Phase II Geo-environmental Site Assessment

The Mole
Neptune road
Barry
CF62 5QR

October 2021

Project Number: 413800.0000.0000

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Executive Summary

TRC Companies Limited (TRC) was commissioned by ABP Property Development Company Limited (ABP) to under a Phase II Geo-environmental and Geotechnical Site Assessment at Neptune Rd, Barry, Vale of Glamorgan, Wales CF62 5AQ (hereafter referred to as the 'Site').

This Executive Summary is part of the complete report; and findings, opinions or conclusions in this Executive Summary are made in context with the complete report. TRC recommends that the user reads the entire report for all supporting information related to findings opinions and conclusions.

Executive Summary	
Site Details	
Client	ABP Property Development Company Ltd (ABP)
Site Address & Grid Reference	The Mole, Neptune Road, Barry, CF62 5QR
Site Area	3.2ha
Current Site Use	The Site predominantly comprises vacant land covered in scrub. The eastern section of the Site is occupied by the Barry Water Activity Club (BWAC), which was separated from the remainder of the Site by a metal fence and gated entrance.
Proposed Development	This assessment has considered development proposals comprising a mixed-use scheme of residential dwellings and commercial facilities. The outline design shows that residential dwellings comprise seven buildings with both town houses and apartments, which coveres a majority of the Site. Blocks B, D, E and G were all proposed to be town houses, with Blocks A, C and F apartments. The buildings vary in shaped from rectangular to L-shaped. The apartments were proposed to be a maximum of four storeys and the town houses a maximum of three storeys. The eastern edge of the Site was proposed to be the marina facilities building, which would be a low-rise structure. Some sections of the Site will comprise soft landscaped areas.
Phase I Summary	
Site History	The Site was part of the Cadoxton River estuary until c.1898 when Barry Docks were constructed. Since then the Site has been used for rail sidings and docks until c.1965. From 1975 an oil storage terminal was present on Site. It is believed that the petroleum depot and tanks were demolished sometime between 1991 and 2001, since when the Site has remained vacant. The surrounding land use has been characterised by the port and docks with infrastructure and industrial land uses neighbouring the Site on all sides. Archive reports show that the Site comprises reclaimed land. Reports indicate that the source of the fill used to raise Site levels was won from the excavation of soft alluvium sediments along with materials originating from the surrounding hillsides and beaches.
Initial Conceptual Site Model	The initial conceptual model identified possible sources of contamination associated with on-Site sources such as Made Ground, historic railway sidings and historic oil tank farm.
Investigation Findings	
Ground Conditions	TRCs investigation and historical investigations have identified that the Site is underlain by significant amounts of Made Ground. During TRCs investigation Made Ground was proven to be present to a maximum depth of 12.5mbgl. CPBH01 in the eastern section of the Site displayed the greatest thickness of Made Ground (12.5m) and CPBH02 in the northern section (eastern half) of the Site displayed the shallowest thickness of Made Ground (7.7m) in the locations where the full

	extent of the Made Ground was proven. The Made Ground was generally granular in nature and comprised various fill materials. The Made Ground was underlain by Alluvium, which was generally very soft to soft in composition. The upper surface of the Alluvium was encountered at depths of between 7.7m and 12.5mbgl, and the Alluvium persisted to a maximum recorded depth of 23.0mbgl. Investigation data indicates that the depth to the base of the Alluvium was relatively uniform ranging from -12.3mAOD to -13.6mAOD. The Alluvium was underlain by the Blue Anchor Formation, which was recorded between depths of 20.8m and 23.0mbgl, which persisted to the base of the boreholes.
Groundwater Conditions	During the Site investigation groundwater was encountered between 2.8m and 10.0mbgl. During subsequent gas and groundwater monitoring, groundwater resting levels were recorded between 1.61m and 4.20mbgl. No tidal level monitoring was performed during the scope of this assessment. TRC consider that groundwater beneath the Site may be tidally influenced, but water level in the dock is managed by locks and the range of elevation changes may not be significant. Future design may need to consider potential for tidal influence and further monitoring may be necessary.
Contaminated Land Risk Assessment	
Human Health	There are minor exceedances of the residential (without gardens) GAC for heavy metals and PAH's. The exceedances are limited to Made Ground samples on Site. In addition, asbestos was encountered within four Made Ground samples but the quantification data indicated a concentration of <0.001% (LOD).
Controlled Waters	There are minor exceedances of the EQS and DWS due to elevated metals within groundwater samples across the Site. Given the conservative nature of the EQS and DWS screening criteria and only minor exceedances noted, TRC deem the water quality on Site satisfactory.
Ground Gas Risk Assessment	TRC has assessed the bulk ground gas concentrations in accordance with current guidance (BS8485:2015). Based on the results, a gas screening value (GSV) of 0.165 l/hr was calculated, which would classify the Site as Characteristic Situation 2 (low risk).
Geotechnical Assessment	
Stress History	Development of Barry Docks began in November 1884, with the area opened to traffic in 1889. The Mole was initially used to store coal, with four masonry towers located to the north of the Site. After the 1930s the Site was redeveloped as a tank farm, primarily storing molasses, heavy fuel oil and gas oil. Each of the tanks had a concrete base. The concrete bases and any associated foundations remain in-situ. The tanks were of varying sizes and measured up to approximately 40m in diameter. As the tanks were located in close proximity of each other it is considered that the stress from the imposed load could be approximated to a rectangle measuring 340m by 40m. Assuming that the tanks were approximately 10m high this would mean that the imposed load from the liquid contents was likely to be in the order of 100kPa. At a depth of 10mbgl, within the top part of the Alluvium, the stress reduction is likely to be in the order of 25% and therefore a loading of approximately 75kPa would still be applicable. Assuming average ground conditions comprising granular deposits with an average SPT N value of 12 to a depth of 12m underlain by soft Alluvium with an average undrained shear strength of 30kPa (estimated from an SPT N value of 8) to a depth of 20.8m this could have resulted in an estimated settlement of approximately 300mm.

	This estimate is an outline appraisal of the settlement that may have occurred beneath the tanks. It is noted that the actual nature of the ground conditions directly beneath the tanks is not known due to the presence of relic tank bases. TRC consider that the zone directly beneath the former tanks is likely to represent a zone where settlement of the underlying soils may already have occurred or that these zones are supported on deep foundations. After removal of the tanks, the near surface soils were investigated and due to contaminant concentrations in near surface soils a capping layer was installed in order to break the direct contact pathway. The capping layer was placed over the existing soils at the Site by Ove Arup in 1998. The Health and Safety File (compiled by White Young Green) included a letter to the Local Authority from Ove Arup which provided some further information on the proposed capping layer. This letter suggests that Site levels will be raised to 8.3mAOD (in compliance with Condition No. 20 of the Outline Application) largely using Type 3 material. It should be noted that there is no specific reference to whether the Type 3 material was equivalent to Type 3 as defined by Specifications for Highway Works Series 800. The recent investigation indicated that the capping layer generally comprised Made Ground to a depth of 0.6m overlying what appeared to be a Type 1 gravel layer with a geotextile membrane placed above and below the Type 1 gravel. It appears that the Type 1 gravel layer was designed to act as a capillary break layer. Table 9 in this report details the capping layer overview. The results of the TRC investigation has indicated that there are materials (cobbles and boulders) that do not meet the requirements of a Type 3 material. In addition, much of the capping layer material was described as sandy gravelly clay and therefore has a fines content greater than the 5% limit placed on a Type 3 material. While the specification indicated that the capping layer material would be compacted, details of the compaction specification and/or methods used were not supplied. It should be noted that the presence of cobbles and boulders within the capping material would mean that it would be difficult to achieve a reliable and consistent compaction and therefore this material should not be classified as an engineered material. A topographical survey of the Site indicated that the access road (located on the northern side of the Site) was generally in the order of 8.6mAOD at the western end but fell to approximately 7.5mAOD at the eastern end. The main part of the Site was generally at an elevation of 8.5 to 9.0mAOD. Localised variations were also recorded, refer to Section 7.2.3 for full details, and an image of the Ove Arup 'Earthworks Final Levels Setting Out' drawing updated to 'As Built' has been provided (drawing reference not readable). The topographical survey has indicated that while much of the Site is located above the original flood risk level of 8.3mAOD to be achieved by Ove Arup during the remedial works, there are areas that are lower than this level. The variation of the elevation appeared to be localised and therefore could potentially due to settlement of underlying soils.

	Estimations are that the Site levels were built up by approximately 0.7 to 1.1m during the Ove Arup remediation. Assuming that this imposed an average load of 15kPa across the entire Site (measuring 75m by 375m) it is anticipated that this would result in settlement in the underlying soils of 60mm (assuming average ground conditions comprising granular Made Ground over Alluvium). However, for areas where poorer ground conditions were present (i.e. soft cohesive Made Ground), this could have resulted in settlement in excess of 100mm. It should be noted that soils that have been preloaded from the tanks (assuming supported on a raft foundation) would have resulted in minimal settlement. Therefore, differential settlements in excess of 100mm would have been expected.
Proposed Redevelopment Loading	The proposed development comprises a number of 3 to 4 storey residential blocks of apartments preferably supported on raft foundations. Site levels will need to be raised further in order to achieve a revised flood risk level of 9mAOD. This would mean that in some areas of the Site, levels would need to be raised by 1.5m whereas other areas (located beneath the existing mounds) would need to be cut by approximately 1.5m. Typically, Site levels would need to be raised by approximately 0.5m across the southwestern portion of the Site and by approximately 1m at the eastern end of the Site. This would mean that the imposed loads from building up the Site would vary from 0kPa to up to 30kPa, resulting in additional settlement in the order of 200mm in some areas. This is before the construction of the proposed buildings. Figure 14 provides an indication of the location of the proposed development overlying the current topographical survey and the location of the former tanks. This demonstrates that in the current layout the buildings span across the location of the former tanks, locations where Site levels will need to be increased and areas where levels may need to be cut. It is therefore considered that the risk of differential settlement is likely to be high.
Retaining Walls	Retaining sea walls are located along the northern, eastern and western sides of the mole. Originally these would have been designed to retain soils raised to a level of circa 6m AOD and any additional loadings from the tanks and other on-Site structures. Since the walls were constructed Site levels have subsequently been raised to circa 8.5mAOD. This additional soil will result in additional loads to the retaining walls. To achieve the finished levels, and a flat building platform Site levels adjacent to the retaining walls will need to be increased. In addition to raising Site levels, supporting the proposed structures on a raft foundation will also result in an increase of loadings to the retaining wall. One potential solution for reducing future settlement to enable a raft foundation to be utilised would be to pre-load the soils by use of a surcharge. This method of ground improvement would also result in a further, albeit temporary, load to the retaining wall. It is not known if repair to the retaining wall has been undertaken and whether an assessment of whether the retaining wall is capable of supporting additional loads resulting from the proposed redevelopment.
Foundation Recommendations	It is considered that supporting the proposed buildings on a raft structure requires further detailed design and assessment. TRC consider that further assessment should consider the additional loadings to the retaining structures that surround the Site and potential for long-term ongoing creep settlement. TRC understands that further detailed appraisal has been performed and reported under separate cover by a third-party geotechnical consultant. These findings should be read in conjunction with this report.

	Other than a raft solution, there are a number of other foundation solutions that could be considered for the proposed development. It is understood that piling the proposed buildings may be applicable. However, there are other potential solutions that could be explored such as use of Controlled Modulus Columns (CMC). With regard to CMC's it will be important that the columns are installed to a depth where competent strata has been identified, which will be circa 20mbgl. Consideration will need to be given to enabling works required prior to installation of CMCs such as removal of concrete bases and inclusion of a granular layer/working platform at surface. For the residential development, the warranty provider should be consulted to ensure their approval of a CMC based foundation design. Budgetary cost estimates for Site remediation and enabling works are provided in Annex H.
Conclusions / Recommendations	
Whilst the TRC investigation detected elevated heavy metals, PAH and asbestos in soils, it is considered that requirements for remediation will be reduced through the development design that will address active risk pathways to future site users. Design mitigation will include placement of hardstanding (i.e. building footprints, roadways etc.) and clean capping in areas of landscaping. Clean capping should comprise a minimum of 300mm of clean cover should be placed above a geotextile marker layer. Verification of the cover system and chemical testing of the imported clean cover soils will be required by a suitably qualified environmental consultant. Minor concentrations of heavy metals within groundwater are not considered to present a significant risk to controlled waters. As such, no active remediation is considered necessary. The gas regime on Site is classified as Characteristic Situation ,2 for which basic gas protection measures are required. Potential for tidal influence may require further assessment to aid the assessment of ground gas risks. The feasibility of utilising a raft foundation for the proposed structures has been assessed by others and is believed to be suitable for the proposed 3 to 4 storey apartment buildings. As well as a raft foundation solution there are other potential solutions that could be explored such as the use of Controlled Modulus Columns (CMC). Should a CMC solution be selected it will be important that the columns are installed to a depth where competent strata has been identified which will be circa 20mbgl. Consideration will need to be given to enabling works required prior to installation of CMCs such as removal of concrete bases and inclusion of a granular layer/working platform at surface.	

1.0 Introduction

1.1 Purpose

TRC Companies Limited (TRC) was commissioned by ABP Development Company (the 'Client') to undertake a Phase II Geo-environmental and Geotechnical Site Investigation at The Mole, Neptune Road, Barry, CF62 5QR (hereafter referred to as the 'Site').

A Site location plan is presented as Figure 1 in Annex A.

The purpose of this Phase II Geo-environmental Site Assessment was to undertake an environmental and geotechnical investigation to support the proposed residential development of the Site. This Phase II Geo-environmental Assessment used intrusive investigation methodologies to aid Site characterisation and to inform the Client of potential environmental liabilities and geotechnical parameters.

This report has been prepared to aid the sale, planning and design of the Site. The geotechnical assessment will assess the feasibility of the most suited foundation solution for the Site.

1.2 Proposed Development

The proposed development concept supplied to TRC by the Client at the time of reporting was a mixed-use scheme, which comprised residential dwellings and the commercial facilities. The residential dwellings comprised seven buildings with both town houses and apartments, which covered a majority of the Site. Blocks B, D, E and G were all proposed to be town houses, with Blocks A, C and F apartments. The buildings vary in shaped from rectangular to L-shaped, the apartments were proposed to be a maximum of four storeys and the town houses a maximum of three storeys. The eastern edge of the Site was proposed to be the marina facilities building, which would be a low-rise structure. Some sections of the Site will comprise soft landscaped areas (communal – without the consumption of homegrown produce).

It is understood that during the remediation works undertaken by Ove Arup, that Site levels were increased to circa 8.3m AOD, for flood protection reasons. It is understood that in order to take account the potential effects of climate change Site levels now need to be increased further to 9m AOD.

The original assessment undertaken by Ove Arup suggested that for lightly loaded structures, that a raft foundation solution could be adopted. The use of a raft foundation is the preferred foundation solution, if deemed appropriate.

The Proposed Development Plan is presented as Figure 2 in Annex A.

1.3 Scope of Services

This report presents the findings of a Phase II Geo-environmental Site Assessment, based on the following information:

- Current use and condition of the Site;
- Environmental setting in terms of geology, hydrogeology, hydrology and surrounding land uses; and,
- Intrusive investigation including environmental and geotechnical sampling and testing.

