RAMBOLL

BUILDINGS

TECHNICAL NOTE

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From	**Niall Meyers**
Copy to	

Prepared by	**James Wherrett**
Checked by	**Tim Clifford**
Approved by	**Simon Miller**
Description	**Supplementary Ground investigation Summary**

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Ramboll
Arkwright House
Parsonage Gardens
Manchester M3 2LF
United Kingdom

T +44 161 827 1890
https://uk.ramboll.com

1 Introduction

1.1 Scope

Wilmott Dixon (WDC) engaged Geotechnics to undertake a supplementary Ground Investigation (GI) with Ramboll UK (RUK) acting as the Engineer. The GI was required for the following reasons:

1. Validate the shallow foundation solution proposed by WSP as part of the Stage 3 Design for the residential tower
    a. Confirm depth to rock head on the north elevation of tower
    b. Assess shallow rock characteristics to confirm WSP assumptions
2. Assess depth to rockhead on the northern elevation of the bus terminal
3. Investigate the Daw Bank Retaining Wall (DBRW) foundation
4. Investigate foundations for the A6 viaduct
5. Additional contamination assessment

Geotechnics mobilised to site on the 7th September 2020 and demobilised on the 28th September 2020.

This Technical Note summarises the ground conditions, the impacts on design and construction, and recommendations for additional surveys.

2 Scope of Ground Investigation

RUK prepared a ground investigation specification (1620008272-RUK-SKZ-ZZ-SP-G-0001) in accordance with the *UK Specification for Ground Investigation* published by ICE Publishing.

The scope of the GI comprised:

- 4No. mechanically excavated trial pits up to approximately 3mbgl (targeting the southern retaining wall feature - DBRW);
- 9No. mechanically/hand excavated observation pits up to approximately 1.2/3.0mbgl (targeting the bridge viaduct foundations);
- 5No. light cable percussive boreholes with rotary follow-on to 20mbgl;
- 3No. window samples to 5mbgl;
- In-situ testing;
- Groundwater monitoring;
- Ground gas monitoring;
- Geotechnical laboratory testing;
- Geoenvironmental laboratory testing; and
- Reporting.

The original scope of the GI included geophysical ground investigation using microgravity to investigate the position of tunnels and underground voids. However, upon discussion with the specialist subcontractor, it was considered that this was unlikely to yield useful information and was withdrawn from the scope of works.

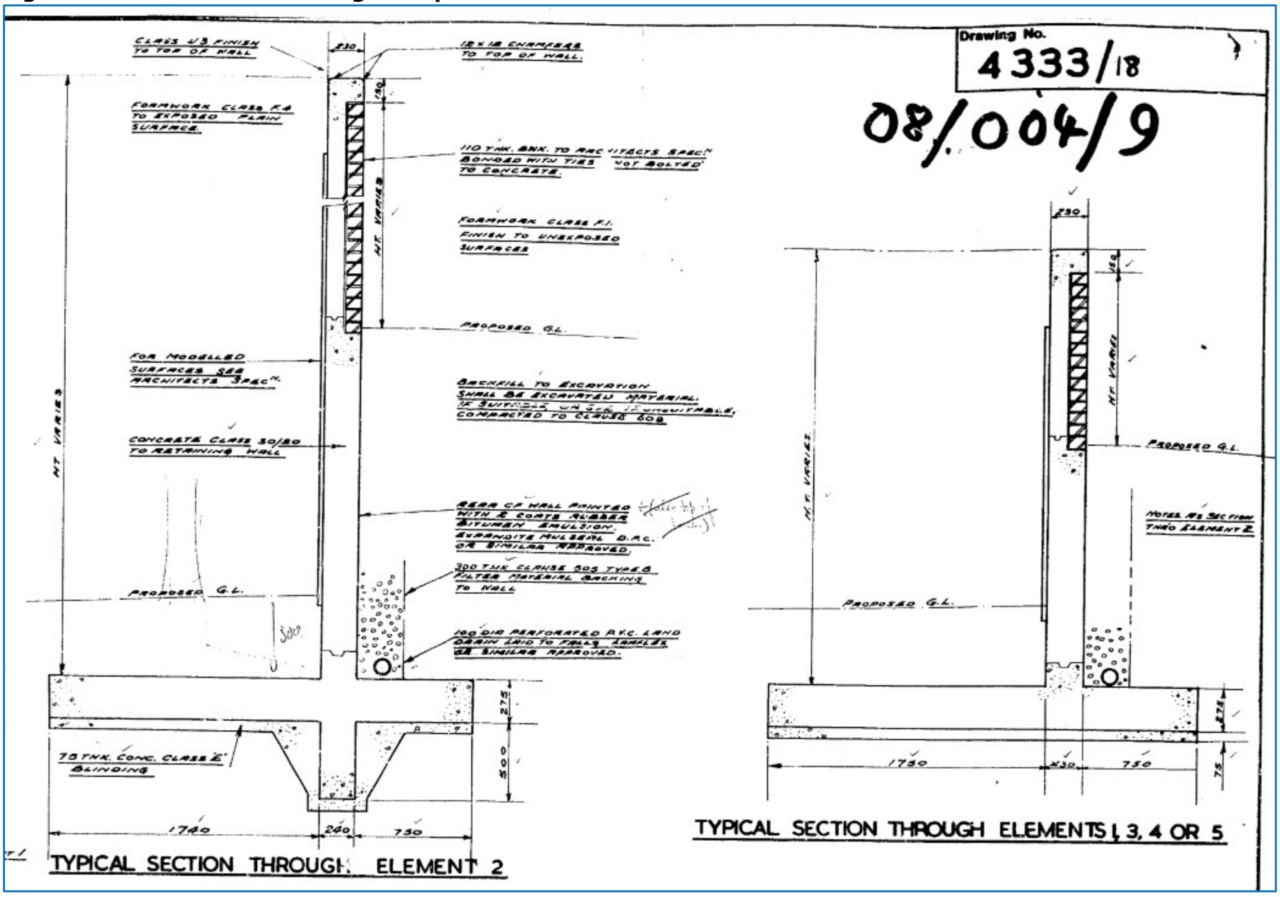
In-situ testing consisted of Standard Penetrometer Tests (SPT) within the superficial deposits and High Pressure Dilatometers (HPD) within the shallow rock layers.

3 Ground Investigation

3.1 Daw Bank Retaining Wall

DBRW runs along the southern boundary of the site and retains ~1.4m of soil at the eastern extent (Swaine Street) and ~2.5m toward the A6 viaduct in parts. The typical sections through DBRW are presented in Figure 1. The current WSP Stage 3 proposals retain the DBRW for the majority of its length and investigations were proposed to validate the geometry of the retaining wall and the founding strata. It is unclear where elements 1, 2, 3, 4 or 5 were at the site. The stability of DBRW will need to be assessed for both sections if it is to be retained as per the WSP Stage 3 proposals.

Figure 1 - Daw Bank Retaining Wall profiles.



The current WSP Stage 3 design envisages that DBRW would remain intact for the section of works adjacent to the Residential tower to provide support to the footpath on Daw Bank during the construction phase. For a section of DBRW, WSP have proposed that a contiguous pile wall be constructed to support DBRW during the construction phase. The WSP geotechnical reports do not assess the stability of DBRW during the demolition or construction phase when the passive soil mass has been removed from the toe of the wall.

