4.5 Substructure

4.5.1 Basement Excavation & Wall Construction

At this stage in design, an outline foundation design has been produced based on the Ground Engineering site investigation factual report dated April 2007. This investigation characterised the ground conditions in detail, though it is anticipated that further site investigation will be undertaken to verify details of the existing foundations to the former Post Office building and the buildings along Earls Court Rd. This information will be used to refine the detailed design.

In addition, detailed discussions in relation to construction of the proposed basement wall were undertaken with specialist substructure contractors experienced in this type of project. The feedback received was used to establish a suitable wall zone accounting for construction tolerances, clearances to adjacent structures and feasible wall types based site spatial constraints. This information was preliminary and exact values will vary dependent of the final design details and contractor selected, therefore full details will be confirmed at later stages of design.

The existing basement condition varies across the site. Two separate single storey basements sit beneath the Odeon, one each under the north and south elevations. It is currently understood that the Post Office building and Whitlock House have no existing basements. Representative assumptions with regard to surcharge loads and proposed retaining wall clearances have been applied in the design to date, which will be verified by site investigation in the next stage of design.

It is proposed that a new four storey basement covers almost the entirety of the site, with one and two storey basements under blocks 3/5 and the northern part of block 2 respectively. The maximum depth of excavation will be approximately 19m depending on the exact location and the final foundation solution. It is proposed that the new basement wall will run behind the Odeon retained facade, with the existing basement in that location being modified to incorporate the foundations for the new structure above.

Prior to excavation of the site, a new, stiff embedded retaining wall will be installed from ground level. Augmented with temporary propping this will facilitate excavation of the basement without detriment to the adjacent structures and in the permanent condition will form the basement wall and be propped by the basement floor slabs.

4.5.2 Basement Wall

A stiff, embedded retaining wall will form the basement box, resisting lateral pressures from groundwater, soil and surcharges as well as vertical loads from the basement floor slabs and structure above. It will be designed to resist ultimate limit state forces and to remain within deflection limits determined on the basis of maximum acceptable ground movements (refer to section 5.6 for further details).

The proposed retaining wall solution will be robust and have been used successfully on similar projects in RBKC and London. To ensure the suitability and buildability of the wall, the design has been developed based on detailed input from two major contractors with specialist experience of constructing deep basements on confined sites within London. Based on this input a number of design refinements and construction processes have been identified:

- Reduced rate of construction and completion of shorter segments of wall during each construction phase
- Injection grouting beneath adjacent foundations (with prior agreement through the party wall process)
- Underpinning of adjacent foundations (with prior agreement through the party wall process)
- A stiff piling mat will be utilised to provide a stable working platform for the plant ensuring accurate installation.
- Where required by the retaining wall type, screens will be erected adjacent to the plant to protect adjacent buildings and members of the public from ‘splashing’ of liquid materials.

It is noted that the scheme calls for two new basement lifts to be formed adjacent to the retained facade. A number of construction options have been investigated. It is proposed to utilise sheet piling to form shafts, which will be lined with RC walls to provide the permanent structure. Low-vibration, limited clearance rigs will be utilised (e.g. Dawson silent piling press). Further details are provided in section 4.6.3.
4.5.3 Foundations and basement structure

The basement depth, ground conditions and irregular column layout make The Kensington well suited to raft foundations and this is the solution proposed beneath all blocks. Piled foundations are also a viable option and may be suitable for some parts of the development depending on the final construction sequence selected (see below).

The basement floor structure will be of-situ reinforced concrete (RC) construction. According to the spans and loading requirements this will be a mixture of flat slab construction and one-way spanning slabs between down-stand beams. Columns and walls will also be of RC construction.

Due to the different uses above and below ground, a number of transfer structures will be utilised, principally below ground level, to reconcile the structural grids where required. Depending on the specific load span and spatial requirements these are either fabricated steel beams, storey height steel trusses or RC downstand beams. Beneath townhouses 1-4, RC walls are utilised as deep transfer beams.

Based on the proposed raft solution and the arrangement of transfer structures, it is anticipated that a ‘bottom-up’ construction sequence will be utilised. Therefore as the basement excavation proceeds downwards temporary props will need to be installed to restrain the retaining wall. The design assumes that these will take the form of whalers across the corners sitting at approximately 1.2m above slab level. The raft and slabs will be connected into and receive lateral restraint from the D-wall, where possible using pull-out bar boxes (e.g. Kwikastrip) or if required post-fixed dowel bars. Late pour strips will be utilised to minimise stresses arising from early-age shrinkage. The floor slabs will act as permanent props to the basement wall. Refer to section 4.6 for further details of the proposed construction sequence.