The Phase II assessment was conducted with due regard to the following guidance:

- The National Planning Policy Framework;
- BS10175 (2017) Investigation of Potentially Contaminated Sites – Code of Practice;
- BS5930 (2020) Code of Practice for Ground Investigations;
- Land Contamination: Risk Management (LCRM)

- BS8485 (2019) Code of Practice for the Design of Protective Measures for Methane and Carbon Dioxide Ground Gases for New Buildings; and
- BS8676:2013 'Guidance on Investigations for Ground Gas – Permanent Gases and Volatile Organic Compounds (VOCs).

1.4 Significant Assumptions

This report presents TRC's observations, findings, and conclusions as they existed on the date that this report was issued. This report is subject to modification if TRC becomes aware of additional information after the date of this report that is material to its findings and conclusions.

The reliability of information provided by others to TRC cannot be guaranteed to be accurate or complete. Performance of this Phase II Geo-environmental Site Assessment is intended to reduce, but not eliminate, uncertainty of environmental conditions associated with the subject Site; therefore, the findings and conclusions made in this report should not be construed to warrant or guarantee the subject Site, or express or imply, including without limitation, warranties as to its marketability for a particular use. TRC found no reason to question the validity of information received unless explicitly noted elsewhere in this report.

1.5 User Reliance

This report was prepared for ABP Development Company Limited. Reliance on the Report by any other third party is subject to requesting and fully executing a reliance letter between TRC and the third party that acknowledges the TRC Standard Terms and Conditions with the Client, to the same extent as if they were the Client thereunder.

TRC has been provided with information from third parties for information purposes only and without representation or warranty, express or implied as to its accuracy or completeness and without any liability on such third parties part to revise or update the information. Where reliance has been provided by third parties to potential purchasers this is noted in our report.

2.0 Site Description

2.1 Site Location and Description

The Site comprised an approximate 3.2ha plot of land centred on National Grid Reference 311525, 167325, which was spur of land jetting out into the Barry Docks. The Site was located approximately 900m south of Barry Town. The Site was on a jetty which was approximately 400 meters in length. A Site location plan is presented as Figure 1 in Annex A.

The Site was located to the east of Neptune Road and accessed via a gate which was located on the western edge of the Site. The Site is derelict and occupied by vacant land covered in scrub. The eastern section of the Site was occupied by the Barry Water Activity Club (BWAC), which was separated from the remainder of the Site by a metal fence and gate. A track led along the northern side of the Site from the gate located in the west. The northern, eastern and western boundaries of the Site were defined by retaining structures. A water outfall was located in the northern section of the Site.

There was portable accommodation on-site along with shipping containers used for storage. A large bund was present along the western boundary of the BWAC.

The Site has an average elevation of approximately 8m to 9m above ordnance datum (AOD), with localised variations of approximately 7mAOD to 11.5mAOD. Topographically the Site was variable. The central and western section of the Site was approximately 8.5mAOD to 9.0mAOD, excluding the track in the northern section which was 7.5mAOD to 8.0mAOD. The area of the BAWC was generally 8.5mAOD; however, a bund in the western section of this area of the Site was approximately 11.5mAOD and 9.0mAOD. The grassland area to the south of the access road is at a higher elevation.

2.2 Subject and Surrounding Area

The Site is located within an area of predominantly commercial and residential land use. Land uses in the immediate vicinity include the following principal features:

Table 1: Summary of Surrounding Land Use

Direction	Land Use
North	To the north of the Site was approximately 140m of open water belonging to the Barry Docks, with residential properties and shopping centres beyond.
East	To the east of the Site was approximately 500m of ocean, with the port entrance beyond.
South	To the south of the Site was approximately 100m of ocean, with residential properties beyond.
West	To the west of the Site was Neptune Road and residential properties, with Asda beyond.

2.3 Previous Environmental Assessment, Investigations or Remediation

The following table presents a summary of the previous information made available for review by the Client at the time of reporting. The following table includes a summary of key information only. For full details refer to each of the individual reports. TRC has utilised this information to inform the report, as detailed in later sections.

The Site was historically located within a sea inlet of the Cadoxton River, which is underlain by mudflats of the Cadoxton Estuary. The Site (i.e. The Mole) was originally formed on reclaimed land using natural material from excavations within the Cadoxton Estuary. Development of the Barry Docks began in November 1884 and the area was opened to traffic in 1889.

Table 2: Summary of Previous Site Assessments

Report Title	Summary of Findings
Reclamation of Tank Farm and Mole at Barry No. 1 Dock, Factual Report on Ground Investigation (Exploration Associates Report Reference 154061 dated August 1994)	The report details that a trial pitting exercise was undertaken at the No. 1 Dock, with subsequent laboratory analysis undertaken. It should be noted that the No 1 Dock is located to the west of the Site and therefore the report includes information that is not specific to the Site. In addition, this factual report was subsequently used to inform assessments undertaken by Ove Arup, described in further detail below. The ground conditions were found to comprise Made Ground to depths of 0.6m and 3.5m. it should be noted that full details were not included in the report provided to TRC. The 0.6m of Made Ground referenced is likely to be on the No.1 Dock and therefore offSite. The Made Ground was underlain by probable Alluvium, which was detailed as soft to firm grey locally silty and sand clay. The report noted that groundwater was encountered in a majority of the exploratory holes at depths between 1.4m and 3.6m. Perched water was also recorded at shallower depths.
Site Investigation Report No. 5 (Phase III), Site 'A' and Mole (Ove Arup and Partners Report, Reference 94/2585, dated October 1994)	The purpose of this investigation was to provide further information on the nature of the soils across the Site, via soil and groundwater chemical analysis. The report aimed to detail the degree of contamination at the Site and potentially identify areas of spillage that may have occurred during removal of the tanks on the Site. The report refers to Site A and the Mole. The following information relates to the Mole only. Site A is located to the west of the Site. Ove Arup provided a finished level drawing, which has been used to inform this assessment and is discussed in later sections. The report detailed that the Site lies within the axis of a bedrock valley. That the Site was underlain by the Triassic Blue Anchor Formation, which overlies Mercia Mudstone. It was noted that the Site lies within the former Cadoxton Estuary, therefore it was detailed that Alluvium should be anticipated at the Site. The report provides details of the tank foundations. At the time of the investigation it was noted that all tank structures and buildings had been demolished. Limited details were provided about the tank foundations, but the information did suggest that they were concrete ring foundations infilled with rolled hardcore topped with sand-bitumen and concrete raft foundations on rolled hardcore fill with sand-bitumen above the raft held in by a brick ring. The ground conditions were detailed to comprise Made Ground to ranging to depths of 1.5m to 2.6m. It should be noted that TRC do not agree with this assessment given that as the Site was historically part of the estuary, Site levels would need to have been raised by in excess of 6m in order to construct the mole. The report indicated that the Made Ground was underlain by Alluvium / Alluvial Fill. No clear distinction was made between the natural Alluvium and what has been interpreted as Alluvial Fill. The bedrock was encountered at between 22.6m and 26.6m at the Site. Groundwater was encountered at depths between 1.8m and 3.1m. The report details that the Site was predominantly contaminated within organic liquid contaminants. The report suggests that the worst soil contamination was limited to the upper 1.5m to 2.0m of soil at the Site. Ove Arup stated that "*Providing any reclamation processes carried out on the Site result in fill densities no less than present values, lightly loaded structures could be founded on shallow foundations or rafts on the fill with allowable bearing pressures limited between 30 and 50kN/m^2. Slightly higher bearing pressures may be acceptable with careful consideration of the effect of settlement on the proposed structures. Heavily loaded structures would require piled foundations*".

<table>
<tr>
<td>Remediation of Ground Contamination – Options Report, The Waterfront, Barry ‘A’ Site and Mole

(Ove Arup and Partners, Report Reference 97/3505, dated December 1997)</td>
<td>The report was commissioned to carry out a study into options for remediation ground contamination at the Site A and the Mole (the Site). The report detailed previous investigation findings and detailed the methodology for this investigation (trial pitting).

The report identified that the Site contamination was limited to the uppermost 0.7m of fill, with some outliers also observed. The report identified four options for remediating the soils at the Site which were capping with removal of hotspots off Site, removal of contaminated soil to tip and replacement with imported fil, soil washing and bio-remediation.</td>
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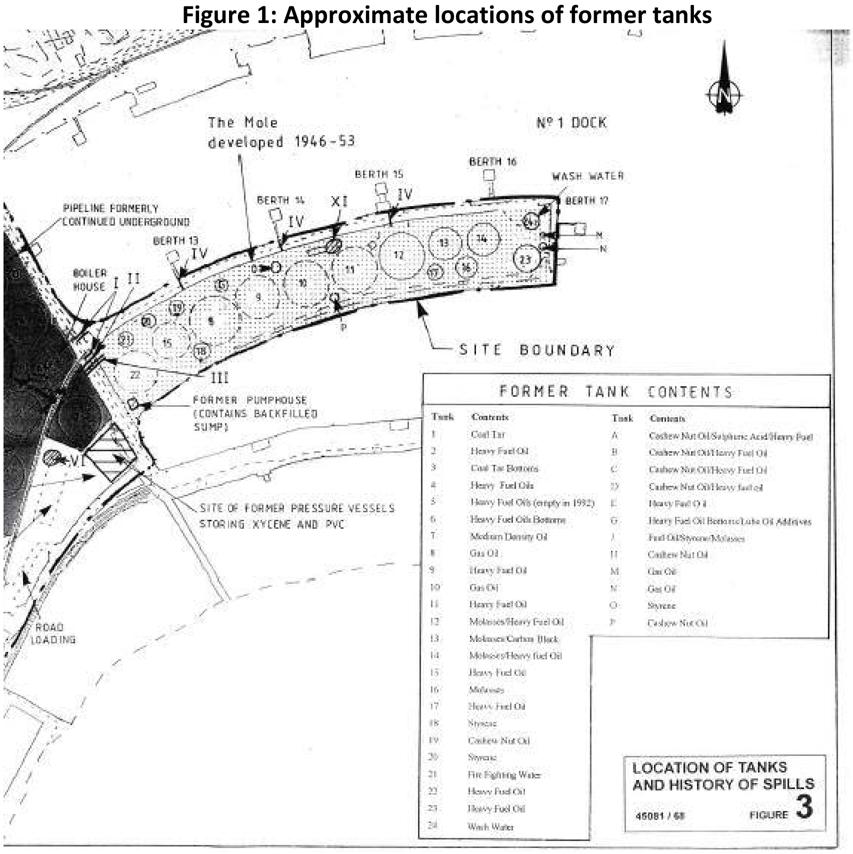
<td>The Waterfront, Barry – Contract 3B, Remediation of ‘A’ Site and Mole, Health and Safety File Volume 1

(Associated British Ports document prepared by White Young Green, dated February 2000)</td>
<td>The report details that the scope of works comprised the removal of contaminated materials to an off-Site licensed tip, Site clearance, regrading and import of clean capping from stockpiles to the North of Barry No. 2 Dock (otherwise known as the ‘borrow’ area). The report details the remediation standards required for use as Type 3 (retail / highways and leisure) and Type 2/3 (general fill). The report details to the planners what the proposal was for the Site (ref: 97/3505, dated 1997). At the time of reporting TRC had not been provided with any evidence of the verification, with the exception of the drawing of the Site depicting final Site levels.

The report also provided details of the approximate locations of former tanks at the Site, as displayed below.

Figure 1: Approximate locations of former tanks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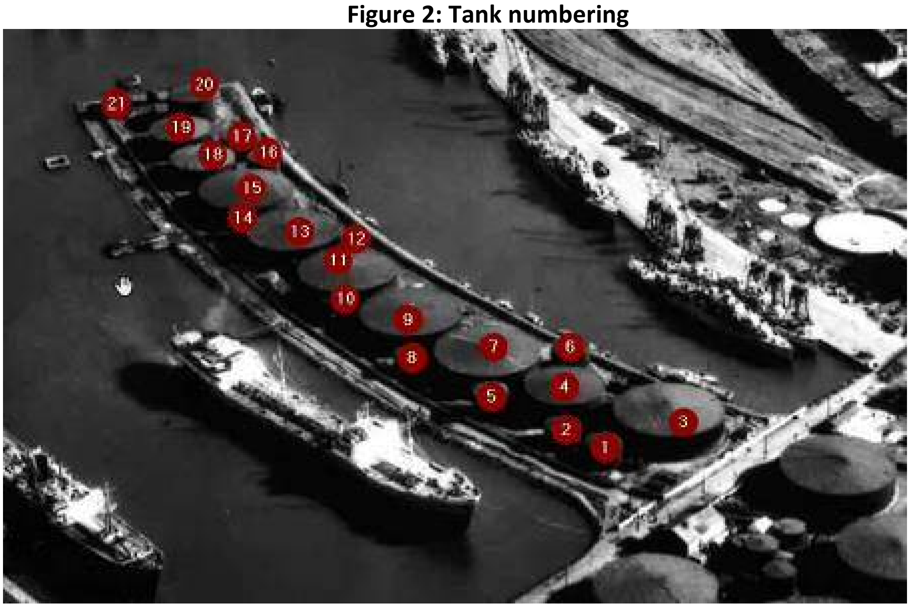
Mole Jetty B26, Port of Barry, Inspection Report (Sub-Surface Engineers Report dated October 2006)	This report prepared by Sub-Surface Engineers relates to the condition of the sea walls and does not include information on the condition of the sub-surface soils. The report concluded that any areas of subsidence, loose blockwork and voiding throughout the revetment should be excavated and the underlying cause determined before rebuilding the revetment. The visual assessment of the Mole Jetty B26 revealed defects, which if not attended to could lead to unnecessary degradation to parts of the structure. It was noted that missing blockwork throughout the revetment should be reinstated and that open joints on all structures should be re-pointed. This report has not been considered further.
Factual Report Barry Waterfront – East Quay (Idom Report Reference FR-17633T-18-549 dated December 2018)	This report prepared by Idom is for a Site located to the east of the Site and as such does not include any Site-specific information. This report has not been considered further.
Trial Excavations Carried out at The Mole, Barry Docks for ABP (Horizon, dated 24-06-20 – 26-06-20)	The scope of works was to locate the bases and thickness of the tanks at the Site. The following image shows the numbering used in the report. **Figure 2: Tank numbering**  A summary of each of the bases is presented below: • Base 4 – Concrete was encountered at 1.4m and the excavation continued to expose the base thickness of 0.27m. • Base 5 – General fill was encountered to 1.0m, underlain by clean stone to 1.1m and then concrete to 1.45m.