RUK and WDC considered that DBRW required further investigation to confirm the accuracy of the drawings provided and to assess the founding conditions. The ground information provided to date inferred that DBRW was founded on Made Ground. The information is required to permit an assessment of the stability of DBRW.

4No. observation pits (OP) were excavated on the lower side of the wall within the bus station. 3No. OPs were unable to assess the foundation condition due to the kerbs from the bus station limiting the extent of the excavation (see Figure 2). The 1No. successful OP403 confirmed that DBRW was consistent with the record drawings and that it was founded on Made Ground consisting of Pinkish brown sandy angular fine to coarse gravel of limestone.

Figure 2 - OP400 excavated at the western end of DBRW (Swain Street end). Extent of excavation limited by kerbs.

The embedment depth at the face of the wall was consistent with the supplied drawings (Figure 1).

3.2 Boreholes – CP+RC

5No. cable percussion boreholes with rotary (coring) follow-on (CP+RC) were advanced at the site. The boreholes were advanced through the superficial (Made Ground, Glacial Deposits) to, or just above, the bedrock. Rotary coring using the Geobore S System then advanced the borehole to depth. For the boreholes on the north face of the residential tower, the first 1.5m core runs in the bedrock were advanced using a T6-H drill bit initially to form the pocket for the HPD tests. Typically, 3No. pockets were attempted to be formed within these boreholes before the rotary coring was advanced to the target depth (20mbgl) using the Geobore-S.

The two previous ground investigations consistently noted core loss or no recovery when drilling at the top of the Chester Formation. This meant that there was limited information to inform the design with parameter section based primarily on SPT data. The SPTs were typically refusing (50+ blows) within

50mm of starting. The design had then extrapolated the refusal to develop stiffness values on which the settlement assessment of the shallow pad foundations for the residential tower were then assessed.

The aim of the additional boreholes was to confirm depth to rockhead and improve the geotechnical data for the shallow bedrock. This can then be used to validate the Stage 3 foundation solution.

3.2.1 Residential Tower

The boreholes within the footprint of the residential tower were located relatively close to Daw Bank. The WSP proposed shallow pad foundation solution requires the foundations to be founded within the bedrock. The available information indicated that the rockhead dropped toward the Mersey River though there were no boreholes on the northern elevation of the residential tower to confirm the depth to rockhead.

RUK considered that to validate the proposed shallow foundation solution for the residential tower, additional boreholes should be advanced on the northern elevation to assess the depth to rockhead and carry out HPD tests to assess the bedrock strength and stiffness.

RUK proposed 5No. HPD tests as part of this supplementary GI. Only the very first test was successful with the other tests cancelled due to instability of the test pocket. The instability of the test pockets is not unexpected as they were formed at the very top of the bedrock which is typically weathered and weak. This does not necessarily imply that excavations within the materials will be unstable. The equipment required to undertake a HPD test is relatively expensive (~£60K for the ground probe) and they're difficult to replace, so there is reluctance from the contractor to place the equipment in a pocket that may collapse. This can damage the equipment and may also trap it in the borehole.

Based on an initial review of the results at site without any detailed data interpretation, the characteristic rock mass modulus for the bedrock of E_m>500MPa is possible. It should be noted that only one test was successfully completed and that this does not form a suitable data set.

The Geobore S drilling system was successful in recovering more of the rock core than in the previous ground investigations. The improved core recovery means that geotechnical testing can be scheduled on the samples that can then be used assess the characteristic parameters for the bedrock. Testing will primarily consist of UCS tests (with stress/strain curves) and point load tests. This can be used to complement the single HPD test completed.

Of the 3No. boreholes advanced on the north elevation of the residential tower, 2No. boreholes (BH401 and BH402) were successfully advanced to their target depth of 20mbgl. BH400 at the western end of the site (Swain St) was terminated at 6.2m bgl. The other boreholes at site had consistently hit bedrock at ~3.0m bgl to 3.5m bgl. To ensure a test pocket for the HPD could be formed as close to the interface with the bedrock as possible, this borehole terminated the CP element at what was anticipated to be ~300mm to 500mm above the bedrock (2.9m bgl). However, when the Geobore-S RC was advanced from 2.9m bgl, the bedrock was not encountered at the anticipated depth. Figure 3 presents the soil and rock retrieved from the RC to a depth of 6.2mbgl. The borehole was terminated at this depth as it collapsed to 5.2m bgl during extraction of the drilling rods and the casing could not be advanced deeper than 4.7mbgl.

Figure 3 - Retrieved soil and rock from BH400.

Core run #1 (2.9mbgl to 4.7 mbgl)			Core run #2 (4.7mbgl to 6.2mbgl)
Cobble (Unknown Origin).	Soft grey sandy Clay (Alluvial).	Weak red brown Mudstone	Rounded pebbles (River Terrace or Glacial Till Origin)

From the retrieved sample, it is inferred that bedrock was not encountered to a depth of 4.7mbgl and unlikely to have been encountered between 4.7mbgl and 6.2mbgl.

3.2.2 Bus Station

Two boreholes were advanced on the northern elevation of the bus station to confirm the depth to rockhead and the suitability of the pile design. Two boreholes in the previous ground investigation had been advanced in the vicinity though one was terminated when it encountered a void (presumed to be the Ø1800mm sewer).

The boreholes show that rockhead was encountered at ~3.0m bgl at both locations (BH403, BH404). Improved core recovery will permit a broader range of tests on the in-situ rock.

BH400 was part of the GI works to validate the residential tower Stage 3 foundation proposal. However, the results also inform the bus station foundation solution. Of note is BH400 adjacent to Swain Street where the depth to bedrock was not proven (see section 3.2.1). It can be readily expected that the ground profile observed there may extend north to the bus station pile group PC-3 or further.

3.3 Window Samples

Three window sample boreholes were advanced to a maximum depth of 3.4mbgl to inform on the contamination risk, and to generally provide coverage of exploratory hole locations in the centre of the site. All of the window sampling locations have been installed with standpipes and fitted with ground gas and groundwater taps to allow for subsequent environmental monitoring at the site.

The encountered geology generally comprised of approximately 1.2m of Made Ground composed of pinkish grey sandy angular to sub-angular fine to coarse gravel of limestone. This was underlain by medium dense greyish brown sandy Silt or silty fine Sand. One location has recorded the presence of very soft dark brown organic clay. Rockhead was not encountered in these locations, however medium dense to very dense brown and orange brown very gravelly fine to coarse sand is noted from approximately 1.8mbgl.

No groundwater was encountered during the advancing of the window samples. No visual or olfactory evidence of contamination has been noted. Further details on contamination are discussed in Section 4.4.

3.4 A6 Viaduct Foundation Assessment

Hand pits have been excavated adjacent to the A6 Viaduct foundations to assess their below ground profile for the WSP and RUK ground investigations. The results on the hand pits are summarised in Table 1.

Table 1 - A6 Viaduct foundation hand pit summary.

