The outcomes of the screening assessment are carried forward into the Basement Impact Assessment in the following report sections.

4. SCOPING (STAGE 2)

4.1 Introduction

This section of the report covers the scoping process (Stage 2) of the assessment in accordance with CPG4, which is used to identify potential impacts of the proposed scheme and establish a conceptual site model. The scoping stage also informs the scope of the site investigation. However, ground investigations have already been undertaken at the adjacent sites Hogarth House and 4 North End. As such, this report will assess the suitability of the existing site investigation data to inform the impact assessment (Stage 4).

The findings of the existing intrusive site investigation are summarised below.

4.2 Existing Site Investigation

An intrusive investigation was undertaken on 23 May 2014 by Fastrack Geotechnical (Fastrack) and details are presented in Appendix G. The investigation comprised the excavation of a single flight auger borehole (BH1) located in the courtyard garden area of the property to a depth of 6m bgl. In-situ testing was undertaken and comprised shear vane testing of arisings of cohesive material and Macintosh Probe testing within the borehole of granular strata. A monitoring well was installed within the borehole and the groundwater level was monitored on two occasions.

Two inspection pits were excavated on the site to expose and record the existing foundations and are presented in Appendix H. The foundations were found to be approximately 0.6m deep and comprised of a masonry wall on a corbelled brick based measuring 740mm wide. These foundations are likely to be consistent with those of the neighbouring properties.

5. GROUND AND GROUNDWATER CONDITIONS (STAGE 3)

5.1 Summary

A summary of soil horizons encountered within BH1 is given in Table 9 below. Note that no stratigraphy was given on the log and so it has been interpreted by CGL based on known regional geology and correlations with previous near-by borehole logs.

Table 9. Summary of ground conditions.

Stratum	Depth to top of stratum (mbgl)	Thickness (m)
(MADE GROUND) Comprising light to mid-brown clay.	0	0.4
Firm light to mid-brown silty sandy CLAY. [BAGSHOT FORMATION]	0.4	3
Stiff mid-brown mottled grey CLAY with pockets of sand. [BAGSHOT FORMATION]	3.4	0.8
Medium dense mid-brown SAND. [BAGSHOT FORMATION]	4.2	Base not proven BH drilled to 6mbgl

Groundwater was struck at 4.2m bgl at the top of the sand unit within the Bagshot Formation. Standing water was noted to be 4.80m bgl at the completion of drilling.

5.2 Ground model

The ground conditions encountered during the Fastrack intrusive investigation are in general agreement with the ground investigations undertaken at the neighbouring properties. The Bagshot Formation was encountered in each location and predominantly comprised cohesive beds (clays/silts) to around 4.0mbgl over a granular horizon (sand). The base of the granular unit was not proven in the Fastrack or Chelmer SI investigation.

A firm, dark grey, silty clay was encountered in the MRH Geotechnical investigation at 7.7mbgl. This is has been interpreted by CGL as possible Claygate Member, and would suggest that the geological setting of the site is characterised by thinning of the Bagshot Formation due to increased erosion on the northern slope of Hampstead Heath.

With reference to the Fastrack (6a North End) and Chelmer SI (6 North End) instigation, the top of the granular bed within the Bagshot Formation appears to shallow from some 4.2mbgl in the east (6a North End) to around 3.8mbgl in the west (6 North End). Assuming the proposed basement will be around 4.0m deep, the basement will formed at, or slightly above the surface of the granular basal beds of the Bagshot Formation.

Groundwater was encountered in the Fastrack investigation at around 4.8mbgl, within the sands of the Bagshot Formation. The Chelmer SI investigation (at 6 North End) encountered water seepage at 5.40mbgl, within the Bagshot Formation. The MRH Geotechnical investigation (at 4 North End) encountered water seepages from 5.1mbgl, within a silt horizon of the Bagshot Formation, with resting groundwater level recorded at 6.32mbgl, rising to 5.3mbgl (around 108.3mOD) during subsequent monitoring. These groundwater levels area reasonably consistent and confirm that groundwater is present within the predominantly granular horizon at the base of the Bagshot Formation, resting above the possible Claygate Member. Based on these levels, groundwater is unlikely to be during the basement construction, although some seepages may occur within granular lenses/horizons within the cohesive beds of the Bagshot Formation.

5.3 Geotechnical Parameters

Geotechnical design parameters for the ground conditions encountered have been derived based on the soil descriptions and in-situ testing within the available borehole records and are outlined in Table 9 below.