	• Base 6 – General fill was encountered to 0.75m, then clean stone to 1.0m, then demolition fill to 1.4m which was underlain by brickwork sections and demolition to approximately 1.7m. Groundwater was present at approximately 1.3m. • Base 7 – Concrete was encountered at 1.3m and trenching was carried out towards the location of base 4. The edge of the concrete slab was encountered within 12.5m and the slab had a 0.2m thickness. • Base 8 – General fill was encountered to 0.75m, then clean stone to 0.85m, then fill to 1.0m which was underlain by concrete to 1.35m. • Base 9 – General fill was encountered to 0.8m, then clean stone to 0.9m, then black fill to 1.2m which was underlain by concrete to 1.6m. Evidence of brickwork was also encountered. Overall the levels of the Site had been raised by approximately 1.0m to 1.4m by general fill and demolition material. The bases of the majority of the tanks in the aerial photograph were still present. They were of concrete construction and varied in thickness between 0.2m and 0.4m.
Phase I Geo-Environmental Desk Study Report (Pick Everard, Ref: MC/MHH/1916 61/17-2/004 - Issue 02, dated: 18/09/2020)	The report identified that historically the Site was part of the Cadoxton River estuary up until c.1898 when Barry Docks were constructed. Since then the Site has been used for rail sidings and docks until c.1965. From 1975 an oil storage terminal is shown on the Site until c.2001. The surrounding land use was characterised as the port and docks with infrastructure and industrial land uses neighbouring the Site on all sides. The report detailed that the Site had been investigated previously and was shown to be contaminated. It detailed that a remediation scheme was undertaken in the late 1990s to remove the worst surface contamination and to install a layer of clean capping. The Site was shown to be underlain by superficial deposits of Tidal Flat Deposits, which comprised clay, silt and sand. This was shown to be underlain by the solid geology of the Blue Anchor Formation mudstone and the Penarth Group interbedded mudstone and limestone. Negligible to moderate risks from natural ground subsidence hazards were identified for the Site. No radon protection measures were considered necessary. The Site was recorded to be within an area of infilled land, probably associated with the creation of the dock. There were also bunds and earthworks noted on the historic mapping of the Site. The bedrock and superficial strata were both classified as Secondary Aquifers. The Site was not found to be located within a Source Protection Zone. The nearest recorded surface water feature was noted to be the Barry Dock immediately adjacent to the Site. The Site was located within 0.5km of two recorded historic or current landfills; one 119m west and the other 470m east. A scrap yard and refuse tip were historically located 65m and 280m northeast respectively. The report identified that there had been demolition of former buildings and tanks on the Site and therefore there the Site was considered to be a high risk of obstructions. Made Ground was anticipated across all areas of the Site and it was noted that the ground levels had been significantly altered to reclaim the land from the former estuary. Geotechnical risks including peat deposits were highlighted as potentially present on the Site which are likely to be soft and compressible. These deposits were classified as being of moderate risk of shrink or swell and that they may cause some differential settlement. Therefore, it was noted that piles are likely to be required, socketed into the rock at depth beneath the Site.

3.0 Ground Investigation Scope of Works

3.1 Scope

The TRC Phase II Geo-environmental and Geotechnical investigation was conducted at the Site during February 2021. The purpose of the investigation was to characterise underlying ground conditions and investigate the potential presence of contamination that may present a risk to the proposed development at the Site. Additionally, a geotechnical investigation was undertaken to aid future foundation design.

The scope of works comprised:

- Supervision of drilling contractors during the advancement of four cable percussive boreholes to a maximum depth of 25mbgl with in-situ geotechnical testing (Standard or Cone Penetration Testing (CPT/SPT), as appropriate);
- Excavation of eight trial pits to a maximum depth of 3.5mbgl;
- Inspection of soils within boreholes/trial pits to facilitate geological logging;
- Collection of soil samples for third party environmental and geotechnical laboratory testing;
- Field monitoring for permanent ground gases and groundwater levels; and,
- Collection of groundwater samples on one occasion.

3.2 Investigation Rationale

The ground investigation was designed by TRC on behalf of the Client to gather information on the environmental and geotechnical ground conditions, groundwater and ground-borne gas conditions at the Site. The TRC investigation aimed to gain good general coverage of the Site. I was not possible to position any cable percussive boreholes in the southern half of the Site, due to the elevation encountered. Therefore, all cable percussive boreholes were located across the northern section of the Site.

The Exploratory Hole Location Plan is presented as Figure 3 in Annex A.

Table 3: Summary of Exploratory Hole Locations

Exploratory Hole	Site Location
CPBH01	Located in the eastern section of the Site, which is adjacent to the east of the Barry Water Activity Club.
CPBH02	Located in the northern section of the Site, which is adjacent to the access track and approximately in the central zone of the Site.
CPBH03	Located in the northern section of the Site, which is adjacent to the access track and approximately in the central zone of the Site.
CPBH04	Located in the western section of the Site, which is adjacent to the entrance to the Site.
TP01	Located in the eastern section of the Site, which is adjacent to the east of the Barry Water Activity Club.
TP02	Located in the eastern section of the Site, which is adjacent to the south of the Barry Water Activity Club.
TP03	Located in the southern section of the Site, which is within the raised area of land and approximately in the central zone of the Site.
TP04	Located in the northern section of the Site, which is within the raise area of land and approximately in the central zone of the Site.
TP05	Located in the northern section of the Site, which is within the raise area of land and west of TP04.
TP06	Located in the southern section of the Site, which is within the raised area of land and west of TP03.

TP07	Located in the western section of the Site, which is within the raised area of land in the south-western corner.
TP08	Located in the western section of the Site, which is within the raised area of land and south of CPBH04.

3.3 Methodology

3.3.1 Ground Investigation

TRC commissioned APEX Drilling (cable percussive drilling contractor) and Garth Plant and Sons Ltd (excavator contractor) to undertake a ground investigation at the Site. Each borehole was advanced using cable percussion drilling methodology and the trial pits via a tracked excavator. The investigation was overseen by a TRC engineer who performed field assessment and logging of soil arisings.

The works included the following key actions:

- Each of the proposed exploratory hole locations was cleared using a Cable Avoidance Tool (CAT) and ground penetrating radar (GPR);
- Cable percussive drilling was performed at each location by the drilling contractor, including in-situ geotechnical testing (Standard / Cone Penetration Testing);
- Trial pits were performed at eight locations by the JCB contractor;
- On-Site field assessment and recording of soil type and potential indicators of contamination;
- Collection of soil samples for environmental and geotechnical laboratory analysis;
- Construction of gas and groundwater monitoring wells in all borehole locations (CPBH01, CPBH02, CPBH03 and CPBH04).

3.3.2 Groundwater and Ground Gas Monitoring

Groundwater and ground gas monitoring was conducted by a technician on four occasions. The dates of the monitoring visits were between 19th March and 21st April 2021.

During each visit, groundwater elevation and potential presence of any free phase oils was measured using an oil/water interface probe. Gas monitoring was undertaken using a portable gas analyser at each monitoring well head. The field assessment gathered data relating to the concentrations of permanent ground gases (e.g. methane, carbon dioxide, carbon monoxide and oxygen).

3.4 Environmental Laboratory Analysis

A total of 19 soil samples was collected for environmental analysis during the investigation works. All soil samples were packed in laboratory provided containers and delivered to I2 Analytical (I2) for chemical analysis.

All soil samples were collected in order to provide environmental data on the quality of near surface and shallow soils beneath the Site. Representative samples of Made Ground / Fill and natural deposits were collected where feasible. The analytical suite of soils included the following parameters:

- Asbestos (Made Ground/Fill Materials only);
- Heavy metals suite;
- Polycyclic aromatic hydrocarbons (PAH); and,
- Total petroleum hydrocarbons – Criteria Working Group (TPH-CWG).

Groundwater samples were collected from three of the boreholes. The samples were sent to I2 Analytical (I2) and analysed for the following:

- Heavy metals suite;
- Polycyclic aromatic hydrocarbons (PAH);

- Total petroleum hydrocarbons – Criteria Working Group (TPH-CWG);
- Volatile organic compounds (VOCs); and,
- pH and sulphate contents

The full set of chemical results are presented in Annex E.

3.5 Geotechnical Laboratory Analysis

Soil sampling for geotechnical testing was undertaken via disturbed and bulk sampling. The geotechnical testing was performed by I2 Analytical (I2) and comprised the following:

- 15 Moisture Content;
- 14 Atterberg Limits;
- 14 PSD by wet sieve;
- 2 Sedimentation Tests;
- 17 pH and water-soluble sulphate;
- 13 Organic matter; and,
- 2 Point Load tests.

The full set of geotechnical results is presented in Annex G.

4.0 Factual Summary of Investigation Findings

4.1 Historical and Archive Information

The following publications of the British Geological Survey (BGS) have been examined in respect of the strata underlying the Site:

- BGS GeoIndex; and,
- BGS Historical Borehole Records.

The Site is shown to be underlain by superficial deposits of Tidal Flat Deposits, which comprise clay, silt and sand. This is shown to be underlain by the bedrock geology of the Blue Anchor Formation (mudstone) and the Penarth Group (interbedded mudstone and limestone). Artificial Ground (Made Ground) is anticipated across the entirety of the Site. The BGS online lexicon describes the strata for the Site as the following:

- Tidal Flat Deposits – *"Tidal flat deposits, including mud flat and sand flat deposits, are deposited on extensive nearly horizontal marshy land in the intertidal zone that is alternately covered and uncovered by the rise and fall of the tide. They consist of unconsolidated sediment, mainly mud and/or sand. They may form the top surface of a deltaic deposit. Normally a consolidated soft silty clay, with layers of sand, gravel and peat. Characteristically low relief".*
- Blue Anchor Formation – *"The formation typically comprises pale green-grey, dolomitic silty mudstones and siltstones with thin arenaceous lenses and a few thin, commonly discontinuous beds of hard, dolomitic, pale yellowish-grey, porcellanous mudstone and silltstone ("Tea Green Marl" of Etheridge, 1865). In southern England and Wales only, the "Tea Green Marl" is overlain by the "Grey Marls" (Richardson, 1906). This unit (equivalent to the upper part of the Rydon Member and the whole of the Williton Member of Mayall, 1981) comprises grey, black, green and, rarely, red-brown dolomitic mudstones with, in the higher beds, yellowish-grey dolostones; also present are laminated siltstone beds with mudcracks, scarce pseudomorphs after halite, and locally abundant gypsum; miospores occur throughout and bivalve fossils and bioturbation become increasingly common upwards".*

An early map from 1879, provided within the Ove Arup 1994 report indicates the location of the former Cadoxton Estuary, which separated Barry Island from the mainland. The location of The Mole was indicated on the map and has been reproduced on Figure 3 below:

Figure 3: Ordnance Survey Map Extract from 1879

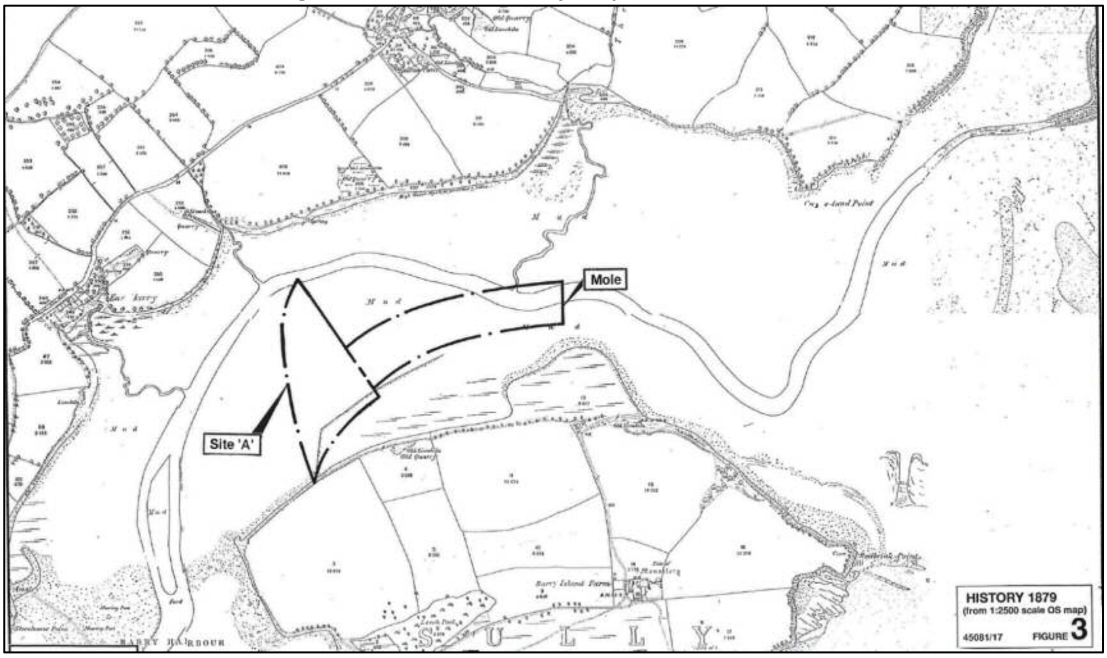
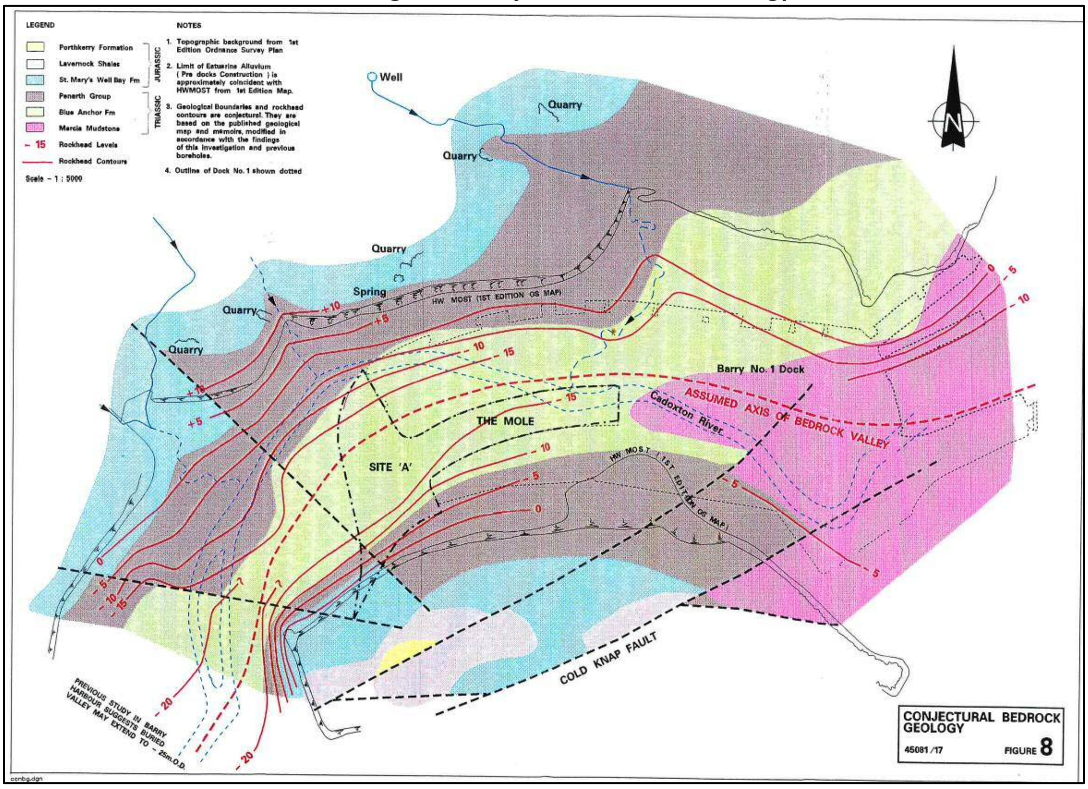


Figure 4 depicts the main location of the channel which coincides with the north eastern corner of the Site. On either side of the channel, the map describes the presence of 'mud'. It is therefore reasonable to assume that the elevation of the area described as mud was likely to be located at approximately sea level (i.e. 0mAOD) with the area of the former channel being deeper.