<table>
<tr><th>Ground Investigation</th><th>Hand Pit</th><th>Location</th><th>Existing Ground level (m OD)</th><th>Depth of Hand Pit (m)</th><th>Top of Foundation (mbgl)</th><th>Projection from face (m)</th></tr>
<tr><td rowspan="5">Geotechnics 2020 (RUK)</td><td>HP01</td><td>TBC</td><td>TBC</td><td>0.60m bgl</td><td>0.35m bgl</td><td>1.0m</td></tr>
<tr><td>HP02</td><td>TBC</td><td>TBC</td><td>0.90m bgl</td><td colspan="2">None observed</td></tr>
<tr><td>HP03</td><td>TBC</td><td>TBC</td><td>1.0m bgl</td><td>0.8m bgl</td><td>1.0m</td></tr>
<tr><td>HP04</td><td>TBC</td><td>TBC</td><td>0.15m bgl</td><td colspan="2">Concrete obstruction</td></tr>
<tr><td>HP05</td><td>TBC</td><td>TBC</td><td>0.80m bgl</td><td>0.4m bgl</td><td>0.1m</td></tr>
<tr><td rowspan="4">Geotechnics 2020 (WSP)</td><td>TP301</td><td>Bus exit arch (NW)</td><td>43.57m OD</td><td>1.2m</td><td>1.2m bgl</td><td>0.25m</td></tr>
<tr><td>TP302</td><td>Bus exit arch (SW)</td><td>43.64m OD</td><td>1.2m bgl</td><td>0.8m bgl</td><td>0.25m</td></tr>
<tr><td>TP303</td><td>Bus exit arch (SE)</td><td>44.16m OD</td><td>1.2m bgl</td><td>0.25m bgl
1.20m bgl</td><td>0.55m
0.70m</td></tr>
<tr><td>TP304</td><td>Bus exit arch (NE)</td><td>43.92m OD</td><td>1.2m bgl</td><td>0.20m bgl</td><td>0.2.0m</td></tr>
</table>

4 Impact on Design

4.1 Residential Tower

The WSP Stage 3 foundation solution for the residential tower consists of shallow pad foundation founded on the top of the bedrock. Bedrock levels in the WSP Stage 3 design had been inferred from interpolating ground levels between the completed GI locations. Bedrock characteristic parameters for the shallow bedrock layers were based primarily on the SPT data due to poor core recovery.

The single HPD test successfully completed inferred that the rock mass stiffness parameters adopted for the Stage 3 design were conservative. However, as only one test was successfully completed, this does not represent a data set that can be relied for design. The improved core recovery using Geobore S will permit UCS testing with stress / strain curves to help develop the characteristic rock mass stiffness of the bedrock. Combined results from the HPD, UCS tests and information collected to date will be used to interpret the characteristic parameters of the bedrock to inform the design and validate the WSP Stage 3 design proposals.

As part of the WSP Stage 3 design validation, the depth to bedrock was assessed at the northern elevation of the residential tower to ensure it was consistent with the assumed profile. The locations for the GI boreholes will be surveyed though the approximate ground levels have been derived from existing information. The ground investigation locations are presented in Figure 4 with a summary of ground conditions in Table 2.

Figure 4 - Ground investigation locations from Aecom, WSP and RUK GI under the residential tower. (extract from WSP drawing 14113-WSP-SKZ-ZZ-DR-Y-0001). Refer to Table 2 for details on individual boreholes. Approximate extent of residential tower footprint shaded green.

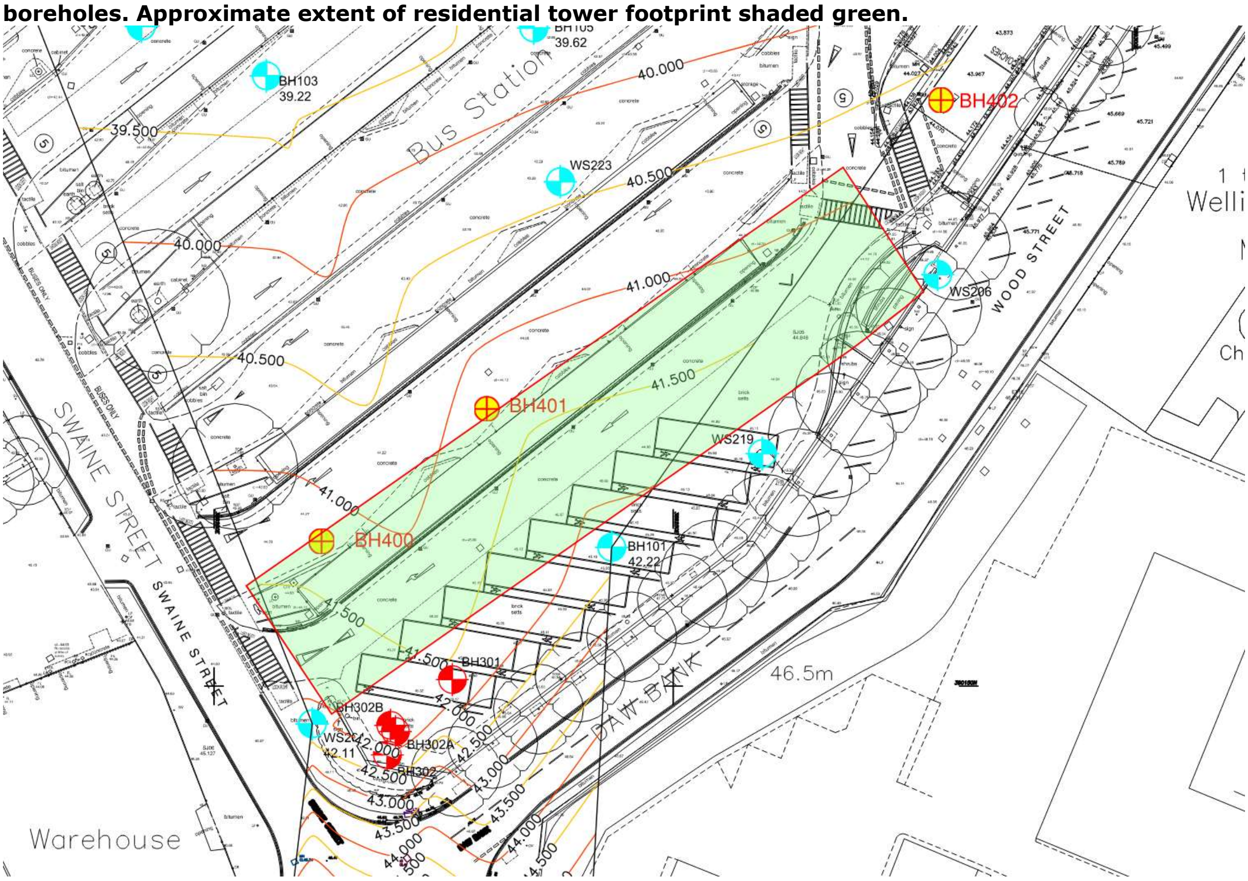


Table 2 - Ground and rockhead level summary for boreholes underlying the residential tower

