

- Method of investigation,
- Site description, including surface and sub-surface conditions,
- Site plan indicating borehole locations and footprint of the proposed roads and buildings within the development,
- Groundwater conditions and management,
- Result of laboratory rock testing,
- Recommendations on the excavation conditions and temporary slope batters,
- Recommendations on vibration control and management,
- Provision of earth pressure parameters for design of retaining structures,
- Recommendations on footings and serviceability bearing pressures,
2.0 AVAILABLE INFORMATION

At the time of writing this report, a set of architectural drawings from the client as listed below, by GM Architects, referenced Project No. 15811 were provided to us.

3.0 PROPOSED DEVELOPMENT

3.1 Site Location and Description

The site is located near the Homebush at approximately 11 Km west of the Sydney CBD as shown in the Locality Plan (Figure 1). The site comprises two adjoining lots, and is bounded by Loftus Crescent to the south and neighbouring properties to other sides. A single storey dwelling is currently located on both lots as shown in the Site Plan (Figure 2). The proposed development comprises a 5 storey building with a 2 level basement carpark.

3.2 Regional Geology

Reference to the Sydney 1:100,000 Geological Series Sheet 9029-9130 Edition 1, 1983, indicates the site to be underlain by Ashfield Shale of Middle Triassic Age. Ashfield Shale is typically consisted of dark grey to black shale and laminate.
4.0 FIELDWORK

Fieldwork for the geotechnical investigation was carried on the 10th and 11th March 2016 and comprised the following works:

- A detailed walk-over inspection of the site and surrounding environment to capture any significant geological features.
- Drilling of two (2) boreholes, BH1 to BH2 using a drilling rig mounted with V-bit attached to a solid flight auger and then using NMLC diamond rock coring techniques to drill down to a total depth of 10m (BH1) and 9m (BH2) respectively below existing ground level.
- Standard Penetration Tests (SPT) was undertaken at regular intervals within the borehole to assess the in-situ strength of subsoil properties.

The approximate locations of the 2 boreholes are shown in Figure 2 and the Engineering Logs are presented in Appendix B.
5.0 FIELD WORK RESULTS

5.1 Subsoil Conditions

Based on information gathered and observations made from the site inspection, it can be inferred that it is likely the subsoil profile comprises fill overlying residual silty CLAY or clayey SILT underlain by the Shale Bedrock.

The subsurface soil profile within BH1 comprises shallow fill underlain by a mixture of stiff to very stiff low plasticity silty clay/clayey silt to a depth of approximately 1.60m below existing ground level. Extremely weathered shale is encountered at approximately 1.60m below existing ground level (bgl) with extremely low strength. Its strength increases gradually with depth from low to medium strength at the depth of about 6.3 m bgl. This borehole was terminates at 10m below existing ground level.

The subsurface soil profile within BH2 exhibits similar subsurface profile to that of BH1. The fill is underlain by a mixture of stiff to very stiff low plasticity clayey silt/silty clay. Weathered Shale bedrock commenced at approximately 1.40m with extremely low to very low strength between 1.40m and 6.0m bgl. This borehole was terminated at 9.0m below existing ground level.

5.2 Ground Water

Groundwater seepage was not observed during auger drilling of the boreholes. Groundwater level was measured at a depth of about 3.4 mBGL, on 20th March 2016, which is expected to be above the final bulk excavation level.

However, due to the expected low permeability of the residual clay and shale bedrock, we do not expect the groundwater to have a major impact on the proposed development or adjoining structures. However, we expect minor groundwater inflows into the excavation at the soil/rock interface and through any defects within the shale bedrock (such as jointing, and bending planes, etc.) following periods of heavy rain.
We expect that any seepage that does occur will be able to be controlled using a conventional sump-and-pump system. In the long term, drainage should be provided behind all basement retaining walls, around the perimeter of the basement and below the basement slab. The hydraulic engineer should inspect the completed excavation to confirm that adequate drainage has been allowed for. Drainage should be connected to the sump-and-pump system and discharging into the stormwater system.
6.0 DISCUSSIONS AND RECOMMENDATIONS

6.1 General

From the result of the investigation obtained on this site, it can be inferred that the subsoil conditions comprise generally shallow fill overlying stiff to hard low plasticity residual clayey silt/silty clay and extremely weathered shale underlain by low to medium strength weathered Shale.

The proposed development involves the demolition of all existing site features and the construction of a 9-storeys residential building with a 3-level basement car parking facility. Formation of the basement is expected to entail excavations down to a level at approximately up to 9m deep. The proposed basement will extend to close proximity of the eastern and western site boundaries, and will be set back by approximately 3.0m from the southern boundary where it adjoins Loftus Crescent.

6.2 Excavation Conditions and Vibration Control

The proposed basement bulk excavation materials are likely to comprise mostly stiff to hard residual soils and extremely weathered shale to the base of excavation.