According to the Ove Arup 1994 report the area was reclaimed with the source of the fill used to raise Site levels comprising materials won from the excavation of soft alluvium sediments along with materials originating from the surrounding hillsides and beaches. This information suggests that the Made Ground is likely to be highly variable including soft clays and silts, as wells as granular materials and potential weathered rock. The location of the former channel would suggest that the greatest thickness of Made Ground is likely to be in the north-eastern corner of the Site.

Figure 4 (below) indicates that the Site is underlain by the Blue Anchor Formation. Of particular note is that the map provides details on the estimated depth to bedrock and provides an indication of the assumed axis of the bedrock valley, which is orientated along the length of the Site and anticipated to be located to the north. The -15mAOD bedrock contour is indicated to cross the Site indicating that there is likely to be in the order of 15m of Alluvium located beneath the Made Ground. The depth to the base of the Alluvium is likely to increase beneath the north and northwest of the Site and decrease beneath the south and southeast.

Figure 4: Conjectural Bedrock Geology



No available BGS historical borehole logs exist on the Site. However, several previous ground investigations have been performed at the Site, which provide information of the likely sequence of deposits at the Site. However, for the purposes of this assessment TRC have used their own definitions. This is because previously terminology such as Alluvial Fill was referenced, which is not considered representative of the ground conditions at the Site. Previous versions of the stratum references should be ignored, and Section 4.2 should be referred to for an accurate account of the ground conditions at the Site.

4.2 Ground Conditions

The TRC investigation observed that the soils underlying the Site generally comprised the following:

Table 4: Summary of Ground Conditions

Strata	Description	Observed Thickness (m)	Range of Depth to top of Strata (mbgl)	Maximum recorded depth (mbgl)
Made Ground	Variable cohesive (clay) and granular deposits (gravel) with sand and cobbles also encountered.	7.7 – 12.5	0	12.5 (CPBH01)
Alluvium	Generally, a clay with sand and gravel content, and occasional rare peat.	8.3 – 14.0	7.7 – 12.5	23.0 (CPBH03)
Blue Anchor Formation	Generally, a mudstone with occasional marl and clay recorded.	>3.2	20.8 – 23.0	>24.0 (CPBH01/2)

Notes:

I. The full extent of the Made Ground thickness was not proven in CPBH04 or any of the trial pit locations. Therefore, the Made Ground could extent deeper than the maximum recorded depth of 15.5mbgl, but this is considered unlikely.

II. The Tidal Flat Deposits were not encountered as part of this Site investigation and therefore are no envisaged to be present at the Site.

The borehole logs are presented in full within Annex C.

The BGS mapping suggests that the Site is underlain by Tidal Flat Deposits. The previous reports have used the term Alluvium to describe the Tidal Flat Deposits and for consistency TRC have retained this terminology. The Alluvium was encountered in CPBH01, CPBH02 and CPBH03 only due to the termination depths of the other exploratory locations. However, it is envisaged that the Alluvium will be present across the entirety of the Site.

4.2.1 Made Ground

Made Ground was encountered across the entirety of the Site with a thickness range of 7.7m to 12.5m, although the full extent of the Made Ground was not proven in all instances. CPBH01 in the eastern section of the Site displayed the greatest thickness of Made Ground (12.5m) and CPBH02 in the northern section (eastern half) of the Site displayed the shallowest thickness of Made Ground (7.7m) in the locations where the full extent of the Made Ground was proven.

The Made Ground was found to comprise both granular and cohesive deposits. Sand was fine to coarse grained and anthropogenic components included concrete, ash, clinker, geotextile membrane, steel and iron. Figure 5 below shows the elevation (mAOD) of the base of the Made Ground in the five exploratory holes that has proved the full thickness, which include historical records from the Site. This indicates that while the deepest Made Ground was located in the north-eastern corner of the Site, deep Made Ground, below an elevation of 0mAOD was also encountered elsewhere on the Site.

Figure 5: Elevation to the base of the Made Ground (mAOD)

Figure 6 provides an indication of the main composition of the Made Ground (i.e. predominantly granular, predominantly cohesive or approximately equal/mixed proportions). Figure 7 then provides an indication of where soft cohesive materials were encountered. As before, both Figure 6 and 7 include historical information to create the geological profile of the Site. It should be noted that the majority of the exploratory holes located at the Site were trial pits and therefore much of the data is limited in depth to typically less than 3m. The figures indicate that there were no particular zones where the Made Ground was either cohesive or granular and that there were soft zones located across the Site at varying depths.

Figure 6: General Composition of Made Ground

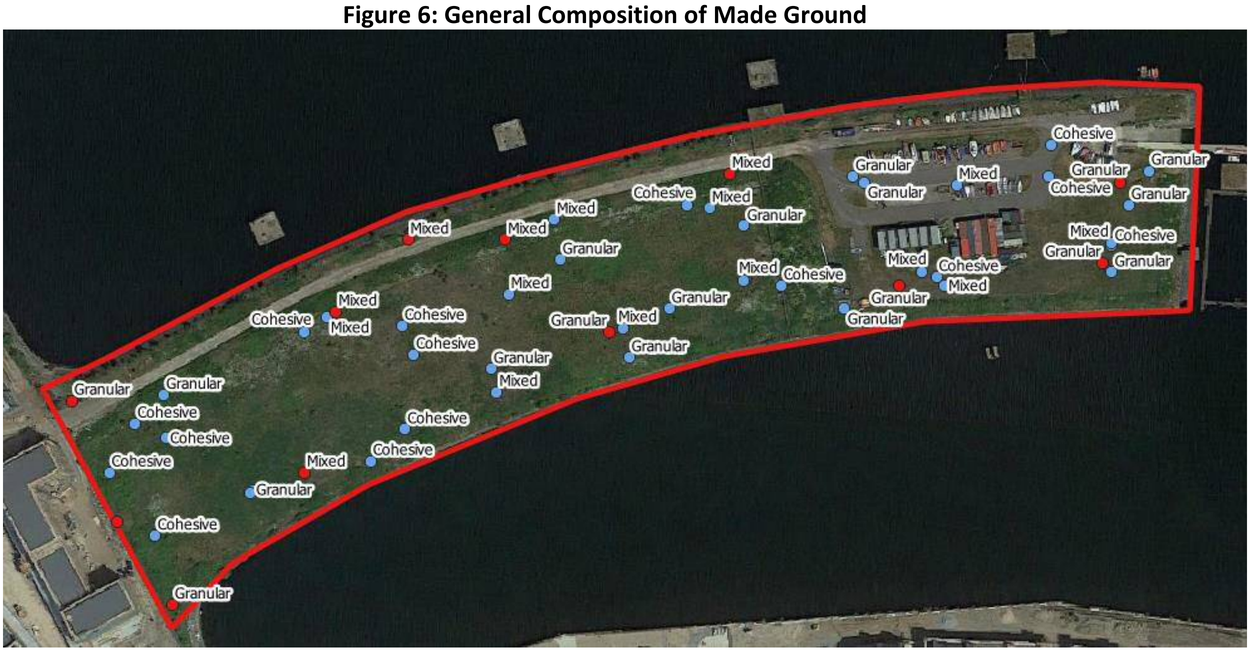


Figure 7: Location of Soft Zones within Made Ground

Made Ground soils are inherently variable in its composition and characteristics. As such, TRC is unable to determine representative values on geotechnical properties. Given the thickness of the Made Ground in-situ testing (SPTs) and general classification testing was undertaken. The following should not be used for design purposes.

SPTs within the cohesive deposits recorded uncorrected SPT 'N' values of 6 to 21 indicating the presence of soft to stiff ground conditions. SPTs within the granular deposits recorded uncorrected SPT 'N' values of 4 to >50 indicating the presence of loose to very dense ground conditions. The results indicate that the Made Ground is variable in strength and density, and these results should not be used for design purposes.

Particle Size Distribution (PSD) testing was undertaken on several samples, with two samples subject to sedimentation. The following table displays the results:

Table 5: Summary of PSD and Sedimentation Testing in the Made Ground

Location	Depth (m)	Very Coarse (%)	Gravel (%)	Sand (%)	Fines (%)
TP02	0.9	3	52	25	20
TP04	0.4	7	16	19	58
TP06	2.8	20	48	13	20
CPBH01	2.0	48	25	11	16
CPBH01	5.0	31	26	13	29
CPBH01	11.0	0	65	29	6
CPBH02	3.0	14	51	16	3 (Silt) 6 (Clay)
CPBH02	4.0	17	31	16	26 (Silt) 10 (Clay)
CPBH02	5.0	37	48	3	11
CPBH03	3.0	21	63	5	11
CPBH03	6.5	0	58	9	33
CPBH04	2.0	0	65	23	12
CPBH04	6.5	17	33	9	41

Notes:

I. Due to the sample size it should be noted that the PSDs do not include boulder content.

The results indicate that the Made Ground is highly variable in composition comprising both fine- and coarse-grained soils and materials. Generally, the very coarse-grained materials are at shallower depths, with decreasing concentrations at greater depths. Overall the Made Ground is predominantly gravel with varying cobble and boulder content.

Atterberg limit testing was carried out on four samples from the Made Ground between depths of 2.5m and 7.0mbgl. The results indicate that the clay content is variable from slightly plastic to highly plastic, recording a maximum plasticity index of 30%. Further analysis has not been undertaken on the Made Ground as these soils will not prove suitable as a founding stratum.

Nine samples were tested for aqueous extract Sulphate (SO_4) and pH, at depths of 0.4m to 7.0mbgl. Water soluble sulphate concentrations were recorded between 35mg/l to 1600mg/l. The worst-case result is indicative of Design Sulphate Class DS-3. The pH values in the soil samples were recorded between 8.0 and 9.0. Mobile groundwater conditions have been assumed, and on this basis, the Aggressive Chemical Environmental for Concrete Class (ACEC) is AC-3.

4.2.2. Alluvium

The Alluvium was encountered in the cable percussive boreholes only, due to the trial pits not reaching the depths of the Alluvium. The Alluvium was encountered between a depth of 7.7m and 12.5mbgl, which persisted to a maximum proven depth of 23.0mbl (CPBH03). The Alluvium was described as a sandy clay with occasional silt and peat content. The sand was fine to coarse grained.

Figure 8 below provides an indication of the elevation of the base of the Alluvium, which includes historical information for the Site as well as TRC data. This indicates that the elevation to the base of the Alluvium was relative uniform ranging from -12.3 to -13.6mAOD. However, the deepest Alluvium was encountered in the borehole located to the southwest of the others. This suggests that the deepest part bedrock valley may actually be to the south of the Site.

Figure 8: Elevation of Base of Alluvium (mAOD)

Several SPTs were performed within the cohesive deposits of the Alluvium which recorded uncorrected SPT 'N' values of 4 to 15, indicating the presence of very soft to firm ground conditions. Undrained shear strengths have been estimated from the SPT 'N' values using the relationship developed by Stroud where mass shear strength equals the SPT 'N' values multiplied by a factor (f1) which is based on the plasticity of the clay. f1 for the Alluvium has been taken as 5.0. Generally, the Alluvium is indicative of soft to firm ground conditions.

Atterberg limit testing was carried out on nine samples of the Alluvium between 8.0m and 18.5mbgl. The results indicate that the clay can be classified as slightly plastic to moderately plastic, with a maximum plasticity index of 25% recorded. The modified plasticity indexes were calculated to be between 9% and 23%, and in accordance with NHBC guidelines the clay is of low to medium volume change potential. The samples analysed are not indicative of a moisture content deficit.

Six samples were tested for SO_4 and pH, at depths ranging from 11.0m to 18.5mbgl. Water soluble sulphate concentrations were recorded between 310mg/l and 1,000mg/l. The worst-case result is indicative of Design Sulphate Class DS-2. The pH values in the soil samples varied from 7.9 to 8.7. Mobile groundwater conditions have been assumed, and on this basis, the ACEC is AC-2.

4.2.3. Blue Anchor Formation

The Blue Anchor Formation (BAF) was encountered in the cable percussive boreholes only. The BAF was recorded from a depth of 20.8m to 23.0mbgl, which persisted to the base of the boreholes (>24.0mbgl). The BAF was described as a mudstone or a gravel with clay and sand content (weathered). The gravel was found to comprise marl.

SPTs performed within the BAF recorded uncorrected SPT 'N' values of 15 to >50, indicating the presence of a weathered zone where rock had weathered to a firm to very stiff clay followed by materials equivalent to rock strength. Undrained shear strengths have been estimated from the SPT 'N' values using the relationship developed by Stroud where mass shear strength equals the SPT 'N' values multiplied by a factor (f1) which is based on the plasticity of the mudstone. f1 for the BAF has been taken as 7.0. Generally, the BAF is indicative of stiff to very stiff ground conditions.

Two samples were submitted for point load strength index testing on the rock cores from the BAF. The results recorded Point Load Strength Indexes of 1.20MPa (Is) to 3.56MPa (Is), with corresponding Is(50) of 1.04MPa and 4.37MPa.

Atterberg limit testing was carried out on one sample from the BAF (CPBH01 at 21.5mbgl). The result indicates that the BAF can be classified as slightly plastic, with a plasticity index of 11%. The modified plasticity index was calculated to be 8%, and in accordance with NHBC guidelines the BAF will not be impacted by volume change potential. The sample is not indicative of a moisture content deficit.

Two samples were tested for SO_4 and pH, at a depths of 21.5m and 23.0mbgl. Water soluble sulphate concentrations were recorded as 130mg/l and 240mg/l. Both results are indicative of Design Sulphate Class DS-1. The pH values of the BAF were recorded as 8.6 and 8.7. Mobile groundwater conditions have been assumed, and on this basis, the ACEC for the BAF is AC-1.

4.2.4. Summary of SPTs/CPTs

Table 5 (overleaf) presents a summary of the SPT/CPT testing undertaken at the Site.

4.3 Groundwater

During the Site investigation groundwater was encountered between 2.8m and 10.0mbgl. During subsequent gas and groundwater monitoring, groundwater resting levels were recorded between 1.61m and 4.20mbgl. Field monitoring data for groundwater monitoring is presented in Annex D. Whilst the monitoring of the tidal influence was not part of this scope of works, monitoring rounds were undertaken during different periods of the day, as summarised below:

- Visit 1 (10:15 – 14:00) – Groundwater levels were only recorded in CPBH03, which recorded a level of 3.05mbgl;
- Visit 2 (12:45 – 14:15) – Groundwater levels were recorded in all boreholes between 3.37m and 4.20mbgl;

- Visit 3 (13:15 – 14:30) – Groundwater levels were recorded in all boreholes between 1.61m and 2.45mbgl; and,
- Visit 4 (14:15 – 15:15) – Groundwater levels were recorded in all boreholes between 3.37m and 4.2mbgl.

The tide will affect the groundwater levels throughout the tidal cycle. It is recommended that accurate monitoring of the tide is undertaken prior to any development taking place. However, given that water levels within the dock are managed by the locks, the tidal range observed may not be significant.

Three samples were tested for SO_4 and pH from the groundwater samples. Water soluble sulphate concentrations were recorded as 311mg/l to 739mg/l. The worst-case result is indicative of Design Sulphate Class DS-2. The pH values were recorded between 7.1 and 7.9. Mobile groundwater conditions have been assumed, and on this basis, the ACEC for the groundwater is AC-2.