Ground Investigation	Borehole	Location	Existing Ground Level (m OD)	Depth to bedrock (m bgl)	Rockhead Level (m OD)
Geotechnics 2020 (RUK)	BH400	North Elevation (west)	44.3m OD	Not encountered (EOB @6.2m bgl)	Below 38.1m OD
	BH401	North Elevation (central)	44.2m OD	3.3m bgl	40.9m OD
	BH402	North Elevation (east)	44.0m OD	3.2m bgl	40.8m OD
Geotechnics 2019 (WSP)	BH301	South podium (west)	45.37m OD	4.0mbgl	41.37m OD
	BH302	South podium (bridge)	45.64m OD	Terminated on obstruction @0.6mbgl	
	BH302A		45.47m OD	3.8m bgl	41.67m OD
	BH302B		45.47m OD	4.0m bgl	41.47m OD
Geotechnics Aecom (2016)	BH101	South podium (central)	45.22m OD	3.0m bgl	42.22m OD
	WS201	South podium (west)	45.61m OD	3.50m bgl	42.11m OD
	WS219	South podium (east)	45.13m OD	Not encountered at EOB = 2.0mbgl	NA
	WS206		48.13m OD	Terminated on obstruction @2.34m bgl	

Figure 5 presents the lower foundation layout with two sections (S-01, S-02) highlighted. S-01 would intersect with the location of BH401 and S-02 would intersect with BH400. Figure 6 and Figure 7 present the sections with the depths to rockhead noted. From the sections it can be seen that the depth to rockhead in Figure 6 is close to that predicted by WSP. However, in Figure 7, it can be seen that rockhead has been interpreted as not encountered. The founding soils at that location could not be confirmed due to the poor recovery.

Figure 5 - Extract from WSP drawing 14113-WSP-SKR-F2-DR-S-2000 with location of sections highlighted (S-02 at left of image)

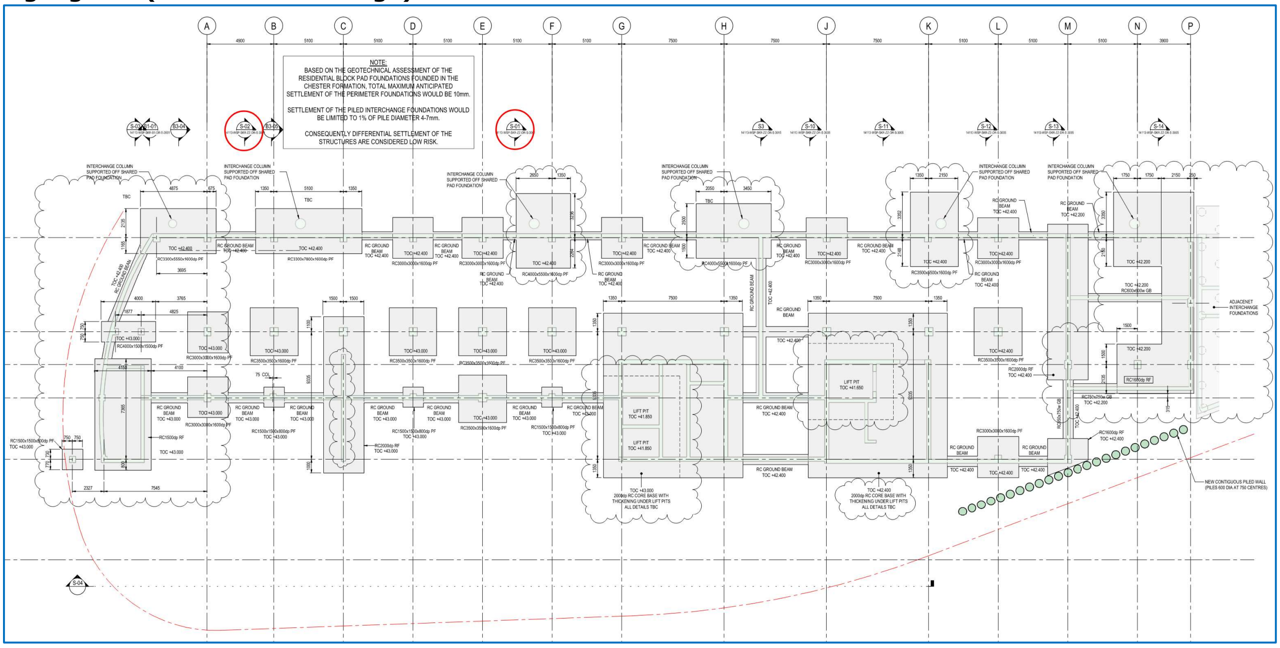


Figure 6 - Section S-01 from WSP drawing 14113-WSP-SKR-ZZ-DR-S-3001 taken at approximate location of BH401. Rockhead level in BH401 at ~40.9m OD with base of foundation at 40.8m OD.