Table 10. Geotechnical design parameters

Stratum	Design level (mOD)	Bulk Unit weight γ_b (kN/m^3)	Undrained Cohesion c_u (kPa) [c']	Friction angle Φ' (°)	Young's modulus E_u (MPa) [E']
Topsoil/Made Ground (cohesive)	113.0	18	20[a] [0]	28	10[d]
Bagshot Formation (cohesive)	112.6	20	55 [0]	29[c]	27.5[d] [20.6][e]
Bagshot Formation (granular)	108.8	20	-	32[b]	[27]
Claygate Beds (cohesive)	108.4	18	68+3.4z [0]	29[c]	34+1.7z[d] [25+1.3z][e]

a. Burland, J., Standing, J. and Jardine, F. (2001). Building Response to Tunnelling, CIRIA.
b. Forster A The Engineering Geology of the London Area TR WN/97/27 British Geological Survey August 1997.
c. Peck, R.B., Hanson, W.E., and Thornburn, T.H., Foundation Engineering, 2nd Edn, John Wiley, New York, 1967, p.310.
d. Based on 500 Cu
e. Based on 0.75E
f. Based on in-situ shear vane tests

The above values are considered to be moderately conservative and are unfactored (Serviceability Limit State) parameters.

5.4 Conceptual site model

A conceptual site model (CSM) has been developed based on the available data and in accordance with the recommendations of the Arup CGHHS report[2] and is presented diagrammatically in Figure 3.

5.4.1 Existing

The existing site condition is characterised by:

1. Gently sloping site, dipping from southeast to northwest.
2. Topsoil/Made Ground deposits to depths of approximately 0.90m.
3. Party walls with Hogarth House to the north, 8 North End to the east and Pitt House to the south.
4. Groundwater flow within the Bagshot Formation at depths greater than 5.40mbgl.

5.4.2 Proposed

The site condition following redevelopment is characterised by:

1. Made Ground is mostly removed from site.
2. New basement extends out beyond the basement footprint, under the property garden.
3. Groundwater likely to be present below level of proposed basement excavation, though some small seepages may be present at shallower depths, and may potentially generate running sands.
4. Underpins acting as gravity retaining walls in temporary condition.
5. Potential deflections and settlement of underpin walls and effect on adjacent structures.

5.4.3 Critical sections

Two critical sections for analysis have been identified for consideration corresponding to:

- Section A-A: from north (6a North End) to south (Pitt House); And
- Section B-B: from west (4 North End) to the east (8 North End).

Based on the available information, it appears that the foundations of the building to the south of the site (Pitt House) are likely to be up to 7m higher than the basement, and around 5m away from the southern basement wall. Ground movements generated due to basement excavation are unlikely to be fully realized beneath these foundations.

These sections have been analysed to assess the potential for ground movements due to the construction of the basement to cause damage to the neighbouring properties.

6. SUBTERRANEAN (GROUNDWATER) FLOW (STAGE 4)

6.1 Introduction

This section addresses outstanding issues raised by the screening process regarding groundwater flow (see Table 3).

6.2 Impact on groundwater flow

Whilst the proposed basement is to extend out beyond the existing building footprint, a garden is proposed above it in the same location as the current garden. A minimum of 1m Topsoil will be present below the garden, both as a growth medium and to facilitate infiltration and drainage. As such there is no significant increase in the proportion of hardstanding/impermeable surfacing. The provision of a topsoil layer above the proposed basement will actually provide betterment in terms of surface water run-off by providing drainage attenuation that does not exist in the current condition.

It is anticipated that groundwater will be flowing towards the north within the Bagshot Formation (present from a depth of approximately to 5mgbl). This is considered to represent an unconfined perched aquifer above the Claygate Member. Groundwater is likely to be approximately 1m beneath the underside of the proposed basement slab.

It is anticipated that groundwater will be able to flow freely beneath and around the basement perimeter within the relatively permeable Bagshot Formation. On the presumption that the existing and proposed basements associated with the surrounding properties are single storey, groundwater will also be able to flow freely beneath them. On this basis, the proposed development is unlikely to have further cumulative impacts on groundwater flow.

Additionally, the adjacent proposed basements are not connected, allowing drainage both below and between the basements, thus avoiding becoming an impermeable barrier to groundwater flow.