4.4 Visual and Olfactory Evidence of Contamination

No visual or olfactory evidence of contamination was encountered during the Site investigation.

Table 6: SPT/CPT Data Summary

SPT N Values																			
Borehole Reference	Depth (m)	Strata	Cohesive/ Granular	1st Seating Blows	1st Seating Penetration (mm)	2nd Seating Blows	2nd Seating Penetration (mm)	1st Increment Blows	1st Increment Penetration (mm)	2nd Increment Blows	2nd Increment Penetration (mm)	3rd Increment Blows	3rd Increment Penetration (mm)	4th Increment Blows	4th Increment Penetration (mm)	N Value	Total Test Penetration (mm)	N Value for full 300mm Penetration	Corrected N Value (N60)
CPBH01	1.2	MG	Cohesive	3	75	3	75	5	75	6	75	5	75	5	75	21	300	21	24
CPBH01	2	MG	Granular	2	75	2	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH01	3	MG	Granular	3	75	4	75	4	75	5	75	5	75	5	75	19	300	19	22
CPBH01	4	MG	Granular	2	75	3	75	3	75	4	75	5	75	5	75	17	300	17	19
CPBH01	5	MG	Granular	1	75	1	75	1	75	1	75	1	75	1	75	4	300	4	5
CPBH01	6.5	MG	Granular	1	75	1	75	2	75	2	75	1	75	2	75	7	300	7	8
CPBH01	8	MG	Granular	1	75	2	75	2	75	3	75	2	75	2	75	9	300	9	10
CPBH01	9.5	MG	Granular	7	75	3	75	3	75	3	75	3	75	3	75	12	300	12	14
CPBH01	11	MG	Granular	2	75	2	75	3	75	3	75	4	75	3	75	13	300	13	15
CPBH01	12.5	MG	Granular	2	75	3	75	3	75	3	75	4	75	4	75	14	300	14	16
CPBH01	14	Alluvium	Cohesive	2	75	2	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH01	15.5	Alluvium	Cohesive	1	75	1	75	1	75	2	75	2	75	2	75	7	300	7	8
CPBH01	17	Alluvium	Cohesive	1	75	1	75	2	75	1	75	2	75	1	75	6	300	6	7
CPBH01	18.5	Alluvium	Cohesive	1	75	1	75	1	75	2	75	1	75	2	75	6	300	6	7
CPBH01	20	Alluvium	Cohesive	1	75	1	75	1	75	2	75	2	75	2	75	7	300	7	8
CPBH01	21.5	BAF	Cohesive	6	75	6	75	7	75	7	75	7	75	8	75	29	300	29	33
CPBH01	23	BAF	Cohesive	25	15			50	2							50	2	7500	8500
CPBH02	1.2	MG	Granular	2	75	2	75	3	75	2	75	2	75	3	75	10	300	10	11
CPBH02	2	MG	Cohesive	1	75	1	75	1	75	2	75	1	75	2	75	6	300	6	7
CPBH02	3	MG	Cohesive	1	75	2	75	2	75	3	75	3	75	3	75	11	300	11	12
CPBH02	4	MG	Cohesive	1	75	2	75	3	75	2	75	2	75	3	75	10	300	10	11
CPBH02	5	MG	Granular	2	75	2	75	4	75	4	75	5	75	5	75	18	300	18	20
CPBH02	6.5	MG	Granular	11	75	5	75	6	75	6	75	7	75	7	75	26	300	26	29
CPBH02	8	Alluvium	Cohesive	1	75	1	75	1	75	1	75	1	75	1	75	4	300	4	5
CPBH02	9.5	Alluvium	Cohesive	1	75	2	75	1	75	2	75	1	75	1	75	5	300	5	6
CPBH02	11	Alluvium	Cohesive	1	75	1	75	2	75	1	75	2	75	1	75	6	300	6	7
CPBH02	12.5	Alluvium	Cohesive	2	75	2	75	3	75	2	75	3	75	3	75	11	300	11	12
CPBH02	14	Alluvium	Cohesive	2	75	2	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH02	15.5	Alluvium	Cohesive	1	75	1	75	2	75	1	75	2	75	2	75	7	300	7	8
CPBH02	17	Alluvium	Cohesive	1	75	1	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH02	18.5	Alluvium	Cohesive	2	75	2	75	3	75	2	75	2	75	2	75	9	300	9	10
CPBH02	20	Alluvium	Cohesive	2	75	2	75	3	75	3	75	3	75	3	75	12	300	12	13
CPBH02	21.5	BAF	Granular	2	75	3	75	4	75	4	75	4	75	4	75	16	300	16	18
CPBH03	1.2	MG	Granular	25	0			50	1							50	1	15000	16500
CPBH03	2	MG	Granular	5	75	6	75	8	75	8	75	9	75	8	75	33	300	33	36
CPBH03	3	MG	Granular	3	75	4	75	4	75	5	75	5	75	6	75	20	300	20	22
CPBH03	4	MG	Granular	3	75	5	75	5	75	5	75	6	75	6	75	22	300	22	24

CPBH03	5	MG	Cohesive	3	75	4	75	4	75	5	75	6	75	5	75	20	300	20	22
CPBH03	6.5	MG	Cohesive	5	75	3	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH03	8	MG	Cohesive	2	75	2	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH03	9.5	Alluvium	Cohesive	1	75	1	75	1	75	1	75	1	75	1	75	4	300	4	4
CPBH03	11	Alluvium	Cohesive	1	75	1	75	2	75	1	75	2	75	1	75	6	300	6	7
CPBH03	12.5	Alluvium	Cohesive	1	75	1	75	1	75	1	75	1	75	1	75	4	300	4	4
CPBH03	14	Alluvium	Cohesive	2	75	2	75	2	75	2	75	2	75	2	75	8	300	8	9
CPBH03	15.5	Alluvium	Cohesive	2	75	3	75	3	75	4	75	4	75	4	75	15	300	15	17
CPBH03	17	Alluvium	Cohesive	1	75	1	75	1	75	1	75	1	75	1	75	4	300	4	4
CPBH03	18.5	Alluvium	Cohesive	1	75	2	75	1	75	2	75	2	75	1	75	6	300	6	7
CPBH03	20	Alluvium	Cohesive	2	75	2	75	3	75	2	75	3	75	3	75	11	300	11	12
CPBH03	21.5	BAF	Cohesive	3	75	3	75	4	75	3	75	4	75	4	75	15	300	15	17
CPBH03	23	BAF	Cohesive	25	16			50	9							50	9	1667	1833
CPBH04	1.2	MG	Granular	2	75	3	75	3	75	4	75	4	75	4	75	15	300	15	17
CPBH04	2	MG	Granular	2	75	2	75	3	75	3	75	3	75	4	75	13	300	13	14
CPBH04	3	MG	Granular	3	75	3	75	4	75	4	75	3	75	4	75	15	300	15	17
CPBH04	4	MG	Granular	1	75	2	75	2	75	3	75	2	75	2	75	9	300	9	10
CPBH04	5	MG	Granular	3	75	2	75	6	75	6	75	3	75	2	75	17	300	17	19
CPBH04	6.5	MG	Granular	2	75	4	75	2	75	3	75	3	75	4	75	12	300	12	13
CPBH04	8	MG	Granular	4	75	4	75	6	75	5	75	6	75	4	75	21	300	21	23
CPBH04	9.5	MG	Granular	3	75	5	75	5	75	2	75	6	75	6	75	19	300	19	21

NB:

I. The Corrected N Value (N_{60}) value displayed in Table 6 has been rounded to the nearest whole value for design assessment.
II. The determined value is referred to as N as N, the Standard Penetration Test resistance, is defined as the number of blows for 300mm penetration.
III. The Corrected N Value (N_{60}) is the equivalent Standard Penetration Test resistance corrected to energy losses, which is defined by the following equation: $N_{60} = (Er/60) \times N$. Where Er is the Energy Ratio for the drilling rigs SPT hammer equipment and N is the SPT N Value. See the drillers certificate in Annex C for the Er adopted.
IV. MG – Made Ground and BAF – Blue Anchor Formation

5.0 Soil and Groundwater Assessment

5.1 Soil Assessment

In order to appraise the significance of the concentrations reported by laboratory testing, TRC has assessed each contaminant species that is elevated above the laboratory LOD against published screening criteria referred to as Generic Assessment Criteria (GAC). GACs are derived from the following reference material:

- Land Quality Management Limited and Chartered Institute of Environmental Health (November 2014), the LQM/CIEH S4ULs for Human Health Risk Assessment. Document reference: S4UL3435;
- Development of Category 4 Screening Levels for assessment of land affected by contamination – SP1010 (September 2014);
- LQM S4ULs: evaluation of 2017USEPA Toxicological Review of Benzo(a)pyrene; and,
- LQM/CIEH S4ULs for Nickel according to land use (Revised August 2015).

TRC has selected GACs for a residential development without gardens, based on the proposed residential apartments. A summary of the laboratory data and the screening tables with relevant GACs is presented in Annex F.

5.1.1 Asbestos

Asbestos fibres were identified within four samples of Made Ground. The samples containing asbestos were found to contain Amosite and Chrysotile fibres in the soil. Asbestos quantification was undertaken on these samples and identified a concentration of <0.001% (less than the laboratories limit of detection) asbestos fibres in the soil in all of the samples.

5.1.2 Heavy Metals

Minor concentrations of heavy metals were detected in soil samples (both Made Ground and natural soils). The chemical screening resulted in minor exceedances of the residential (without gardens) GAC for Lead and Elemental Mercury within Made Ground samples. The exceedances are detailed in Table 7.

Table 7: Summary of Soil Exceedances- Metals

Contaminant	GAC (mg/kg)	Maximum Concentration (mg/kg)	Location of Maximum Concentration	No. of exceedances
Lead	330	440	TP01	2
Elemental Mercury	1.2	1.8	TP07	1

5.1.3 Petroleum Hydrocarbons

Minor concentrations of hydrocarbons were detected in soil samples (both Made Ground and natural soils). None of the concentrations exceed the GAC for a residential development without gardens.

5.1.2 Poly Aromatic Hydrocarbons (PAH)

Minor concentrations of Poly Aromatic Hydrocarbons were detected in soil samples (both Made Ground and natural soils). The chemical screening resulted in minor exceedances of the residential (without gardens) GAC for Benzo(b)fluoranthene, Benzo(a)pyrene and Dibenz(a,h)anthracene. The exceedances are detailed in Table 8.

Table 8: Summary of Soil Exceedances- PAH's

Contaminant	GAC (mg/kg)	Maximum Concentration (mg/kg)	Location of Maximum Concentration	No. of exceedances
Benzo(b)fluoranthene	3.9	5.5	TP06	1

Contaminant	GAC (mg/kg)	Maximum Concentration (mg/kg)	Location of Maximum Concentration	No. of exceedances
Benzo(a)pyrene	2.5	4.8	TP06	1
Dibenz(a,h)anthracene	0.31	0.99	TP06	1

As shown above, there are minor exceedances of the residential (without gardens) GAC for heavy metals and PAH's. The exceedances are limited to Made Ground samples on Site. In addition, asbestos was encountered within four Made Ground samples but the quantification data indicated a concentration of <0.001% (LOD).

Given the minor exceedances and presence of asbestos, it is proposed that the soils are suitable for reuse underneath hardstanding and buildings. However, within soft landscaping areas a geotextile marker layer should be placed between the Made Ground soils and clean cover. It is recommended that 300mm of clean cover should be placed above a geotextile marker layer. Verification of the cover system and chemical testing of the imported clean cover soils will be required by a suitably qualified environmental consultant.

5.2 Groundwater Assessment

In order to appraise the significance of the groundwater concentrations recorded, TRC has assessed each contaminant species that is elevated above the laboratory LOD against the following published guidance values:

- Drinking Water Standards England and Wales (2000) (amended);
- Environmental Quality Standards (EQS) for freshwater; and,
- SoBRA, Development of Generic Assessment Criteria for Assessing Vapour Risks to Human Health from Volatile Contaminants in Groundwater, Version 1 (2017).

Groundwater monitoring recorded groundwater in 4No. monitoring wells ranging from 3.37 to 4.20mbgl. No free phase oils or hydrocarbons odours were identified. Groundwater samples were taken from 3No. boreholes and was sent to I2 Analytical for laboratory analysis.

The laboratory analysis reported elevated concentrations of heavy metals exceeding the EQS and DWS in various samples collected and analysed. The exceedances are summarised in the below table.

Table 9: Summary of Groundwater Exceedances- EQS

Contaminant	GAC (µg/l)	Maximum Concentration (µg/l)	Location of Maximum Concentration	No. of exceedances
Cadmium	0.08	0.11	CPBH04	1
Copper	1	23	CPBH04	3
Nickel	4	6.1	CPBH01	2

Table 10: Summary of Groundwater Exceedances- DWS

Contaminant	GAC (µg/l)	Maximum Concentration (µg/l)	Location of Maximum Concentration	No. of exceedances
Arsenic	10	33	CPBH01	3
Selenium	10	21	CPBH01	3

There were no exceedances of the SoBRA commercial GACs.

As shown above, there are minor exceedances of the EQS and DWS due to elevated metals within groundwater samples across the Site. Given the conservative nature of the EQS and DWS screening criteria and only minor exceedances noted, TRC deem the water quality on Site satisfactory. The groundwater quality within the Site will be significantly influenced by the surface water quality associated with Barry Docks. NR Wales are responsible for monitoring water quality of local surface waters. At a nearby sampling point (Whitmore Bay- Barry Island), data is available for the four-year assessment period from 2017-2020. Sewage and debris were observed in trace amounts during between ten and twenty per cent of monitoring visits. There were three observations of tarry residues at this Site in 2018 and one in 2019.

6.0 Ground Gas Assessment

Field monitoring for permanent ground gases was performed at four monitoring well locations on 4No. occasions between the 25th May and 30th April 2021. The maximum concentrations are summarised in the table below and the complete monitoring data are provided within Annex D.

Table 11: Summary of Gas Monitoring Results

Location	Methane (%v/v)		CO_2 (%v/v)		CO (ppmv)		Oxygen		Flow Rate (l/hr)		PID (ppm)
	Peak	Steady	Peak	Steady	Peak	Steady	Min.	Steady	Peak	Steady	Peak
CPBH01	0	0	1.0	1.0	3	3	16.0	16.2	-9.6	-2.1	0
CPBH02	0.1	0.1	0.7	0.5	1	1	19.4	20.2	2.7	0.3	0
CPBH03	0.5	0.1	0.3	0.1	2	2	19.9	20.4	0.2	0.2	0
CPBH04	0.1	0.1	0.1	0.1	1	1	19.9	20.1	3.1	3.1	0

Methane was detected at relatively low levels of 0.1% on average, peaking at 0.5%. Carbon dioxide concentrations were detected in all of the monitoring wells with concentrations ranging from 0.1% to 1.0%. Carbon monoxide peaked in CPBH01 at 3.0% during the visit on the 25th of March but decreased down to 1.0% on the final visit in 21st April. Flow rate was detected in all of the monitoring wells during all monitoring rounds, having both negative and positive flow rates measured. It is considered likely that the variation in flow rates are due to tidal influence.