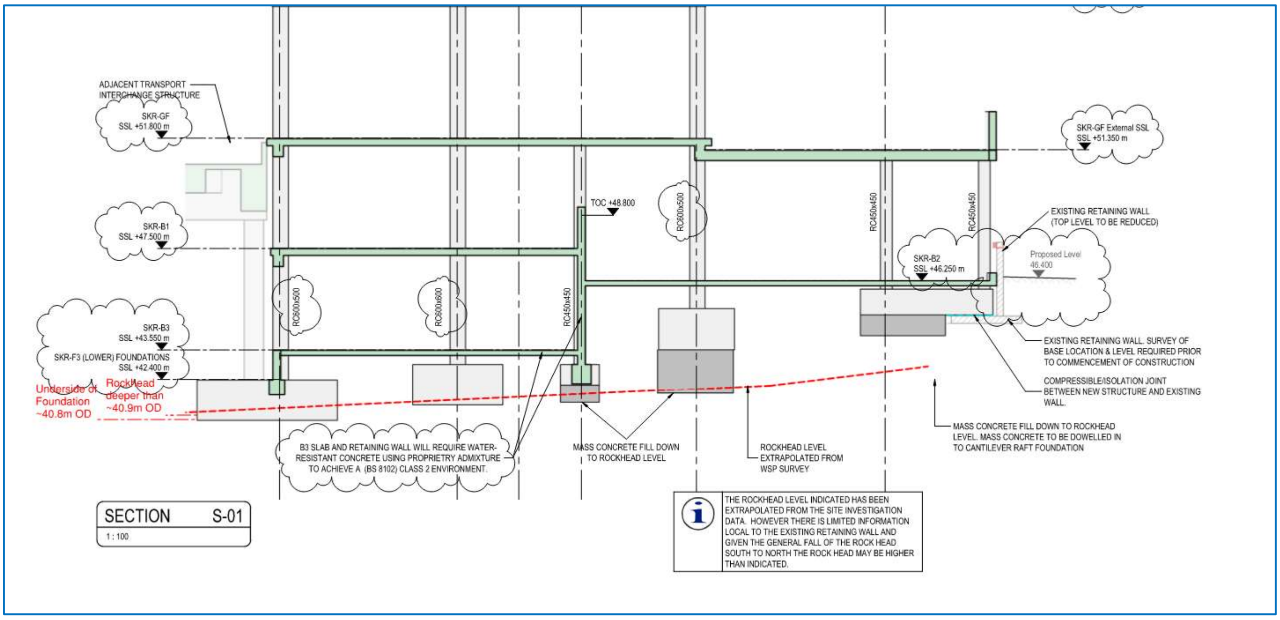
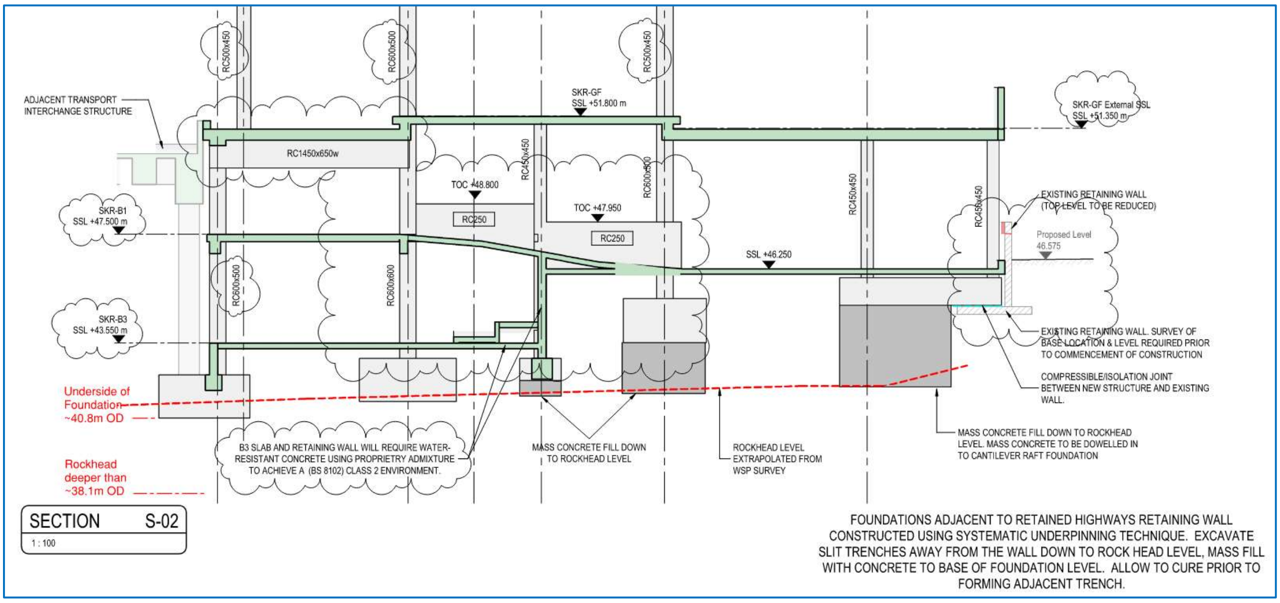


Figure 7 - Section S-02 from WSP drawing 14113-WSP-SKR-ZZ-DR-S-3001 taken at approximate location of BH400. Rockhead level in BH400 was not proven though is anticipated to be deeper than 38.1m OD with base of foundation at 40.8m OD.



Based on the information from the supplementary ground investigation, Ramboll considers that the WSP Stage 3 proposed shallow foundation solution in the vicinity of BH400 is not viable without some form of ground treatment. This would most likely require the excavation of all unsuitable materials under the footprint of each foundation affected and replace with a lean mix concrete.

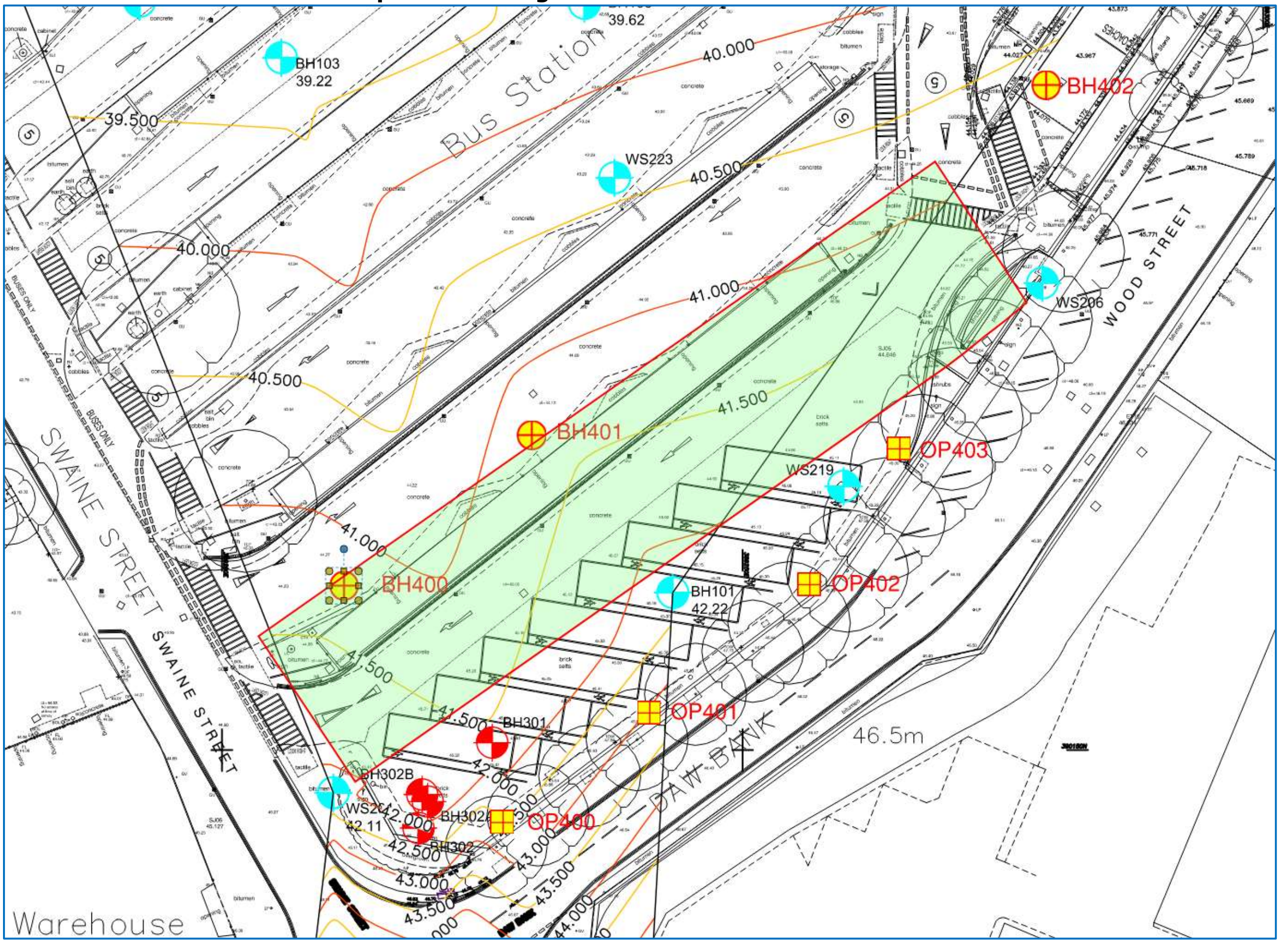
The extent of unsuitable materials cannot be determined at this stage without extensive additional ground investigation, which is unlikely given the operational requirements of the bus station, and this therefore represents a significant risk to the project. It is therefore recommended that one additional cable percussion borehole is advanced next to BH400 to confirm the depth to bedrock and the geological profile. The CP drilling method will allow full recovery and profiling of the soil at this location. This can then confirm that the shallow foundation solution is invalid without some form of ground treatment. There is a small though not insignificant risk that the superficial deposits are anthropogenic in nature and may present a contamination risk (potential cellar for instance). The CP borehole will permit sampling for environmental purposes and allow for the potential installation of a monitoring well for groundwater and ground gas if required.