6.3 Recommendations for groundwater control

Observations on groundwater should be carefully recorded during excavation and appropriate mitigation strategies put in place prior to the first excavation. Groundwater has been encountered within the granular Bagshot Formation corresponding to a depth below the proposed basement. Water or moisture was not reported in the cohesive horizons of the Bagshot Formation during the recent site investigations.

Should water bearing sand horizons/lenses be encountered at shallower depths than the proposed basement formation level (i.e. <4mbgl), then some limited seepage into excavations may be encountered. Under such conditions, ‘running sands’ could potentially generate voids beneath adjacent structures and cause collapse of the excavated wall if unsupported. Although such conditions are not anticipated based on the available information, an effective contingency plan for shallow granular soils and/or shallow perched water and running sand conditions will need to be agreed with the contractor at the time of commencement. This will likely take the form of a temporary shoring system to prevent collapse and void formation. Such shallow water seepages are likely to be limited in volume and should be readily controlled with a sump pump. Prolonged groundwater lowering by pumping is not anticipated.

Trench sheets, shoring and a pump will need to be available at all times during the works in case of such an event. There should also be preparation to use no fines concrete where appropriate.

7. LAND STABILITY (STAGE 4)

7.1 Introduction

This section provides calculations to assess ground movements that may result from the construction of the proposed basement and how these may affect the adjacent structures. It is understood that an underpinning construction method will be adopted throughout to form the basement walls and support to the existing foundations. Possible ground movement mechanisms based on the above assumption are outlined below.

- Heave movements: During excavation the soils at formation level will be subject to stress relief as some 4m of overburden are removed. Given that the soils are predicted to behave as drained materials, any minor heave movements in the form of elastic recovery will be removed during levelling for casting of the basement slab. No long term heave is predicted.

- Global stability of the underpins: This relates to an ultimate limit state failure (i.e. sliding/overturning/bearing capacity) of the underpins when they are acting as gravity retaining walls. The stability of underpins, therefore, needs to be considered in the design.

- Long term ground movement: The net loading on the formation soils will generate ground movement, which could affect adjacent foundations. The net loading takes into account the existing stress conditions, additional loads from the basement structure and the weight of soil removed.

- Underpin deflection: Underpins will be acting as stiff concrete retaining walls, which limits the potential for wall deflection. However, deflections that do occur may generate surface settlements that could impact adjacent properties.

7.2 Assumed construction sequence

It is assumed that the basement will be constructed using underpinning techniques excavated sequentially in typically 1.2m wide bays. Given the relatively shallow depth of the proposed basement, it has been assumed that the underpins will be constructed in a single lift. A toe projection will be cast at the base, forming an L-shaped reinforced retaining wall in the temporary condition to resist sliding, overturning and excessive bearing pressures.

The underpins will be constructed in supported trenches with a central soil mass retained to provide support for temporary props and formwork. Sacrificial trench sheeting should be used to provide support to the rear face of the underpin excavations as there is the potential for instability in the sand lenses (if encountered) and horizons (unlikely to be encountered given basement depth) of the Bagshot Formation during excavation, particularly if such granular soils are water bearing.

The underpins will be generally supported in the permanent condition by the ground floor and basement slab, which should be cast before removing the temporary propping.

A plan layout of the external party and internal load bearing walls showing various line loads has been provided by the client and can be found in Appendix A.

7.3 Ground movements arising from basement excavation

During excavation the soils at formation level will be subject to stress relief as some 4m of overburden is removed. Due to the cohesive nature of some Bagshot Formation horizons and the underlying Claygate Member, it is considered likely that some seasonal shrink-swell will occur, causing some volume change during unloading and loading.

A ground movement assessment has been undertaken using OASYS Limited VDISP (***V**ertical* ***DISP**lacement*) analysis software. VDISP assumes that the ground behaves as an elastic material under loading, with movements calculated based on the applied loads and the soil stiffness (E_u and E') for each stratum input by the user.

The proposed development gives rise to a net unloading of the Bagshot Formation and underlying Claygate Member, both during construction and over the long term. The excavation will unload the soils at formation level by some 80kPa (assuming an excavation depth of 4m and an overburden unit weight of $20kN/m^3$). The combined effect of both the immediate undrained unloading and the long-term drained recovery of pore pressures have been analysed.

A contour plot summarising the VDISP displacement output for both short and long term ground movement is provided in Figures 4 and 5, respectively. Full VDISP output can be provided upon request.