Based on the available information from BH1 and BH2, we anticipate that the excavation of such materials can be achieved by conventional excavation methods. However, harder shale may encounter towards the base of the excavation, where rock breaking hammer may be required to increase the excavation performance. It is less likely that high strength shale bedrock is encountered within the proposed excavation (This is to be confirmed within the northern section of the site by additional boreholes when access is provided after demolition of existing buildings); however, if it is encountered saw cutting method may be used along the excavation perimeter followed by rock breaking technique. This is to reduce the vibrations transferred to neighbouring properties and reduce any risk to the integrity of the nearby structures.
Should vibratory rock breaking technique be required, we recommend that ground vibrations induced by rock hammering be monitored along the site boundary, particularly at vicinity of the existing rail corridor to the south, to ensure that it is within acceptable level. Induced vibrations in structures adjacent to the excavation should not exceed a peak particle velocity (PPV) of 10mm/sec for structures in good condition or 2mm/sec for heritage or poor-conditioned structures.

We recommend that where percussive excavation techniques are to be adopted, dilapidation reports are to be carried out on all adjoining buildings, roads and civil structures so that an accurate record of the existing conditions of these elements are mapped prior to the commencement of excavation. These records shall be agreed by the respective owner in order to reduce the risk of future owner’s dispute on subsequent potential damage claims.

### 6.3 Ground Water Management

Seepage of groundwater was observed during the site investigation works, however with regard to the site topography, groundwater seepage into the proposed excavation is likely to be minor if any, and can be managed relatively easily during excavation works using toe drainage and sump pump.

### 6.4 Temporary Batter Slopes

With the proposed development, there is very limited room for the use of temporary batter slopes during construction as most of the building perimeter walls are very close to the boundaries. However wherever required (such as an access ramp for construction propose) the safe temporary batters are recommended as presented in Table 2, provided that the basement excavation is set back sufficiently from the common site boundaries to facilitate for the formation of the batters.
Table 2: Minimum temporary batter slopes

<table>
<thead>
<tr>
<th>Materials</th>
<th>Temporary (Horizontal: Vertical)</th>
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<tr>
<td></td>
<td>Exposed</td>
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<tr>
<td>Silty and clayey soils, sand-clay mixtures</td>
<td>2.0:1.0</td>
</tr>
<tr>
<td>Weathered Shale/Sandstone bedrock</td>
<td>1.0:1.0</td>
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Temporary surface protection against erosion may be provided by covering the batter with plastic sheeting, and these should be applied for a limited time and inspected by Geotechnical Engineers after significant events. It should be noted however that the plastic sheeting should extend at least 1.5m behind the crest of the cut face or at least up to the common site boundaries.

6.5 Retaining Structures

With most of the proposed basement footprint close to the site boundaries, the forming of basement will preferably need to adopt a construction methodology that exposed faces are supported at all times in particularly, to manage ground movement by limiting wall deflections to acceptable limits during all stages of the excavation works. The retaining structures should be designed to withstand the applied lateral pressures of the soil layers, the hydrostatic groundwater pressure, and all existing surcharges in their zone of influence that includes existing nearby structures and traffic loadings.

In general, it is considered a contiguous pile wall system may be necessary to ensure that the excavation faces are protected at all times. Other methods such as a soldier pile wall with shotcrete infill panels can also be adopted provided the soil and groundwater conditions are further assessed and considered in their design and the period of exposed faces should be kept to a minimum.

The pressure distribution on cantilever retaining structures may be assumed to be triangular and estimated as follows:

\[ p_h = \gamma k H + qk \]
Where,

\[ p_h = \text{Horizontal pressure (kN/m}^2\text{)} \]
\[ \gamma = \text{Wet density (kN/m}^3\text{)} \]
\[ k = \text{Coefficient of earth pressure (}k_a\text{ or }k_o\text{)} \]
\[ H = \text{Retained height (m)} \]
\[ q = \text{Surcharge pressure behind retaining wall (kN/m}^2\text{)} \]

For the design of flexible retaining structures, where some lateral movement is acceptable, an active earth pressure coefficient is recommended. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient at rest should be considered. Recommended parameters for the design of retaining structures are presented in the following Table 3.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Unit Weight (kN/m³)</th>
<th>Active Earth Pressure Coefficient ((k_a))</th>
<th>At Rest Earth Pressure Coefficient ((k_o))</th>
<th>Passive Earth Pressure Coefficient ((k_p)) or Pressure</th>
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<tr>
<td>Fill</td>
<td>18</td>
<td>0.4</td>
<td>0.6</td>
<td>ignore</td>
</tr>
<tr>
<td>Residual Soil</td>
<td>19</td>
<td>0.35</td>
<td>0.50</td>
<td>ignore</td>
</tr>
<tr>
<td>Extremely Weathered Shale (Class V or IV)</td>
<td>20</td>
<td>0.25</td>
<td>0.40</td>
<td>250kPa</td>
</tr>
<tr>
<td>Distinctly to Slightly Weathered Shale (Class IV or III)</td>
<td>21</td>
<td>0.20</td>
<td>0.3</td>
<td>375kPa</td>
</tr>
<tr>
<td>Slightly Weathered to Fresh (Class III or II)</td>
<td>22</td>
<td>0.15</td>
<td>0.25</td>
<td>500kPa</td>
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The above coefficients assume that ground level behind the retaining structures is horizontal and the retained material is effectively drained. It should be noted that hydrostatic pressures due to ground water table (if present) and surcharge due to nearby structures (within the influence zone) should also be taken into the account in the design of the retaining structures. This is particularly the case for the retaining walls located immediately adjacent to the neighboring buildings.