Should the shallow foundation solution prove not viable, then the CP borehole will also inform the design profile for any piled foundation solution that may need to be considered as an alternative. The design of the adjacent interchange piled foundations (PC-3, 14113-WSP-SKZ-F1-DR-S-2030) may also be affected by the ground conditions. Should this be the case, the piles will need to be advanced deeper to ensure the design resistance is achieved. An additional borehole could also be considered at this pile cap, though the risk to the design strategy is considerably less for this foundation solution than for the shallow foundation solution.

4.2 Daw Bank Retaining Wall (DBRW)

DBRW supports the footpath running along Daw Bank. The OPs were excavated only along the section of DBRW accessible at the time of investigation as shown in Figure 8. OP402 was not excavated due to access restrictions at the time and the likelihood that the kerbs would impede the excavation. Of the three pits completed, only OP403 was able to assess the foundation material which consisted of Made Ground (Pinkish brown sandy angular fine to coarse gravel of limestone). The other locations were restricted by the pavement kerbs.

Figure 8 - Ground investigation locations from Aecom, WSP and RUK GI under the residential tower. (extract from WSP drawing 14113-WSP-SKZ-ZZ-DR-Y-0001). OP pits shown along DBRW. Approximate extent of residential tower footprint shaded green.



A preliminary stability assessment has been undertaken assuming Rankine earth pressures based on an existing retained height of ~1.4m and cover to the top of the DBRW toe ~0.6m. A maintenance surcharge load to the footpath of 5kPa has been adopted. Based on this assessment there is a risk that DBRA will not be Eurocode compliant if the passive ground level is reduced to the base of the retaining wall. The assessment indicates that the Factor of Safety (FoS) is greater than one though the Eurocode overdesign ratio is less then one for DA1-DC2 design case. To maintain stability the active ground level could be reduced and loads restricted on the active side of the wall. A detailed assessment will need to be undertaken assessing additional cross sections.

The Stage 3 WSP design drawings state that the shallow foundations supporting the southern podium require underpinning. The note on the cross-section drawing (Figure 7) states the following:

Foundations adjacent to retained highways retaining wall constructed using systematic underpinning technique. Excavate slit trenches away from the wall down to rock head level, mass fill with concrete to base of foundation level. Allow to cure prior to forming adjacent trench.

Given the granular nature of the made ground materials underlying DBRW, it is considered that the proposed slit trenches are at risk of collapse during excavation and would most likely require trench shields. These would not prevent collapse of materials on the short side of the slit trench that could lead to undermining of DBRW. Undermining of DBRW will decrease the stability further and would need further assessment once the geometry of the slit trenched is confirmed. WSP have also noted that piled foundations may be adopted if underpinning the foundations is not considered viable.

4.3 Bus Terminal

The WSP Stage 3 foundation proposals for the bus terminal consists of piled foundations with a ground bearing slab and pavement. Additional boreholes were undertaken by RUK to confirm the depth to rock head in the central and eastern sections of the northern elevation of the bus terminal building. The locations of the boreholes are presented in Figure 9. The location of the supplementary GI locations and references will need to be confirmed with Geotechnics.

Figure 9 - Ground investigation locations from Aecom, WSP and RUK GI under the bus terminal. (extract from WSP drawing 14113-WSP-SKZ-ZZ-DR-Y-0001).

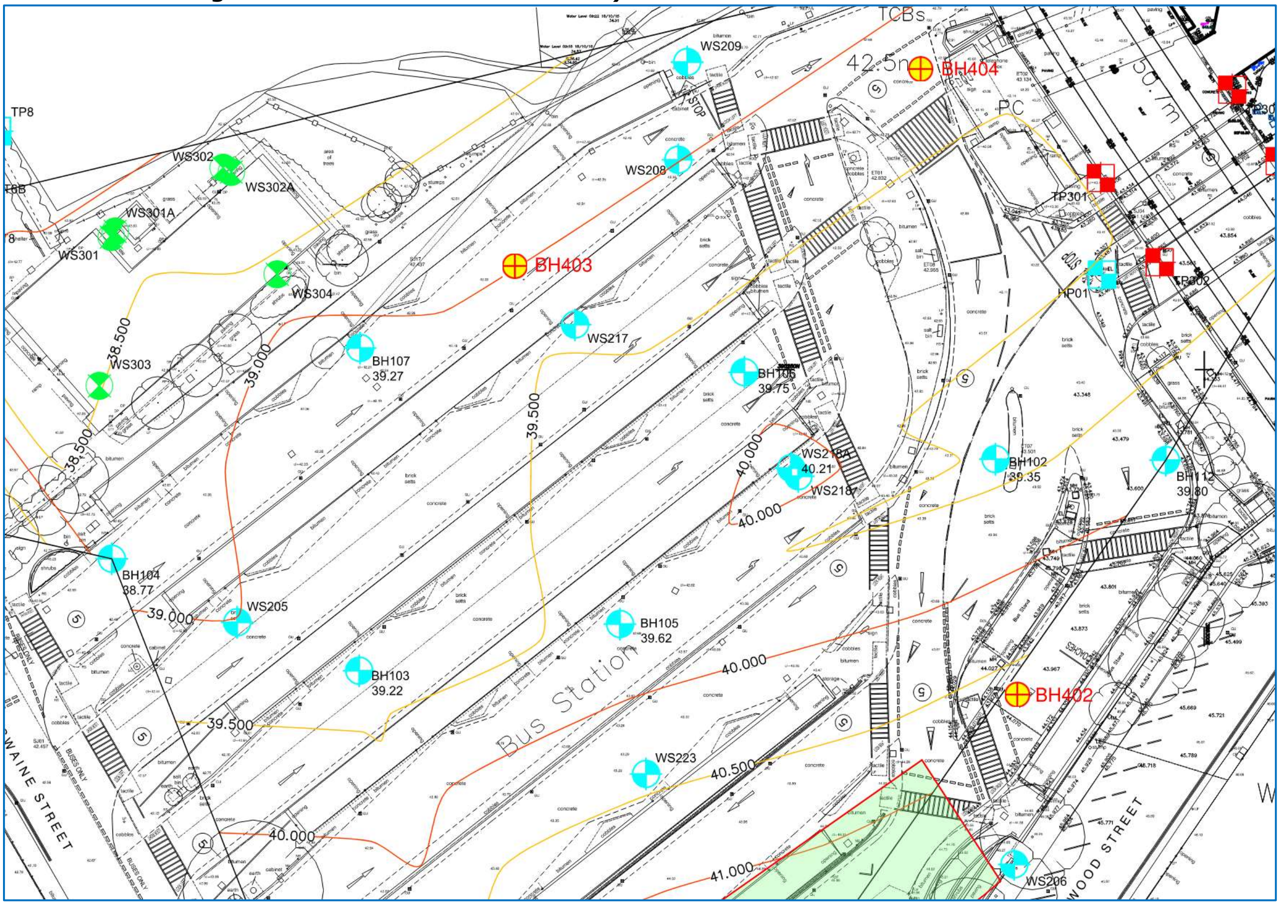


Table 3 - Ground and rockhead level summary for boreholes underlying the bus terminal.