Alternatively, for programme reasons it may be desirable to pursue a full or partial ‘top-down’ sequence utilising plunge columns in large diameter piles. Due the irregular column grid and the large number of transfers at ground level the ‘top-down’ scheme may feature:

- Piles and plunge columns installed from LG1 level (with basement wall cantilevering to ground level)
- Temporary columns - used to facilitate construction, but redundant in the permanent case and therefore cut-out
- Raft foundation for permanent foundations, or hybrid with piles

Indicative cut-away visualisation illustrating nature of structure below ground level
4.6 Construction

4.6.1 Demolition & Temporary Works

All buildings and substructure on site will be demolished, except for the retained facade and the basement and footings directly beneath. Based on the date of construction it is assumed that the Odeon sits on shallow foundations which would be removed as part of the demolition and excavation. Similar construction is assumed for the former Post Office building.

Whitlock is a newer construction that occupies the site of previous building which incorporated a basement. No details of the substructure are currently available. In discussions with the current building manager it was noted that no basement existed for the building; this seems to be borne out by the details indicated on the survey drawings. On this basis and having looked at adjacent buildings, it would seem likely that made ground would exist under and around Whitlock house and this would in turn suggest that piled foundations would be the likely foundation system. This will need to be verified at a later stage. As the building is tightly hemmed in by existing buildings to the north and south it would seem likely that piled foundations along these lines would have to be inset away from the boundary location, and this would in turn suggest that ground beams or a thick raft would be required to pick up the columns located on the N and S perimeter of the building. Since the building proposed to replace Block 3 has deep single storey basement (approx. 4m) and raft foundation, it is likely that any piles in this location would need to be broken down or extracted to accommodate the new construction. This would need to be coordinated with installation of the new secant pile wall around the proposed basement and temporary propping of the adjacent buildings and pavement vaults (if required) and underpinning.

Temporary works to support the retained facade are covered in the Block 1 superstructure section.

4.6.2 Construction Sequence

The sequence shown adjacent outlines the initial principles for constructing The Kensington based on raft foundations and a ‘bottom-up’ sequence. Mace have taken these principles and developed a more detailed sequence based on logistical constraints and their construction expertise. This can be found in the separate construction management plan prepared by Mace.
The proposed scheme calls for two new basement lifts to be formed adjacent to the retained facade. A number of construction options have been investigated and comparatively assessed.

It is proposed to utilise sheet piling to form shafts, which will be lined with RC walls to provide the permanent structure. The shafts would be expected to be installed prior to construction of the adjacent diaphragm wall using low-vibration, limited clearance rigs will be utilised (eg Dawson silent piling press). Internal propping will be utilised within the formed shaft, utilising the balance of earth pressures this affords.

Following construction of the D-wall the shaft may be tied into the main basement volume and progressively excavated. In the final condition RC liner walls with provide the permanent structure, with an additional RC wall constructed in place of the sacrificial sheet piling to form the front wall of the final lift shaft. Water resistant concrete detailing and construction will be utilised incorporating a water-proofing admixture.

**Stage 1 - Assumed existing condition based on record information and visual inspection on site**

**Stage 2 - Installation of external retention system and construction of new support structure for facade comprising RC walls and strip footings, with load jacked into new structure to minimise movement and packed tight; once re-supported existing transfer beams may be cut back**

**Stage 3 - New sheet pilled shaft constructed using limited clearance, low vibration rigs with propping installed; main diaphragm wall constructed and tied into sheet piles; RC lining wall constructed and propping removed**

**Fig: Part plan showing sheet pilled wall with typical temporary propping arrangement to be removed sequentially following construction of RC walls**

**Fig: Part plan showing final as built condition with RC liner wall and additional front wall to lift shaft constructed**

**Fig: Part plans indicating extent of proposed RC support structure and extent of existing structure to be subsequently deconstructed to permit the new basement to be constructed**
4.7 Impact Assessment

Detailed assessments as outlined below will generally be undertaken at the appropriate stage of design by AKTII Limited. AKTII have extensive experience in the development of designs for deep basements in London and throughout the world. Given the size and importance of the project, construction will be undertaken by an established main contractor and associated supply chain with significant experience of such ground works. Comprehensive calculation packages will be prepared by AKTII Limited for Building Control approval at the relevant stage of design.