TRC has assessed the bulk ground gas concentrations in accordance with current guidance (BS8485:2015). Based on the results, a gas screening value (GSV) of 0.165 l/hr was calculated, which would classify the Site as Characteristic Situation 2 (low risk).

However, it was noted that the response zones within the monitoring wells were submerged at the time of monitoring which will alter the results of the gas monitoring.

Whilst the monitoring of the tidal influence on the gas regime was not part of this scope of works, monitoring rounds were undertaken during different periods of the day, as summarised below:

- Visit 1 (10:15 – 14:00) – Groundwater levels were only recorded in CPBH03, which recorded a level of 3.05mbgl;
- Visit 2 (12:45 – 14:15) – Groundwater levels were recorded in all boreholes between 3.37m and 4.20mbgl;
- Visit 3 (13:15 – 14:30) – Groundwater levels were recorded in all boreholes between 1.61m and 2.45mbgl; and,
- Visit 4 (14:15 – 15:15) – Groundwater levels were recorded in all boreholes between 3.37m and 4.2mbgl.

The water level in Barry Docks is believed to be kept at an average height of 10.00 metres, however, on spring tides it is predicted to exceed a height of 13.70 metres the docks become tidal and water levels may rise to 12.00m or greater at high water.

Tidal influence may affect the ground gas regime throughout the tidal cycle. It is recommended that accurate monitoring of the tide and the ground gas regime is undertaken prior to any development taking place. This could be achieved by continuous monitoring.

Field monitoring detected no elevated PID readings in any of the boreholes.

7.0 Geotechnical Assessment

7.1 Summary of TRC Investigation

The following section provides a summary of the intrusive investigation geotechnical parameters. During the intrusive investigation, TRC gathered both in-situ and laboratory geotechnical data for each of the exploratory hole locations at the Site. Four cable percussive boreholes were drilled to a maximum of 24.0mbgl and seven trial pits excavated. Groundwater and gas wells were installed in the four boreholes drilled. The borehole and trial pit logs are presented in Annex C.

The proposed development concept supplied to TRC by the Client at the time of reporting was a mixed-use scheme, which comprised residential dwellings and the commercial facilities. The residential dwellings comprised seven buildings with both town houses and apartments, which covered a majority of the Site. Blocks B, D, E and G were all proposed to be town houses, with Blocks A, C and F apartments. The buildings vary in shaped from rectangular to L-shaped. The apartments were proposed to be a maximum of four storeys and the town houses a maximum of three storeys. The eastern edge of the Site was proposed to be the marina facilities building, which would be a low-rise structure.

It is understood that during the remediation works undertaken by Ove Arup, that Site levels were increased to circa 8.3m AOD, for flood protection reasons. It is understood that in order to take account the potential effects of climate change Site levels now need to be increased further to 9m AOD.

7.2 Geotechnical Assessment

7.2.1 General

This geotechnical assessment is based on the parameters determined from the field work and laboratory analysis described within this report. This geotechnical assessment provides an overview of possible foundation solutions and infrastructure design and does not constitute a detailed design report for the proposed development.

For the purposes of this assessment, TRC has assumed that finished ground levels will be at, or close to, existing ground levels. In the event that these levels are changed, then TRC would recommend that this assessment is revisited to examine potential changes in recommendations.

The original assessment undertaken by Ove Arup suggested that for lightly loaded structures, that a raft foundation solution could be adopted. The use of a raft foundation is the preferred foundation solution, if deemed appropriate. In order to assess whether a raft foundation is viable for the Site it is important that the stress history of the soils underlying the Site is understood. This has been a primary consideration within this geotechnical assessment.

7.2.2 Ground Conditions Summary

TRCs investigation and historical investigations have identified that the Site is underlain by significant amounts of Made Ground. During TRCs investigation Made Ground was proven to persist to a maximum depth of 12.5mbgl. CPBH01 in the eastern section of the Site displayed the greatest thickness of Made Ground (12.5m) and CPBH02 in the northern section (eastern half) of the Site displayed the shallowest thickness of Made Ground (7.7m) in the locations where the full extent of the Made Ground was proven. The Made Ground was generally granular in nature and comprised various fill materials. Figure 6 displays the elevation to the base of the Made Ground (using historical and TRC data), which indicates that the Made Ground varies across the Site between +0.1mAOD to -3.32mAOD.

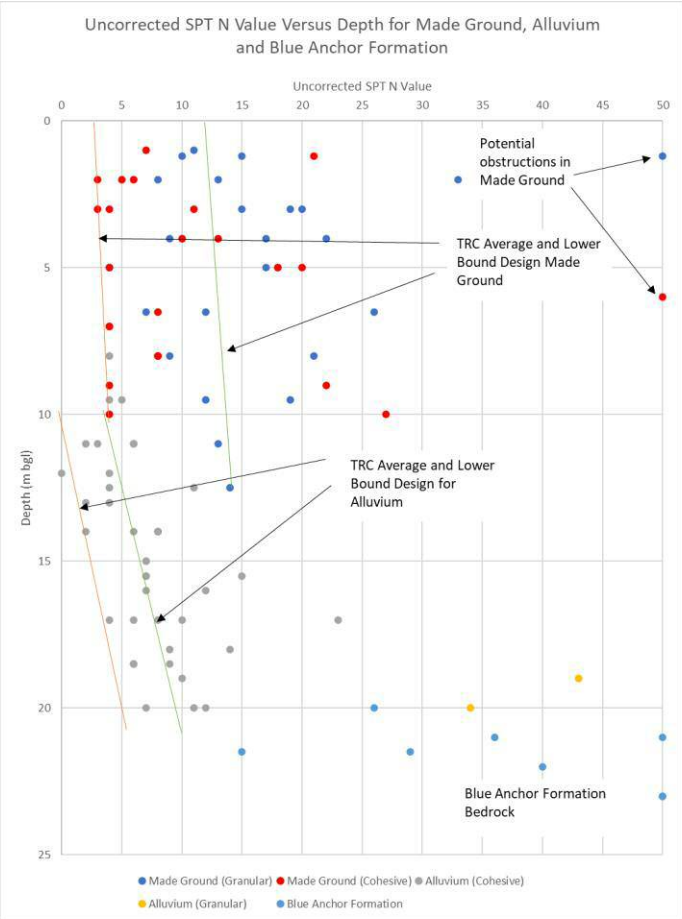
The Made Ground was underlain by Alluvium, which was generally very soft to soft in composition. The upper surface of the Alluvium was encountered at depths of between 7.7m and 12.5mbgl, and the Alluvium persisted to a maximum recorded depth of 23.0mbgl. Figure 7 displays the elevation to the base of the Alluvium (using historical and TRC data) which indicates that the depth to the base of the Alluvium was relatively uniform ranging from -12.3mAOD to -13.6mAOD.

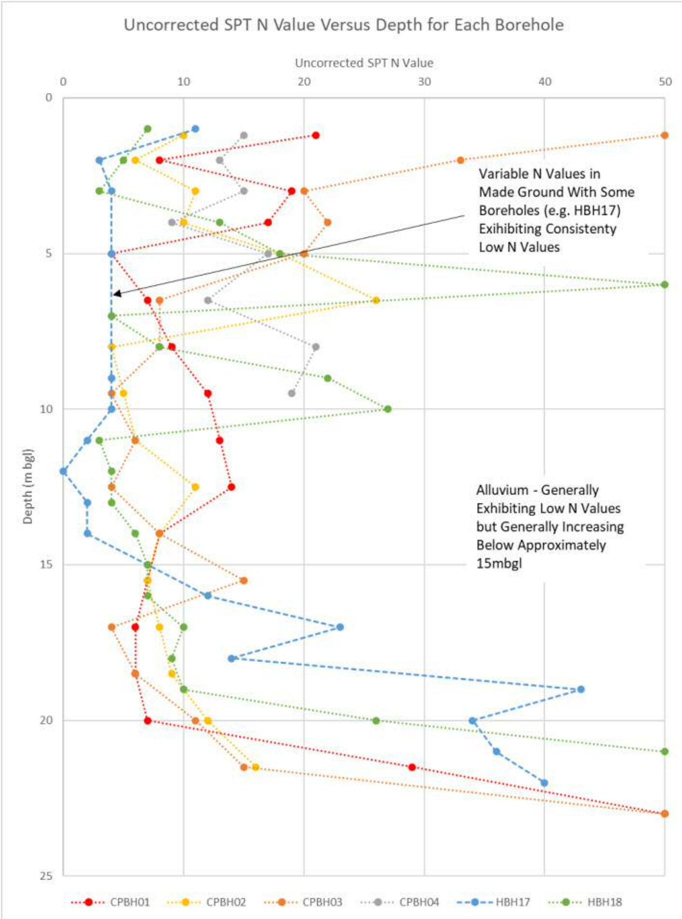
The Alluvium was underlain by the Blue Anchor Formation, which was recorded between depths of 20.8m and 23.0mbgl, which persisted to the base of the boreholes.

Figures 9 and 10 provide an indication of the variability of the ground conditions beneath the Site with uncorrected SPT N values plotted against depth. Figure 10 presents the data distinguishing the results based upon material type (e.g. granular Made Ground, cohesive Made Ground, granular Alluvium, cohesive Alluvium and Blue Anchor Formation) and Figure 11 presents the data separating the results based upon borehole. Data from the four TRC boreholes as well as the two historical boreholes have been used.

The results indicate that the Made Ground was highly variable with very low N values recorded alongside high values. It should be noted that N values in excess of 50 have been recorded (plotted as 50) and that these are considered to be associated with obstructions either in the Made Ground or associated with bedrock. Of particular note is that HBH17 consistently recorded low N values indicating that at some locations poor ground conditions were recorded throughout.

Figures 9 and 10: Uncorrected SPT N Value Versus Depth for Different Material Types and for Each Borehole





7.2.3 Stress History

Development of the Barry Docks began in November 1884, with the area opened to traffic in 1889. The Mole was initially used to store coal, with four masonry towers located to the north of the Site. After the 1930s the Site was redeveloped as a tank farm, primarily storing molasses, heavy fuel oil and gas oil. Each of the tanks had a concrete base, which are all understood to remain in-situ.

The previous reports indicated that limited investigations confirmed the bases typically comprised:

- Concrete ring foundation, 450mm wide and between 840 and 1450mm deep infilled with hardcore.
- 230mm thick concrete raft foundation on rolled hardcore fill.

The tanks were of varying sizes and measured up to approximately 40m in diameter. As the tanks were located in close proximity of each other it is considered that the stress from the imposed load could be approximated to a rectangle measuring 340m by 40m. Assuming that the tanks were approximately 10m high this would mean that the imposed load from the liquid contents was likely to be in the order of 100kPa. At a depth of 10mbgl, within the top part of the Alluvium, the stress reduction is likely to be in the order of 25% and therefore a loading of approximately 75kPa would still be applicable.

Assuming average ground conditions comprising granular deposits with an average SPT N value of 12 to a depth of 12m underlain by soft Alluvium with an average undrained shear strength of 30kPa (estimated from an SPT N value of 8) to a depth of 20.8m this could have resulted in an estimated settlement of approximately 300mm.

This is a broad estimate of the settlement that may have occurred beneath the tanks. However, it is noted that the actual nature of the ground conditions directly beneath the tanks is not known. It is feasible that compacted granular fill material was used directly beneath the tanks that would result in a reduced amount of settlement. Alternatively, it is possible that some form of pre-loading was undertaken prior to construction of the tanks or that the tanks are in fact supported on piled foundations. It is noted that the previous reports state that the confirmation of tank foundation was based upon limited investigation. Nevertheless, this means that the zone directly beneath the former tanks is likely to represent a zone where settlement of the underlying soils may already have occurred or that these zones are supported on deep foundations.

After removal of the tanks, the near surface surrounding soils were investigated. Due to contaminant concentrations detected in near surface soils, a capping layer was installed in order to break the direct contact pathway. The capping layer was placed over the existing soils at the Site by Ove Arup in 1998. It is not known if the capping layer was placed over the entire Site and details of the actually installed capping layer has not been provided. However, the Ove Arup Remediation Options Appraisal Report suggested that heavy surface contamination and deeper hotspots would be removed off-Site and the Site capped with imported Type 3 fill. A capillary break layer was to be laid beneath the cap to protect the surface landscaping layer from recontamination via capillary rise. It was recommended that the capillary break layer should be a minimum of 150mm thick and should consist of single size granular material in the 20 to 40mm nominal grainsize range.

The Health and Safety File (compiled by White Young Green) included a letter to the Local Authority from Ove Arup which provided some further information on the proposed capping layer. This letter suggests that Site levels will be raised to 8.3mAOD (in compliance with Condition No. 20 of the Outline Application). The letter goes on to state that Type 3 material would be used for the capping with Type 2 or 3 material used for bulk fill material beneath the capping layer. The proposed capillary break layer would be 150mm thick and would be laid over areas not covered by the tank bases. Site levels would be raised up to 8.3mAOD (the flood protection level) with fill of minimum quality Type 3. The fill would be laid to a suitable fall to facility drainage and compacted in layers. Tank bases will be perforated near to their centres to facility drainage prior to being covered.

It should be noted that there is no specific reference to whether the Type 3 material was equivalent to Type 3 as defined by Specifications for Highway Works Series 800. Within this specification a Type 3 materials is regarded as being an open graded material with maximum allowed particle size of 80mm and with a fines content (particle size of less than 0.063mm) of less than 5%. A Type 3 material is normally composed of crushed rock, crushed blast furnace slag or recycled concrete aggregate.

The recent investigation indicated that the capping layer generally comprised Made Ground to a depth of 0.6m overlying what appeared to be a Type 1 gravel layer with a geotextile membrane placed above and below the Type 1 gravel. It appears that the Type 1 gravel layer was designed to act as a capillary break layer. The following table provides further details of the capping layer, capillary break layer and depth of tank base (where encountered).

Table 12: Capping Layer Overview

Trial Pit	Cover Layer Thickness (mbgl)	Base of Capillary Break Layer (mbgl)	Notes
TP01	0.6m	0.74m	Cover layer material included cobbles and boulders of concrete.
TP02	0.6m	0.7m	Cover layer material included cobbles and boulders of concrete. Concrete obstruction (tank base) at 1.1mbgl.
TP03	0.6m	0.7m	Concrete obstruction (tank base) at 1.1mbgl.
TP04	0.6m	0.7m	Cover layer material included cobbles and boulders of concrete.
TP05	0.8m	1.1m	
TP06	0.7m	0.7m	No evidence of Type 1 capillary break layer – only a single geotextile membrane installed. Cover layer included cobbles and boulders of concrete.
TP07	0.6m	0.7m	

The results of the TRC investigation has indicated that there are materials (cobbles and boulders) that do not meet the requirements of a Type 3 material. In addition, much of the capping layer material was described as sandy gravelly clay and therefore has a fines content greater than the 5% limit placed on a Type 3 material.