Ground Investigation	Borehole	Location	Existing Ground Level (m OD)	Depth to bedrock (m bgl)	Rockhead Level (m OD)
Geotechnics 2020 (RUK)	BH402	Residential Tower North Elevation (east)	44.0m OD	3.2m bgl	40.8m OD
	BH403	Northern Concourse (central)	42.2m OD	3.0m bgl	39.2m OD
	BH404	Northern Concourse (east)	42.7m OD	2.8m bgl	39.9m OD
Geotechnics Aecom (2016)	BH102	Southern Concourse	43.35m OD	4.0m bgl	39.35m OD
	BH103	Bus Parking (west)	42.42m OD	3.2m bgl	39.22m OD
	BH104	Northern Concourse (west)	42.47m OD	3.7m bgl	38.77m OD
	BH105	Bus Parking (south)	42.62m OD	3.00m bgl	39.62m OD
	BH106	Bus Parking (east)	42.45m OD	2.70m bgl	39.75m OD
	BH107	Northern Concourse (central)	42.27m OD	3.00m bgl	39.27m OD
	BH112	Southern Concourse	43.70m OD	3.90m bgl	39.80m OD

The depth to rockhead for the two additional boreholes is consistent with boreholes previously completed within the footprint of the bus terminal. However, the ground conditions observed in BH400 may affect the pile design for the south west section of the bus terminal, in particular pile group PC-3. This would most likely require the piles to be extended to achieve a minimum rock socket within competent bedrock. No significant change to the pile layout is expected though.

4.4 Contamination

Summary of the preliminary findings of potential contamination discussed within this section are made with the laboratory results available to date. This excludes any ground gas and groundwater monitoring; groundwater sampling and subsequent chemical analysis; and chemical analysis of hand pit samples HP01 to HP05, and one sample from BH402 which are yet to be received.

Evidence of visual potential contamination was noted within the hand pit location. Ash was noted within HP01 to HP03. Gravel and cobble of glassy vitrified tar was recovered from hand pit HP01 (and recorded at an approximate depth of 0.50mbgl). The chemical results from the hand pits are currently outstanding. No visual or olfactory evidence of potential contamination was recorded within the borehole records.

Twenty-four samples were scheduled for Suite E analysis and asbestos identification at a certified laboratory. Six of these were further scheduled for volatile and semi-volatile organic compounds testing (VOCs and SVOCs). Additional testing also includes polychlorinated biphenyls (PCBs).

Preliminary screening of the soil data received to date is summarised below:

- Metal concentrations are recorded above the limit of detection (LOD), however the concentrations are likely to be insufficiently elevated to pose a significant human health risk;
- Cyanide recorded below the LOD across the site;
- Polyaromatic hydrocarbons (PAHs) are recorded either below the LOD, or at low concentrations across the site;
- Aliphatic and aromatic total petroleum hydrocarbons (TPH) are recorded either below the LOD, or at low concentrations towards the southern boundary of the site. Heavy end, and therefore less mobile TPH fractions noted within observation pits OP401 and OP403. The concentrations are likely to be insufficiently elevated to pose a significant human health risk;
- BTEX hydrocarbons (benzene, toluene, ethylbenzene and xylenes) are all recorded below the LOD across the site;
- VOCs are all recorded below the LOD across the site;
- SVOCs are generally all below the LOD, with exception of low concentrations of carbazole, dibenzofuran and 2-methylnaphthalene in two soil samples;
- Asbestos was not identified within any of the analysed soil samples;
- Alkaline and strongly alkaline pH in boreholes and observation pits samples;
- PCBs below the LOD in scheduled samples; and
- Photoionisation Detector (PID) concentrations of volatile organic compounds ranged between the limit of detection (<0.1ppm) and 0.1ppm.

Further assessment is required upon the receipt of all data; including ground gas and groundwater monitoring and groundwater chemical analysis.

A Contaminated Land Interpretative Report is required to assess any risks associated with potential contaminants in the ground and groundwater in accordance with current UK legislation and guidance.

4.5 Buildability

Adjacent to the A6 Viaduct is a sewer that in the current WSP Stage 3 design will be maintained. Piled foundations are required to support the overlying structure. The Stage 3 WSP foundation proposal for the pile layout is presented in Figure 10. The section and elevation of the sewer are presented in Figure 11 and Figure 12 respectively.

Figure 10 - Pile and Pile cap layout over sewer and adjacent to the A6 Viaduct. (14113-WSP-SKZ-F1-DR-S-2040)

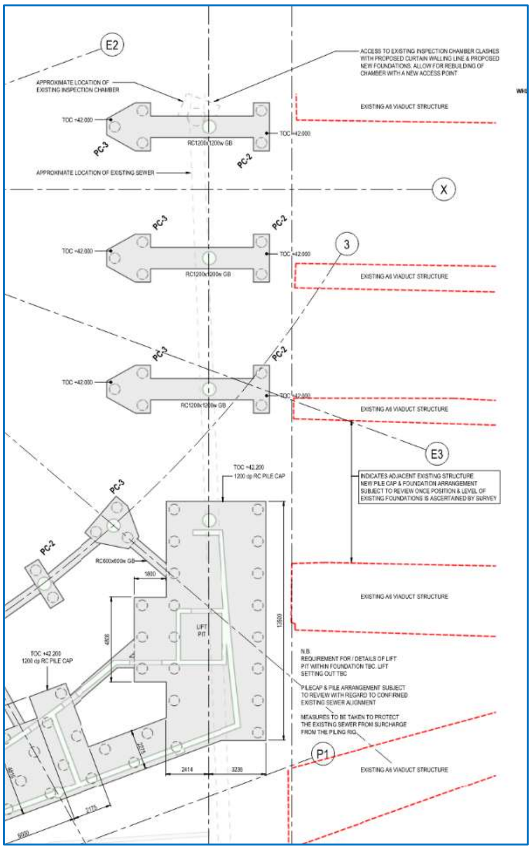


Figure 11 - Section through foundation arrangement over sewer and adjacent to the A6 Viaduct (14113-WSP-SKZ-F1-DR-S-3001).