4.7.1 Ground Movements

4.7.1.1 Soil parameters and design approach

A detailed assessment of ground movements for comparison against relevant acceptance criteria will be undertaken during the detailed design stage to ensure that there is not a detrimental impact on adjacent buildings in the short and long term. The procedure for implementing this analysis is outlined below.

Preliminary assessments of likely ground movements have been made on the basis of the site investigation results obtained to date bench-marked against typical stiffness parameters for the London Clay deposits. These preliminary analyses have formed the basis of the assessment of the viability of a raft foundation and retaining wall proposals, which are to be verified in the next stage of design. A detailed design philosophy shall be developed in preparing the Geotechnical Design Report as required under the provisions of Eurocode 7. Ground modelling for the assessment of ground movements shall principally comprise the following analyses addressing both vertical and lateral movements:

- Elastic halfspace analysis of the proposed raft foundation considering short and long term conditions accounting for variation in pore water pressure and considering time dependency of movements; use of halfspace model to assess vertical ground movements below and adjacent to the excavation with consideration of rebound heave effects in both short and long term conditions.
- Elastic plane-strain 2D section cut analyses for assessment of lateral and vertical ground movements in regions adjacent to the excavation including the effects of basal heave settlement.
- Review of predicted ground movements against empirical derivations and case study data (eg CIRIA C580 data). The results will be assessed against relevant acceptance criteria as outlined below in order to secure the relevant formal approvals for the works to be undertaken. Any resulting requirements with regard to the Contractors’ methodology will be detailed and enforced through the project specifications and preliminaries.

4.7.1.2 Construction phasing

The construction phasing shall be considered in the assessment of time dependency effects. At this stage of design development the following stages have been identified as critical to one or all of the structures that may be affected by the proposed development:

- Stage 1a: Short term heave due to demolition
- Stage 1b: Short term heave due to excavation
- Stage 2a: Short term settlement due to construction before application of brittle finishes
- Stage 2b: Short term settlement due to construction after application of brittle finishes
- Stage 3a: Consolidation settlement due to building occurring during 100 year design life
- Stage 3b: Consolidation settlement due to building occurring after 100 year design life
- Stage 4a: Long term heave due to gross unload occurring during 100 year design life
- Stage 4b: Long term heave due to gross unload occurring after 100 year design life

Through the use of superposition the relevant design conditions for the affected structures may be determined.

4.7.1.3 Time dependency effects

With reference to Tomlinson (2001) and Brien & Sharp (2001), relevant apportioning of the total ground movement into short- and long-term response has been determined.

With reference to Tomlinson (2001), the rate of consolidation settlement will be assumed to conform to drainage model type 1 using the coefficient of consolidation (Cv) value as determined by relevant soil tests to be undertaken. Consolidation curves shall be derived for the London Clay deposits. It will be assumed that the rate of long term heave development follows the same progression through time.

4.7.1.4 Design standards

Design will be undertaken in accordance with Eurocode 7 with load factors taken from Eurocode 1. All relevant ULS and SLS cases shall be considered, which in the case of this design are likely to comprise structural (STR), geotechnical (GEO) and uplift (UPL) cases. The partial safety factors to be utilised in design together with the relevant load case combinations are listed below:

i. Load factors

Demolition
Case 1: 1.00Gk, existing + 0.30Qk, existing.

Excavation
Case 1: 1.00Gk, existing

Where Gk,soil is equal to the weight of soil in accordance with the recommendations of the SI report

Proposed building

Short term effects
Load from proposed building at end of construction Ref: EN1990:2002 Quasi-permanent combination (6.16a)
Case 1: 1.00Gk, proposed + 1.00Qk, proposed
Case 2: 0.90Gk, proposed

Where Qk, proposed is determined considering live load reduction by factor α (Ref: EN1991-1-1:2002 eq NA.2)

Long term effects
Load from proposed building Ref: EN1990:2002 Quasi-permanent combination eq 6.16a
Case 1: 1.00Gk, proposed + 0.30Qk, proposed
Case 2: 0.90Gk, proposed

Where Qk, proposed is determined considering live load reduction by factor α (Ref: EN1991-1-1:2002 eq NA.2)

ii. Material factors

Soil parameters

For the purpose of ultimate limit state design using Type 1 combinations (which encompasses the structural design of the raft, retaining walls and assessment of any secondary effects generated from tilt of cores, etc.) the undrained shear strength will be modified in accordance with EN1997-1:2004 Annex A.