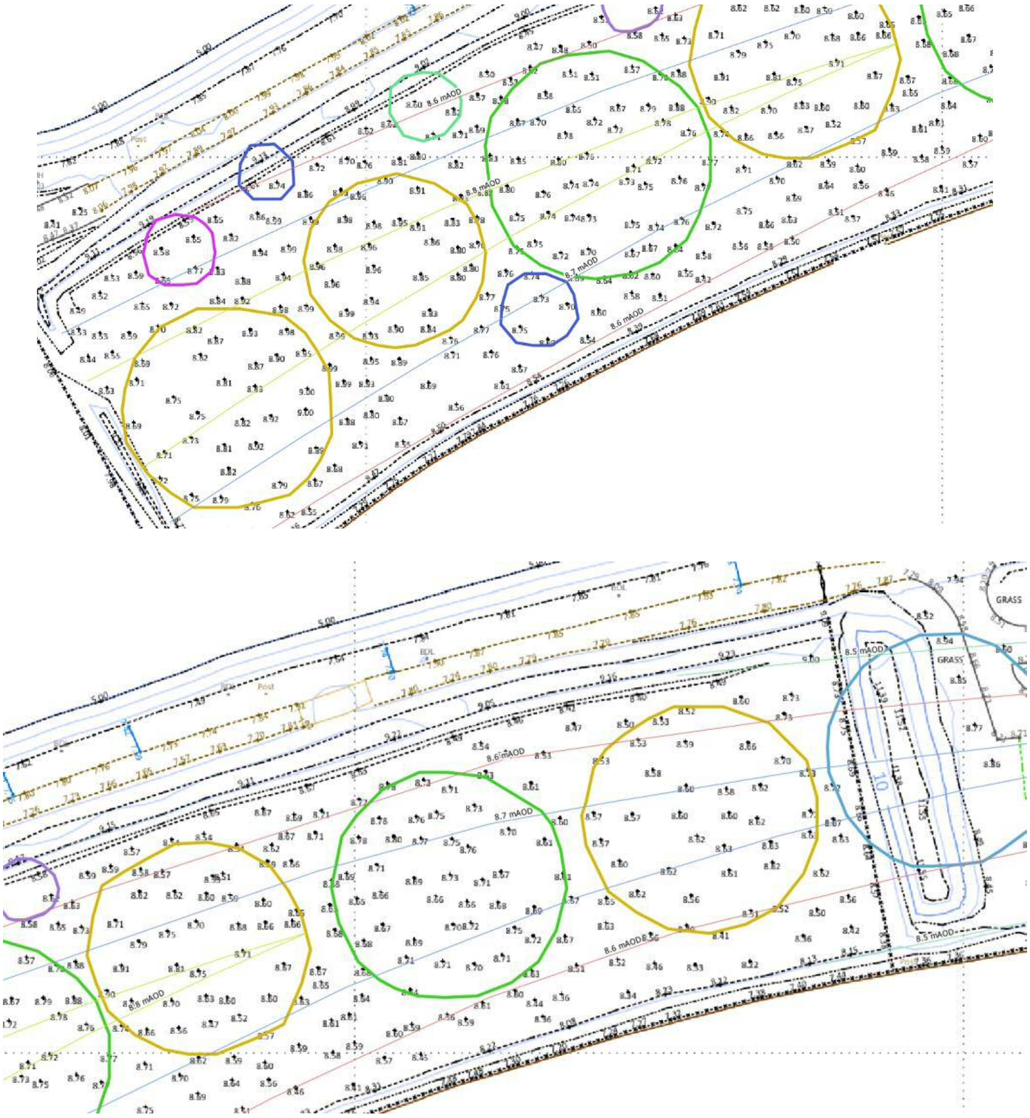
While the specification indicated that the capping layer material would be compacted, details of the compaction specification and/or methods used were not supplied. It should be noted that the presence of cobbles and boulders within the capping material would mean that it would be difficult to achieve a reliable and consistent compaction and therefore this material should not be classified as an engineered material. TRC undertook PSD testing on the shallow and deeper stratums at the Site (see Section 4.2). It should be noted that the PSD testing does not include large cobbles and boulders.

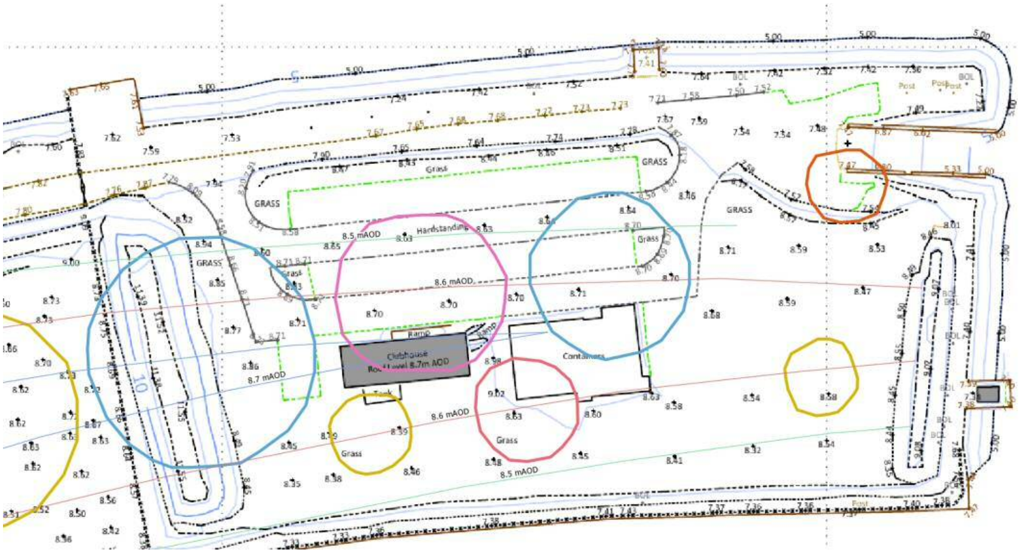
A topographical survey of the Site indicated that the access road (located on the northern side of the Site) was generally in the order of 8.6mAOD at the western end but fell to approximately 7.5mAOD at the eastern end. The main part of the Site was generally at an elevation of 8.5 to 9.0mAOD.

A bank was present around the platform area with was approximately 0.5m higher than the platform area. At the eastern end of the Site there was an area of hardstanding along with a grassed area, containers and a clubhouse. The elevation of this area varied from approximately 7.6m to 8.7m. At the western end of this section there was a mound with the top of the mound at an elevation of approximately 11.5mAOD and was between 10 and 15m wide.

An image of the Ove Arup 'Earthworks Final Levels Setting Out' drawing updated to 'As Built' has been provided (drawing reference not readable). This indicates that the finished levels were highest along the central part of the Site and falls away to the south and north. Figure 11 provides an indication of the finished levels compared with the current levels on the Site. This has been subdivided into sections so that the levels are readable.

Figure 11: Finished Remediation Level Contours Compared with Current Levels

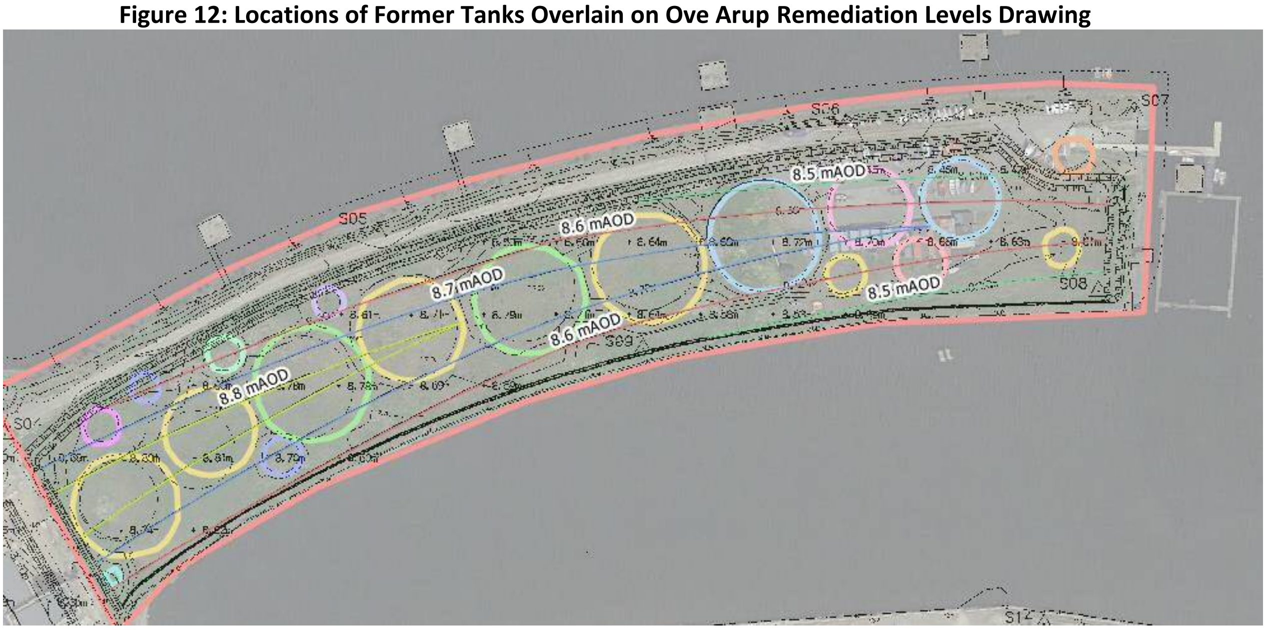




The topographical survey has indicated that while much of the Site is located above the original flood risk level of 8.3mAOD to be achieved by Ove Arup during the remedial works, there are areas that are lower than this level. The remediation should have achieved a consistent level with designed falls to allow for drainage. However, the topographical survey has indicated that levels varied by approximately 0.5m over relatively short distances. The variation of the elevation appeared to be localised and therefore could potentially due to settlement of underlying soils.

It should also be noted that the Ove Arup finished levels drawing also provided an indication of the levels prior to the remediation being undertaken. This indicates that in some parts of the Site, particularly along the southern and northern side the Site level was increased by other 1m. It should also be noted that the location and size of some of the tanks are not consistent with the other drawings showing the tank locations. Figure 12 provides details of the Ove Arup survey overlain with tank profiles taken from other provided drawings. The location of the tanks on the Ove Arup survey are outlined in black.

Figure 12: Locations of Former Tanks Overlain on Ove Arup Remediation Levels Drawing



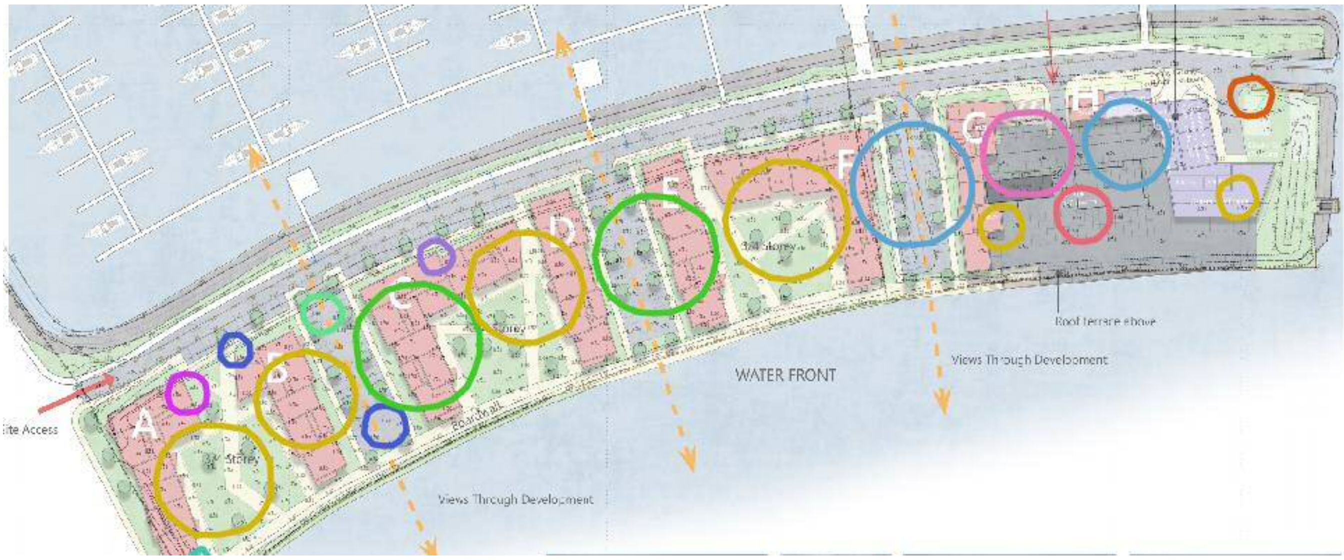
Estimations are that the Site levels were built up by approximately 0.7 to 1.1m during the Ove Arup remediation. Assuming that this imposed an average load of 15kPa across the entire Site (measuring 75m by 375m) it is anticipated that this would result in settlement in the underlying soils of 60mm (assuming average ground conditions comprising granular Made Ground over Alluvium). However, for areas where poorer ground conditions were present (i.e. soft cohesive Made Ground), this could have resulted in settlement in excess of 100mm. It should be noted that soils that have been preloaded from the tanks (assuming supported on a raft foundation) would have resulted in minimal settlement. Therefore, differential settlements in excess of 100mm would have been expected.

7.2.4 Proposed Redevelopment Loading

The proposed development comprises a number of 3 to 4 storey residential blocks of apartments preferably supported on raft foundations. Site levels will need to be raised further in order to achieve a revised flood risk level of 9mAOD. This would mean that in some areas of the Site, levels would need to be raised by 1.5m whereas other areas (located beneath the existing mounds) would need to be cut by approximately 1.5m. Typically, Site levels would need to be raised by approximately 0.5m across the southwestern portion of the Site and by approximately 1m at the eastern end of the Site. This would mean that the imposed loads from building up the Site would vary from 0kPa to up to 30kPa, resulting in additional settlement in the order of 200mm in some areas. This is before the construction of the proposed buildings.

Figure 13 provides an indication of the location of the proposed development overlying the current topographical survey and the location of the former tanks. This demonstrates that in the current layout the buildings span across the location of the former tanks, locations where Site levels will need to be increased and areas where levels may need to be cut. It is therefore considered that the risk of differential settlement is likely to be high.

Figure 13: Proposed Development Plan



7.2.5 Retaining Walls

Retaining sea walls are located along the northern, eastern and western sides of the mole. Originally these would have been designed to retain soils raised to a level of circa 6m AOD and any additional loadings from the tanks and other on-Site structures. Since the walls were constructed Site levels have subsequently been raised to circa 8.5mAOD and Site levels are due to raised further for flood protection reasons. This additional soil will result in additional loads to the retaining walls. To date, it appears that Site levels reduce towards the edges of the Site, which assists in limiting the load to the back of the retaining wall. However, to achieve the finished levels, and a flat building platform Site levels adjacent to the retaining walls will need to be increased.

In addition to raising Site levels, supporting the proposed structures on a raft foundation will also result in an increase of loadings to the retaining wall. The redevelopment plan indicates that in some cases the proposed buildings are in close proximity to the retaining wall.

One potential solution for reducing future settlement to enable a raft foundation to be utilised would be to pre-load the soils by use of a surcharge. This method of ground improvement would also result in a further, albeit temporary, load to the retaining wall.

The condition of the retaining walls has been assessed by Sub-Surface Engineers in 2006, which indicated that there was evidence of subsidence, loose blockwork and voiding throughout the revetment. It is not known if repair to the retaining wall has been undertaken and whether an assessment of whether the retaining wall is capable of supporting additional loads resulting from the proposed redevelopment.