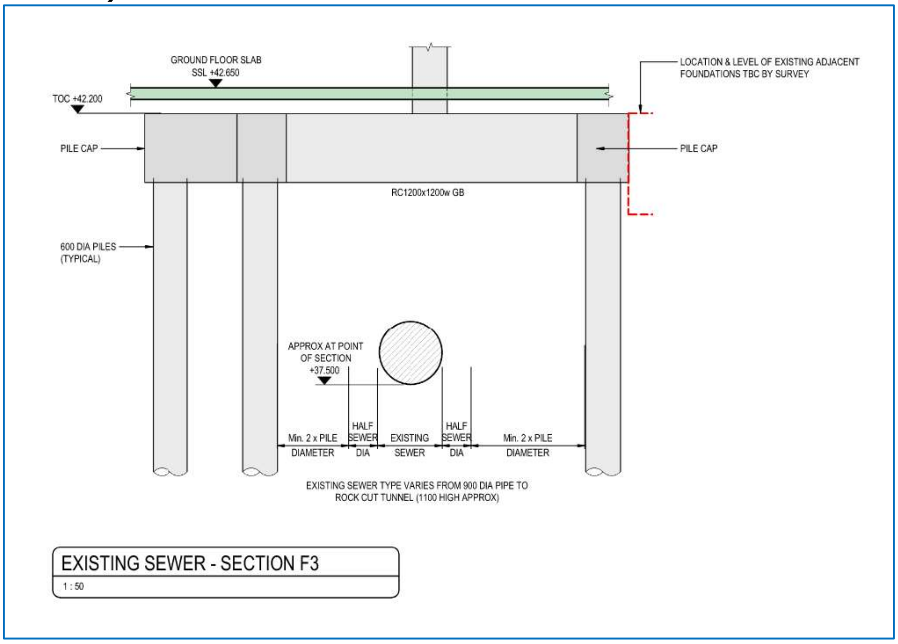
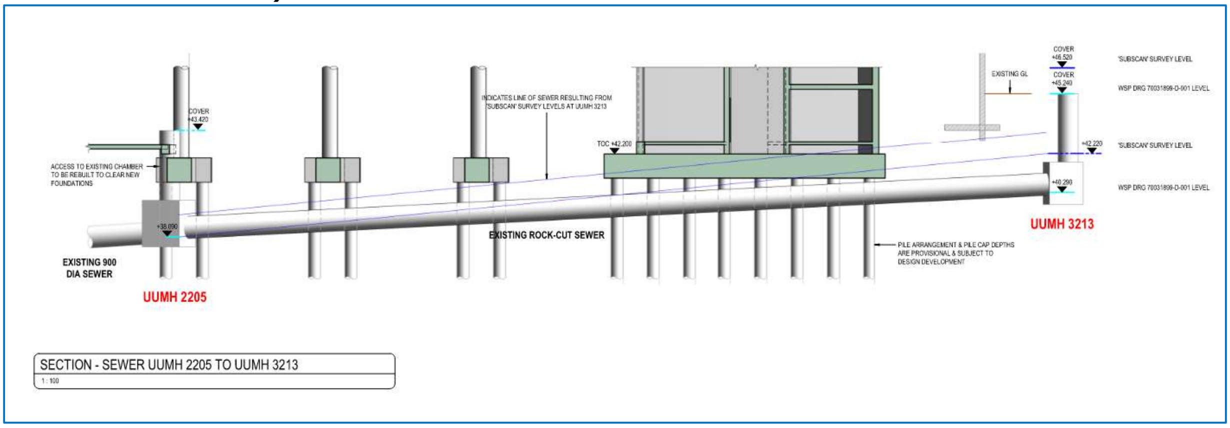


Figure 12 - Elevation of Sewer relative to piled foundations for sewer adjacent to the A6 Viaduct (14113-WSP-SKZ-F1-DR-S-3002).



WSP have provided offset guidance from the rock cut sewer and rely on the pile caps being cast against the A6 Viaduct foundations to achieve the offset. From the available information, it appears that the sewer runs closer to the A6 Viaduct than the drawings show. The alignment and elevation of the sewer needs to be confirmed before the WSP Stage 3 foundation solution can be validated for the area.

Also of concern is the profile of the A6 Viaduct foundations. The handpits excavated to assess the foundation profile below ground infer that it projects up to 1.0m from the face of the above ground pier. It is unclear where the hand pits were excavated at this stage and further information is currently being sought from Geotechnics. However, it can be inferred that the projection will inhibit the installation of the piled foundations and the construction of the pile caps.

5 Conclusion

The objectives of the ground investigation were to validate the WSP Stage 3 foundation and ground treatment proposals for the Stockport Bus Interchange.

For the residential tower, the ground investigation was targeted to validate the shallow foundation proposal. This required the depth to rockhead and the shallow rock characteristic design parameters to be assessed. The boreholes advanced on the northern elevation of the tower have shown that at the western end (Swain St end) the ground conditions were inconsistent with the interpolated rockhead levels. Rockhead was not proved though is inferred to be at least 3.0m deeper locally than shown on the WSP drawing 14113-WSP-SKZ-ZZ-DR-Y-0001. Further geotechnical testing is required to confirm the suitability of the characteristic design parameters though from the available data, the values used by WSP in their Stage 3 design appear to be conservatively appropriate.

For the bus terminal, the additional boreholes advanced on the northern elevation confirmed that the rockhead levels were consistent with the rest of the site. When considering the boreholes undertaken for the residential tower, there is a risk that the variation in rockhead levels noted at BH400 will likely extend to within the bus terminal. Pile group PC-3 is the closest pile group and if the bedrock levels vary as shown in BH400, then it is likely that the piles will need to be extended to form the minimum rock socket.

The unexpected ground conditions encountered during the supplementary GI at BH400 present a significant risk to the viability of the WSP Stage 3 foundation proposals for the project. Hence it is recommended that the ground profile at BH400 be further investigated to confirm the soil profile and depth to rockhead to better inform the foundation solution. A single cable percussion borehole advanced to rockhead would be sufficient. It is anticipated that this would not extend deeper than 8.0m bgl and it would be recommended to be undertaken in conjunction with any supplementary GI to the Bridgescape area.

The DBRW observation pits confirm that the geometry of the constructed wall is consistent with the record drawings. The founding stratum was only observed at one location though appears to be a structural granular fill. A preliminary stability assessment of DBRW suggests there is a risk of the wall being non-Eurocode compliant if the passive soil mass is removed. Undermining of DBRW to place the concrete underpinning for the adjacent residential tower shallow foundations would further compromise the foundation stability.

The foundations for the A6 viaduct protrude from the face of the above ground structure by up toa 1.0m. The current WSP Stage 3 design requires the piles and pile caps to be constructed immediately adjacent to the viaduct to achieve the required offset from the assumed alignment of the sewer. The alignment of the sewer needs to be verified and surveyed to be included within a 3D model of the site, with the viaduct foundations, that can then be used to inform the foundation options.

The location and alignment of tunnels beneath the site was not investigated as part of this stage of works. The proposed geophysical microgravity investigation was considered unlikely to return data that would inform the tunnel locations. Other options are currently being discussed to mitigate the risk to the project.

This technical note will be updated if additional information from the GI laboratory testing further impacts the Stage 3 WSP design. The data collected from the supplementary GI will be incorporated with the previous GI data to develop the ground model. This will be used to inform the Ground Investigation Report and Contaminated Land Interpretative Report.