Should the retaining structures be anchored or strutted, the pressure distribution is assessed to be rectangular, with magnitude estimated as follows:

\[ p_h = 0.65 \gamma kH \]

The design of any retaining structure should be checked for bearing capacity, overturning, and overall stability of the slope.

### 6.6 Foundation Systems

The loading conditions for the proposed development are not known at the time of preparation of this report. However, considering the scale of the development, it is envisaged that foundation materials required to support the proposed structure would comprise Class V/IV or better shale bedrock. Based on the borehole information and rock strength testing, it is envisaged that the shale bedrock likely to be exposed at basement bulk excavation levels would comprise Class V/IV Shale.

Spread footings comprising strip or pad footings founded in Class IV/III or better shale bedrock below the basement bulk excavation level may be designed for a serviceability end bearing capacity of 1000kPa. Higher bearing capacities may be adopted subject to confirmation of additional boreholes taken to at least 3m under the proposed basement base level. It is recommended that a further drilling for assessment of the foundation material of the proposed footings be carried out once excavation to the final basement level has been reached. The footing inspection and assessment requirement can be referred to the guidelines given in accordance with Pells et al (Reference 5).
Footing inspections by a Geotechnical Engineer will be required during footing excavation to confirm presence of appropriate founding materials which meet the serviceability bearing pressures and to ensure that all soft and wet materials have been removed from the foundation footprint prior to concrete placement.

6.7 Rail Corridor

Figure 3 shows the proposed basement outline with respect to the rail corridor on the southern side of Loftus Crescent. It shows that there is a minimum distance of 16 meters between the northern boundary of the railway corridor and footprint of the proposed excavation (considering the 4m setback of the basement excavation from the site boundary adjoining Loftus Crescent). Based on the survey information provided, the base of the excavation is about 4.5 m lower than the ground level at the rail corridor. It can be seen that the proposed basement excavation is unlikely to have any adverse impact on the railway asset. This is subject to the recommendations provided in Sections 6.2 to 6.6. In particular, should vibratory rock breaking technique be required at any stage, we recommend that the vibration levels transferred to the road infrastructure and the rail corridor is closely monitored to remain within acceptable levels, set out in Section 6.2. This also includes potential vibrations induced during contiguous or soldier piles constructions. Potential displacements of retaining walls along the southern boundary should be limited appropriately to prevent any damage to the road pavement and potential buried services.

For assessing vibration induced by rock hammering method in excavating hard rock when encountered, this can be achieved by providing a hammer source with the intended rock excavation equipment (preferably in a located furthest away from the rail corridor and road infrastructure) and carry out mapping of the ground vibration at specific distances from the source. This will enable a set of site specific attenuation curves be made and the level of ground vibrations with distance from source be assessed. The radius of influence on the vibration attenuation characteristics can be made and the distance from source to provide a Peak Particle Velocity (PPV) of 5mm/sec and/or 1mm/sec then be established. In the event that the PPV
exceeds the prescribed limits along critical locations (such as at the southern site boundary adjoining Loftus Crescent) from hammering works, then presplitting technique by saw cutting is necessary to form a slot in the rock mass along the site boundary to provide a free face which act as a damper to prevent vibration from transmitting further southwards towards Loftus Crescent.

### 7.0 CONCLUSIONS

This report presents the findings and recommendations for the proposed residential development at 37-39 Loftus Crescent, Homebush NSW. It is based on the geotechnical investigation results available to date. It considers that the proposed development is feasible subjected to the recommendations presented in this report.
LIMITATIONS

The assessment of the sub-surface profile within the proposed development area and the recommendations presented in this report are based on limited information available to date.

The recommendations and advice presented in this report on soil and rock condition is considered to be indicative only as only very limited areas were assessed on site to date. Site inspection by a consulting Geotechnical Engineer or Engineering Geologist are to be undertake when further investigation works are to be carried out to confirm the condition of founding materials in which this geotechnical assessment recommends.

Anecdotal evidence and Information provided by client is assumed to be relevant and to the best of knowledge be appropriate for its interpretation.

There is a possibility that the actual geotechnical and groundwater conditions across the site could differ from the inferred geotechnical assumptions and derivations on which our recommendations are presented in this report.