7.3.1 Assessment of short-term heave/settlement

Maximum short term heave is of the order of some 8mm and will occur under the centre of both the existing building footprint and property garden. This decreases to an average of some 2mm of heave around the perimeter of the part of the excavation underlying the garden, including adjacent to the garages in the northwest corner of the site. Along the eastern party wall and southern basement wall, negligible movement is anticipated to occur in the short term. In the northeast corner of the site, and along the northern party wall shared with Hogarth House (6 North End), settlement in the order of some 4mm is anticipated.

Short term heave in the central excavation areas will be removed during construction by re-levelling to achieve foundation/slab formation levels.

A contour plot showing the variation of short term ground movements across the basement excavation is presented within Figure 4.

7.3.2 Assessment of long-term heave/settlement beneath basement slab

Maximum long term heave is in the order of 11mm and will occur in the same locations as the maximum short term heave. Maximum heave along the south and eastern party walls is anticipated to be some 2mm whilst movement along the party wall shared with Hogarth House (to the north) are anticipated to be negligible. Along the northern party wall adjacent to garages, heave will be in the order of some 5mm. around 2mm of heave is predicted beneath the southern basement wall, reducing to negligible movements within 5m of the wall.

Bearing pressures below underpins should be limited to 175kPa to control ground movements. This assumes that formations are within the Bagshot Formation.

A contour plot showing the variation of long term ground movements across the basement excavation is presented within Figure 5.

7.3.3 Settlement due to workmanship

The heave/settlement assessment undertaken within VDISP assumes perfect workmanship in the underpin construction and does not allow for settlement of the dry pack between existing footings and the new concrete. With good construction practice, these would be expected to not exceed 5mm. This value will be applied to the overall ground movement and corresponding impact assessment to give a worst case damage category for the adjacent party wall properties. A temporary works strategy should be developed as part of the structural design to ensure the underpins are stable prior to casting of the basement and ground floor slabs.

7.4 Ground movement due to underpin wall deflection

7.4.1 General

The lateral movements of underpins during the construction sequence cannot be fully modelled. The underpin walls have been modelled as 300mm thick concrete walls in Geosolve WALLAP embedded retaining wall analysis software to provide indicative wall displacements in the long-term, drained condition. Although WALLAP is designed to analyse embedded walls, indicative underpin deflections can be reasonably estimated as a cantilever beam by modelling a prop at the base of the wall to mimic the reinforced L-section between the wall and basement slab. In this regard, the wall is effectively modelled as embedded only within the concrete base and does not generate restoring moments based on embedment in soil below the proposed basement formation level.

Ultimately, the short term movement of the underpins will be governed by the quality of the workmanship. Good quality workmanship is therefore considered to be critical in ensuring the stability of the underpins and adjacent structures. Long term stability of the underpins will be controlled by the basement and ground floor slabs.

7.4.2 Analysis

One representative section was analysed, on the eastern site boundary, which forms the party wall with 8 North End, to estimate the lateral movements resulting from the construction of the underpin retaining structures. It is understood that a large single storey basement is under construction on the site located adjacent to the western boundary of the property garden (4 North End). These proposed buildings are considered outside the zone of influence from the predicted heave and settlement movements (see Figure 3) associated with the proposed basement at 6a North End.

Early propping at ground floor level has been assumed in the analysis. A conservative 10kPa surcharge has been included to model the live loads and dead loads (from ground floor slabs etc.) of the adjacent properties.

On the basis of the WALLAP assessment a maximum horizontal wall deflection of 1.5mm has been calculated. This could translate to an effectively negligible 0.75mm of additional settlement behind the party wall foundation with 8 North End[7].

The amount of ground movement will depend largely on the quality of the underpinning workmanship, particularly with the implementation of the dry pack. The WALLAP analysis

[7] CIRIA, *Embedded retaining walls-guidance for economic design* - C580, London, 2003.

has assumed a 'continuous' un-reinforced mass concrete retaining wall has been installed instantaneously. The detailing and construction of the reinforcement and connections between underpin sections will be important in controlling deflections.

High level temporary propping will be required at the top of the excavation (some 0.3mbgl) to control wall deflection during construction. The analysis results indicate that prop loads will bear the order of 32kN/m. Full WALLAP output is provided in Appendix G.