7.2.6 Foundation Recommendations

The recent Phase II investigation undertaken by TRC in February 2021 has indicated that the Site is underlain by a considerable thickness of Made Ground that was found to be highly variable overlying soft Alluvium. The desk study has identified that the stress history of the soils at the Site is complex and that while some areas are likely to have consolidated this is not the case for all soils beneath the Site. The assessment has also indicated that there are potential significant gaps in our current understanding of what has been constructed at the Site and the construction methods used. In addition, there is some discrepancies between the specified remediation specification and what was actually installed.

TRC consider that supporting the proposed buildings on a raft structure will require further consideration of the above factors, particularly given that the raft will result in additional loadings to the retaining structures that surround the Site. In addition, the soils beneath the Site will be prone to long-term ongoing creep settlement and given that the design life for residential buildings is normally in excess of 100 years this could be significant.

TRC note that the traditional method of reducing long-term settlement to enable a raft foundation to be constructed is the use of surcharging. If this method of ground treatment is considered appropriate, further investigation will be required to enable the surcharge to be appropriately designed. However, the effects of additional loading on the existing retaining walls cannot be under-estimated and a full assessment of the structural integrity of the retaining walls will need to be made prior to considering this as a viable option.

The design and geotechnical appraisal of a proposed raft solution has been appraised in greater detail by Card Geotechnics Limited (CGL) on behalf of the Client. This report should be read in conjunction with this TRC report.

It is understood that piling the proposed buildings may be applicable. However, there are other potential solutions that could be explored such as use of Controlled Modulus Columns (CMC). CMC is a ground improvement technique that is suitable for a range of soil types including very soft and soft clays/silts. CMC involves inserting low strength concrete columns into the ground using displacement tools, which will enhance the bearing capacity for any given Site. At ground surface a granular layer is installed to assist in the transfer of the structural loads. It will be important that the columns are installed to a depth where competent strata has been identified which will be circa 20mbgl. A specialist ground improvement contactor has been consulted and confirmed that CMCs would be suitable for the Site. Consideration will need to be given to enabling works required prior to installation of CMCs such as removal of concrete bases and inclusion of a granular layer/working platform at surface. Cost estimates for the enabling works and foundation solution options can be found in Annex H.

It is recommended that specialist advice is sought from a ground improvement contractor with regard to the suitability of this technique in these ground conditions / location and the anticipated achievable bearing capacities. If either of these techniques is selected, it is recommended that appropriate in-situ testing is specified in order to assess the performance of these foundations.

7.2.7 Excavations

Excavations for foundations above CMCs, floor slabs, and laying services should be readily achievable using standard excavation plant. The developer should consider the potential for random and sudden falls from the faces of near vertically sided excavations at the Site. This may be prevalent in the Made Ground soils and in low strength natural strata. Potential for excavation collapse may be exacerbated by water inflows.

A risk assessment on the stability of any open excavation should be undertaken by a competent person and appropriate measures employed to ensure safe working practice in and around open excavations. Temporary trench support or battering of excavation sides should be considered for all excavations, particularly where personnel are required to enter the excavations.

Variable groundwater strikes were recorded during drilling, between 2.8m and 10.0mbgl. During subsequent monitoring shallow groundwater levels were recorded. If groundwater was to accumulate in hallow excavations during development, it is likely that this could be managed via sump pumping. The developer should consider the impact of weather and potential for rainwater and surface run-off to accumulate within excavations.

Water pumped from excavations may require pre-treatment prior to discharge. This could include settlement tanks to reduce silt and suspended solids. No significant contamination has been identified at the Site, therefore further filtration or other such treatment stage is considered unlikely. However, the developer should consult with the local water authority and/or Environment Agency to obtain necessary discharge consents and agree the scope of pre-treatment prior to discharge.

7.2.8 Ground Floor Slabs

The use of ground-bearing floor slabs following ground improvement treatment (CMCs) is likely to be the most cost-effective solution for the proposed developed, provided that the required settlement tolerances can be achieved. Advice from a specialist ground treatment contractor should be sought once the required floor loadings and settlement tolerances are known.

7.2.9 Below Ground Concrete

As detailed in Section 4.0, water soluble sulphate analysis was carried out on 17 samples from the Made Ground, Alluvium and Blue Anchor Formation. Samples of groundwater were also collected for pH and sulphate analysis. In accordance with BRE Digest 1 (2005), the worst-case Design Sulphate Class is DS-3. The associated worst case ACEC assuming mobile groundwater conditions would therefore be AC-3.

7.2.10 Soakaway Potential

The Site is unsuitable for use as a soakaway due to the nature of the ground and the presence of variable Made Ground deposits to a significant depth.

7.2.11 Pavement Construction

No testing of near surface conditions for pavement design was included within this scope. Given the nature of the Made Ground, it is recommended that a CBR of less than 2% is conservatively assumed for pavements across the Site for preliminary design purposes.

8.0 Environmental Risk Assessment

8.1 Conceptual Site Model

The ground investigation performed at the Site by TRC identified minor concentrations of heavy metals, speciated PAHs and hydrocarbons in Made Ground soils. Asbestos fibres were identified in some of the samples of Made Ground, however, quantification tests indicate that the concentration of asbestos in soils was less than 0.001% the laboratories limit of detection.

The methodology of this risk assessment uses the source-pathway-receptor pollutant linkage to provide a qualitative appraisal of environmental risks and potential liabilities associated with soil and groundwater contamination at the Site. The conceptual Site model (CSM) has been prepared considering the proposed end use as a mixed residential and commercial development.

The following CSM has been prepared to take into consideration the findings from the initial Phase 1 report (Pick Everard, Phase 1 Geo-Environmental Desk Study Report, dated 18 September 2020 ref. MC/MHH/191661/17-2/004)) and updated to reflect the findings of the Site investigation.

Table 13: Revised Conceptual Site Model

Source	Pathway	Receptor	Risk
On-Site Sources			
Made Ground: Various contaminants including speciated PAHs, hydrocarbons, heavy metals and TPH associated with former industrial land use at the Site. Potential for gas generation from Made Ground and infilled materials. Historic railway sidings: Potential source of asbestos, heavy metals, phenols, sulphates and fuel oils (TPH and polycyclic aromatic hydrocarbons (PAH).	Dermal contact, ingestion and inhalation pathways	Future Site users	**Low** Concentrations are not significant and unlikely to require active remediation. Risks to be mitigated via placement of hard standing for building footprints and roadways. Engineering cover layers to be used in areas of landscaping. Cover to comprise geotextile membrane and clean cover system (minimum of 300mm) will be required to break the contaminant linkage. If cover systems are placed within soft landscaping areas then the risk will be low.
		Neighbouring residents	**Low** Not applicable given the Site setting surrounded on three sides by Barry Dock. Above development led risk mitigation methods will minimize potential for contaminant risks through removal of risk pathways.
		Construction workers	**Low** Risk pathway to be mitigated via Personal Protective Equipment (PPE), good hygiene

Source	Pathway	Receptor	Risk
			practices and construction Site management.
	Leaching of contaminants and vertical migration into groundwater	Controlled waters	**Low** Screening of the groundwater data indicates minor exceedances of the EQS and DWS criteria due to elevated heavy metals. The EQS and DWS are conservative criteria and given the minor exceedances, TRC deem the water quality on Site satisfactory for the proposed end use.
	Contact with buried services	Buried services	**Low** Proposed development to consider risk of residual contamination and incorporate protective measures as appropriate. This may include clean service corridors and / or use of chemically resistant pipework. In addition, hydrocarbon concentrations on Site are nominal, which pose a threat to water supply pipelines.
	Migration of ground gases onto Site and ingress into buildings	Future Site users	**Low** Gas monitoring on Site indicated that the gas regime on Site is classified as Characteristic Situation 2, whereby basic gas protection measures are required. Consideration of tidal influence may be required to finalise ground gas risk assessment and mitigation measures.
		Construction workers	**Low** Pathway to be managed through good construction practices and mitigation of risks when working in confined spaces.

9.0 Conclusions

9.1 Findings

Investigation at the Site has identified that the Site is underlain by significant amounts of Made Ground placed during the construction of The Mole and surrounding docks. During TRCs investigation Made Ground was proven to persist to a maximum depth of 12.5mbgl. CPBH01 in the eastern section of the Site displayed the greatest thickness of Made Ground (12.5m) and CPBH02 in the northern section (eastern half) of the Site displayed the shallowest thickness of Made Ground (7.7m) in the locations where the full extent of the Made Ground was proven. The Made Ground was generally granular in nature and comprised various fill materials. Figure 6 displays the elevation to the base of the Made Ground (using historical and TRC data), which indicates that the Made Ground varies across the Site between +0.1mAOD to -3.32mAOD.

The Made Ground was underlain by Alluvium, which was generally very soft to soft in composition. The upper surface of the Alluvium was encountered at depths of between 7.7m and 12.5mbgl, and the Alluvium persisted to a maximum recorded depth of 23.0mbgl. Figure 7 displays the elevation to the base of the Alluvium (using historical and TRC data) which indicates that the depth to the base of the Alluvium was relatively uniform ranging from -12.3mAOD to -13.6mAOD.

The Alluvium was underlain by the Blue Anchor Formation, which was recorded between depths of 20.8m and 23.0mbgl, which persisted to the base of the boreholes.

During the Site investigation groundwater was encountered between 2.8m and 10.0mbgl. During subsequent gas and groundwater monitoring, groundwater resting levels were recorded between 1.61m and 4.20mbgl. The tide may affect the groundwater levels throughout the tidal cycle. It is recommended that accurate monitoring of the tide is undertaken prior to any development taking place.

9.2 Summary of Environmental Risk

Whilst the TRC investigation detected elevated heavy metals, PAH and asbestos in soils, it is considered that requirements for remediation will be reduced through the development design that will address active risk pathways to future site users. Design mitigation will include placement of hardstanding (i.e. building footprints, roadways etc.) and clean capping in areas of landscaping. Clean capping should comprise a minimum of 300mm of clean cover should be placed above a geotextile marker layer. Verification of the cover system and chemical testing of the imported clean cover soils will be required by a suitably qualified environmental consultant.

Minor concentrations of heavy metals within groundwater are not considered to present a significant risk to controlled waters. As such, no active remediation is considered necessary.

The gas regime on Site is classified as Characteristic Situation ,2 for which basic gas protection measures are required. Potential for tidal influence may require further assessment to aid the assessment of ground gas risks.

9.3 Summary of Geotechnical Assessment

This report has identified significant stress history associated with the Site, which will need to be considered for geotechnical design of the Site. Due to the extensive details to consider information has not been supplied in this summary, details relating to this can be found in Section 7.2.3.

The proposed development comprises a number of 3 to 4 storey residential blocks of apartments preferably supported on raft foundations. Site levels will need to be raised further in order to achieve a revised flood risk level of 9mAOD. This would mean that in some areas of the Site, levels would need to be raised by 1.5m whereas other areas (located beneath the existing mounds) would need to be cut by approximately 1.5m. Typically, Site levels would need to be raised by approximately 0.5m across the southwestern portion of the Site and by approximately 1m at the eastern end of the Site. This would mean that the imposed loads from building up the Site would vary from 0kPa to up to 30kPa, resulting in additional settlement in the order of 200mm in some areas. This is before the construction of the proposed buildings.

Figure 14 provides an indication of the location of the proposed development overlying the current topographical survey and the location of the former tanks. This demonstrates that in the current layout the buildings span across the location of the former tanks, locations where Site levels will need to be increased and areas where levels may need to be cut. It is therefore considered that the risk of differential settlement is likely to be high.

Retaining sea walls are located along the northern, eastern and western sides of the mole. Originally these would have been designed to retain soils raised to a level of circa 6m AOD and any additional loadings from the tanks and other on-Site structures. Since the walls were constructed Site levels have subsequently been raised to circa 8.5mAOD. This additional soil will result in additional loads to the retaining walls. To achieve the finished levels, and a flat building platform Site levels adjacent to the retaining walls will need to be increased.

In addition to raising Site levels, supporting the proposed structures on a raft foundation will also result in an increase of loadings to the retaining wall. One potential solution for reducing future settlement to enable a raft foundation to be utilised would be to pre-load the soils by use of a surcharge. This method of ground improvement would also result in a further, albeit temporary, load to the retaining wall.

It is not known if repair to the retaining wall has been undertaken and whether an assessment of whether the retaining wall is capable of supporting additional loads resulting from the proposed redevelopment.

It is considered that supporting the proposed buildings on a raft structure requires further detailed design and assessment. TRC consider that further assessment should consider the additional loadings to the retaining structures that surround the Site and potential for long-term ongoing creep settlement.

TRC understands that further detailed appraisal related to the feasibility of a raft foundation has been performed and reported under separate cover by a third-party geotechnical consultant. These findings should be read in conjunction with this report. The traditional method of reducing long-term settlement to enable a raft foundation to be constructed is the use of surcharging. However, further investigation will be required to enable the surcharge to be appropriately designed.

It is understood that piling the proposed buildings may be applicable. However, there are other potential solutions that could be explored such as use of Controlled Modulus Columns (CMC). It will be important that the columns are installed to a depth where competent strata has been identified which will be circa 20mbgl. Consideration will need to be given to enabling works required prior to installation of CMCs such as removal of concrete bases and inclusion of a granular layer/working platform at surface.

Annex A: Figures

NOTES

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Google Imagery June 2018.

REVISIONS

REV.				
P01	FIRST ISSUE			
	Initials	NJ	NJ	13/05/21
REV.	REVISION NOTES/COMMENTS			
	Initials			

TRC
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Brown Street,
Manchester,
M2 1DH

ABP PROPERTY

PROJECT
THE MOLE, BARRY

TITLE
SITE LOCATION PLAN

TRC PROJECT NO.
413800

SCALE
MAP A- 1:6000 @ A3
MAP B- 1:2500 @ A3

PURPOSE OF ISSUE
SUITABLE FOR INFORMATION

STATUS

DRAWING NO
FIGURE 01

REVISION
PO1

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PROJECT
THE MOLE, BARRY

TITLE
SITE DEVELOPMENT PLAN

TRC PROJECT NO.	SCALE
413800	1:1000 @ A3

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FIGURE 02	PO1

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		P01	FIRST ISSUE				
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TITLE		
EXPLORATORY HOLE LOCATION PLAN		
TRC PROJECT NO. 413800	SCALE @ A3 1:1,500	
PURPOSE OF ISSUE ISSUED FOR INFORMATION		STATUS S2
DRAWING NO. FIGURE 03		REVISION P01

PROJECT
THE MOLE, BARRY

Annex B: Site Photographs

Photographic Log

Client Name:	Site Location:	Project No.:
ABP Development Company	The Mole, Barry	413800.0000.0000

Photo No.	Date	
1	May 2021	
Description: Standing at the boat clubhouse area looking west to the rest of the Site and vehicle road situated on the northern boundary.		

Photo No.	Date	
2	27 April 2021	
Description: Standing at the boat clubhouse area looking west to the rest of the Site along the southern boundary.		

Photographic Log

Client Name:	Site Location:	Project No.:
ABP Development Company	The Mole, Barry	413800.0000.0000

Photo No.	Date	
3	27 April 2021	
Description: Turning around from previous photo (photo no 2), looking east at the boat clubhouse area.		

Photo No.	Date	
4	27 April 2021	
Description: Looking at CPBH 101 at the north eastern top corner of the Site.		