It follows that the Young’s modulus values derived from the undrained shear strength will be subject to the same partial safety factor. When considering serviceability effects the soil parameters will be utilised without the application of partial safety factors.

Concrete

To account for cracking in concrete elements in the short-term the Young’s modulus for the concrete will be modified to 0.75Ec. To account for the combined effects of creep, shrinkage and cracking in concrete elements in the long-term the Young’s modulus will be modified to 0.60Ec.
### Ground Water Flow

On the basis of the findings from site investigations as outlined elsewhere in this report, a perched water table is thought to exist on the site overlaying the relatively impermeable London Clay deposit. Perched water levels can vary seasonally and are prone to rapid changes through heavy rain events on permeable surfaces, accidental events (such as burst water mains) and the introduction of new underground construction causing blockages to natural perched water flow. The River Thames is located approximately 3km to the South and it is assumed that the ground water flow would be in this direction. The presence of open green space in nearby Holland Park may also impact on the prevailing groundwater flow regime.

In common with the existing Odeon basement, the proposed basement will extend down into relatively impermeable London Clay deposits. In contrast to the existing basement, but in common with the consented scheme, the proposed basement will extend laterally for the complete width of the site. Groundwater monitoring has continued, in order to assess the potential impact on peak groundwater levels from interruption of flow within the gravel strata and the need for any resulting mitigation measures. It is noted however that the impact of the proposed development is not anticipated to be significant due to the isolated nature of the proposed basement, the low flow rates in the terrace gravels and the ground surface would be unduly conservative.

#### Groundwater levels for design

The design philosophy in relation to ground water levels to be assumed in design of the permanent works is outlined below for reference. Analysis will be completed at the appropriate stage of design.

**BS EN1997-1:2004 & National Annex**

The Kensington project will be designed to Eurocode 7 (BS EN1997-1:2004 & National Annex) and as such the following limit states shall be considered:

1. **Ultimate Limit State (ULS):** Design values shall be the most unfavourable design conditions that could occur during the lifetime of a structure during an extreme or accidental event.

2. **Serviceability Limit State (SLS):** Design (characteristic) values shall be the most unfavourable design conditions that could occur in normal circumstances. The option is given to the designer to determine design values for ground water by either applying partial factors to the characteristic water pressures or by applying a safety margin to the characteristic water level (cl.2.4.6.(18)). Partial safety factors for use under normal conditions are noted within the code; guidance for accidental conditions comes from cl.2.2.1(3) & the National Annex. For the uplift design condition UK National Annex Table A.NA.15 can be used.

In summary:

- **ULS (STR & GEO) Design Cases:**
  - normal conditions $f = 1.35$
  - accidental conditions $f = 1.00$

- **ULS (UPL) Design Cases:**
  - normal conditions $f = 1.10$

- **SLS (ALL) Design Case:**
  - normal conditions $f = 1.00$

It is expected that the upper layers of the London Clay deposits will contain fissures, leading to penetration of the perched water in the long run and the potential to form an increased head of water. The depth of penetration is subject to engineering judgement based on knowledge of the surrounding ground conditions and on site investigation results.

#### Concrete Basements – Guidance on the design and construction of in-situ concrete basement structures (Narayanan & Goodchild)

With reference to BS EN1997-1-1, the use of a partial factor of 1.2 on the most unfavourable or accidental ULS design case is recommended, this level being taken as ground level unless there is high confidence in the water table. The factor of 1.2 is in accordance with the provisions of BS8102 and is applied to the Eurocode 7 design procedure, with the exception that the accidental groundwater level will be taken as 1m below ground level as per BS8102, which has been the adopted figure in UK construction for many years. It is considered that taking the accidental level at the ground surface would be unduly conservative.

### Surface Water Flow

With reference to section 3.0, according to the Environment Agency Flood map, the site is not at risk of flooding and consequently lies in flood zone 1 with a return period of event of 1 in 1000 years. No change or increase to the extent of hard standing over the site is proposed. New surface drainage will be installed in areas of hardstanding within the proposed development discharging to the public sewer with consent via development discharging to the public sewer with consent via the basement at high level into the public sewers.

In order to meet BREEM requirements, predicted increases in peak rainwater due to climate change must be attenuated. Storage tanks are being located within the site boundary to attenuate surface water prior to being discharged into existing sewers.