7.5 Damage Category Assessment

Ground movements have been analysed based on the construction scheme as currently envisaged to provide an indication of the potential damage that may be caused to neighbouring structures and infrastructure due to lateral and vertical ground movements.

The calculated ground movements have been used to assess potential 'damage categories' to the neighbouring properties. The methodology proposed by Burland and Wroth[8] and later supplemented by the work of Boscardin and Cording[9] has been used, as described in CIRIA Special Publication 200[10] and CIRIA C580.

Assumed damage categories are summarised in Table 11.

Table 11. Classification of damage visible to walls (reproduction of Table 2.5, CIRIA C580).

Category	Description
0 (Negligible)	Negligible – hairline cracks
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm).
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks > 3mm).
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows (crack width 15mm to 25mm but also depends on number of cracks).
5	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks).

[8] Burland, J.B. and Wroth, C.P. (1974). *Settlement of buildings and associated damage*, State of the art review. Conf on Settlement of Structures, Cambridge, Pentech Press, London, pp611-654

[9] Boscardin, M.D. and Cording, E.G., (1989). *Building response to excavation induced settlement*. J Geotech Eng, ASCE, 115 (1); pp 1-21.

[10] Burland, J.B., Standing J.R. and Jardine F.M. (eds) (2001), *Building response to tunnelling, case studies from construction of the Jubilee Line Extension London*, CIRIA Special Publication 200.

(Very Severe)	

For the critical party wall section the combined impact of short term heave, settlement due to underpin loading and assumed settlement due to workmanship have been combined to determine the overall ground movement of the underpins and adjacent properties due to the construction of the basement.

Worst case maximum combined vertical movements have been calculated to be approximately 3mm of heave below party walls of 6a and 8 North End. The control of ground loss during construction and lateral movement of the underpins will be critical to ensure the global stability of party wall and neighbouring foundations. Assuming good quality workmanship, underpin wall deflections should be minimal (i.e. <2mm) based on CGL previous experience on similar projects and the WALLAP analysis. Staged propping of the underpins will be essential in controlling movement. This is particularly relevant where the basement is adjacent to the foundations of the building to the south of the site (Pitt House), which given the relative levels of the existing foundations and proposed basement is susceptible to ground loss during underpin wall construction.

Table 12 incorporates superimposed horizontal and vertical movements derived from both the underpin wall construction (i.e. workmanship), wall deflection, short term heave due to excavation and heave/settlement over the long term due to the reapplication of structural loads. The method of deriving these values and establishing an appropriate deflection ratio for the neighbouring structure is illustrated graphically in Figure 6.

This deflection ratio has then been used to establish a limiting horizontal displacement of 5mm to ensure that the predicted damage category does not exceed Category 1 'very slight' damage. Estimated horizontal movements are anticipated to be lower than 5mm, of the order of 1.5mm.

The width of the adjacent structures of 6 and 8 North End has been assumed from development plans to be approximately 9m.

Table 12. Summary of ground movements and corresponding damage category.

Party Wall Reference	Horizontal movements (mm)	Maximum deflection (mm)	Horizontal Strain Δ/L^b (%)	Deflection ratio δ_h/L^a (%)	Damage category
6 and 8 North End	<5.0	0.7	0.02	0.008	1 – Very slight

a. *See Figure 2.18 (a) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (L = length of adjacent structure in metres, perpendicular to basement; Δ = relative deflection)*

b. *See Box 2.5 (v) CIRIA C580 (2003) Embedded retaining walls guidance for economic design. (δ_h = horizontal movement in metres*

The predicted damage category imposed on the neighbouring party wall properties due to the proposed basement development and assuming a good standard of workmanship will be 'Category 1' corresponding to 'very slight' damage. Assuming good quality workmanship, and maximum horizontal movements of around 1.5mm, the damage category would be 'Category 0' corresponding to 'negligible' damage.

The building interaction chart for the adjacent party wall structure is presented in Figure 1.

7.6 Construction monitoring

The results of the ground movement analysis suggest that with good construction control, damage to adjacent structures generated by the assumed construction methods and sequence are likely to be (within Category 1) 'very slight'. On this basis, it is recommended that a formal monitoring strategy should be implemented on site in order to observe and control ground movements during construction, and in particular movements of the adjacent property.

The system should operate broadly in accordance with the 'Observational Method' as defined in CIRIA Report 185[11]. Monitoring can be undertaken by using positional surveys compared to baseline values established before any excavation work is undertaken onsite. Survey targets can be affixed to exposed sections of footings and along the face of the adjacent buildings. Regular monitoring of these positions will determine if any horizontal translation, tilt or differential settlement of the neighbouring structure is occurring as the construction progresses. Alternatively, precise levelling can be undertaken at regular intervals around the perimeter of critical neighbouring properties to give an early and accurate indication of deviating ground movements at these critical locations. Monitoring data should be checked against predefined trigger limits and can also be further analysed to assess and manage the damage category of the adjacent buildings as construction progresses.

[11] Nicholson, D., Tse, Che-Ming., Penny, C. (1999). *The Observational Method in ground engineering: principles and applications*. CIRIA report R185.

8. SURFACE FLOW AND FLOODING (STAGE 4)

It is noted in Section 3.4 of this report that the proposed basement will not alter present surface water conditions as no additional hardstanding or paved surfaces will be created and no existing surface water routes will be altered.

As already identified, the site lies outside any EA designated Flood Zone and the site is not located on a street that flooded in the 1975 and 2002 events.

Surface waters will join the existing drainage infrastructure (via basement pumping if a gravity fed solution is not feasible), with no significant changes in drainage outflows anticipated from the site.

As such the development will have a negligible impact on surface water flow and flooding. In addition, the basement is likely to provide enhanced attenuation given its requirement to be drained in accordance with building regulations and the provision of a 1m thick topsoil layer above the proposed basement..

9. NON-TECHNICAL SUMMARY

9.1 General

The findings of this Basement Impact Assessment are informed by site investigation data at the adjacent Hogarth House, information regarding construction methods provided by the client and assumed construction sequence and detail.

- From the available information, it is considered that the proposed basement construction will have a negligible effect on groundwater, surface water and flooding at this site.

- Bearing pressures below underpins should be limited to 175kPa to control ground movements. This assumes that formations are within the Bagshot Formation.

- The construction of the basement will generate ground movements due to a variety of causes including; heave, underpin settlement and underpin wall deflection during and after excavation. Calculations indicate that these will give rise to a damage category within 'Category 0' (negligible damage) for the adjacent properties assuming a good standard of workmanship.

- Observations on groundwater should be carefully recorded during excavation and appropriate mitigation strategies put in place prior to any excavation. Should perched groundwater be encountered within the Bagshot Formation, a temporary pumping strategy will need to be implemented to allow the underpins to be cast. This could be achieved by the use of, for example, a sump chamber.

- It is recommended that an appropriate monitoring regime is adopted to manage risk and potential damage to the neighbouring structures during construction.

- The analyses reported are based on the information currently available and should be revised if changes are made to the proposed design, loading, construction method or sequence.

9.2 Cumulative impacts

Based on a review of the Camden planning portal, aerial photographs and development drawings, it is clear that numerous properties surrounding the site have basements either planned (6 North End; with planning consent), recently built (9 North End) or under construction (4 North End).

The ground movement and building damage category assessment have indicated that damage to neighbouring, party wall properties will be limited to 'very slight damage'. The proposed basement will have a negligible effect on the non-party wall properties to the west (4 North Road) or the south (Pitt House) given the relative foundation depth (4 North Road includes a basement level) and/or the distance between the proposed basement and neighbouring structures (the Pitt House to the south is over 5m from the proposed basement). Additionally, the adjacent proposed/built basements are also single storey, to a similar depth as proposed on site, and will therefore not impact upon or be impacted by the neighbouring and proposed basement, respectively.

On this basis, it is considered that there are no significant cumulative impacts in respect of ground or slope stability due to the proposed development.

The shallow ground conditions beneath the site comprise Made Ground over cohesive Bagshot Formation, granular Bagshot Formation and cohesive Claygate Member. The proposed basement is likely to be founded at the base of the cohesive Bagshot Formation, and possibly in some part the top of the granular unit. Groundwater has been encountered within the granular deposits, corresponding to a depth below the basement. The adjacent basements are founded at a similar depth and ground and groundwater conditions are relatively consistent across this area. On this basis, groundwater is free to flow beneath the proposed and built basements, and it is therefore considered that the proposed development would not contribute further to any cumulative effects.

The proposed development will not materially alter the proportion of hardstanding across the site. It is understood that the existing surface water run-off is currently, and will be discharged to the sewer network through existing connections. On this basis, the development is not considered to contribute to any significant cumulative impact with regard to surface flow or flooding.