4.2.5 Benefits to wider community

The Flood Emergency Plan provided in Appendix B provides wider opportunities for reducing flood risk to the wider community. Specifically, these include:

- Provision of emergency car parking;
- Allowing neighbours to use the proposed emergency access
- Allowing use of any emergency transport along Broom Road;
- Use of the site as a refuge
- Provision of access/egress route for the Lensbury Hotel

4.3 Off Site Impacts (B7a, B3d, B7b)

It is a fundamental requirement under NPPF that any proposed development should not have adverse impacts upon others. In this Section, the impacts are reviewed in terms of:

- Flow paths
- Flood plain storage
- Runoff

4.3.1 Flow paths

Model output from the 1%CC runs is presented in Figure 4-10 and Figure 4-11 for depths and velocities respectively. Note that these show the maxima in each case – which are not necessarily coincident. These figures show that the site is characterised by relatively shallow depths and low velocities compared to areas immediately around it.

Areas of particular note close to the site with deep and/or fast flowing water are as follows:

- Along Broom Road and in the adjacent Sports Ground, there are areas of deep flow and high velocity. This represents a significant flow path, with associated hazard. It does not affect the site directly, but does impact on the access along Broom Road.

- The grounds of the Lensbury are characterised by deep water (> 1.5 m). However, this is generally characterised by low velocity.

These confirm that the site is protected to some extent from flow paths. This is partly due to its elevation and partly due to the buildings on and adjacent to the site.

The modelled flood level data has shown that the maximum water levels in the river are slightly higher than those on the flood plain. At the 1%CC level, this is about 0.20 m from the river to the south of the site. This likely reflects a minor flow path from the Thames towards Broom Road.

The opportunity for flow in this direction on the existing site is limited. The only break in the buildings where this flow could occur is at the gatehouse (Figure 4-12). The ground level at this point is at around 6.41 mAOD and is a local high point. The width of the openings at this point is of the order of 7 m. Reference to Table 3-5 with design flood level data from the Environment Agency shows that the current probability for this level, in the river, is about 1%. Given that levels are lower in the vicinity of the Gatehouse, the probability of this flow path being active is small. Furthermore, for floods at this level, water levels will vary slowly over time by perhaps a few cm per day. Accordingly, this flow path will not be called on to rapidly convey water; it will rather provide a means for water to cross the site in a gradual way, in line with the progression of the flood. This means that hydraulic considerations of velocity and flow area are not limiting factors in its behaviour.

Figure 4-10 Maximum depths: 1% CC

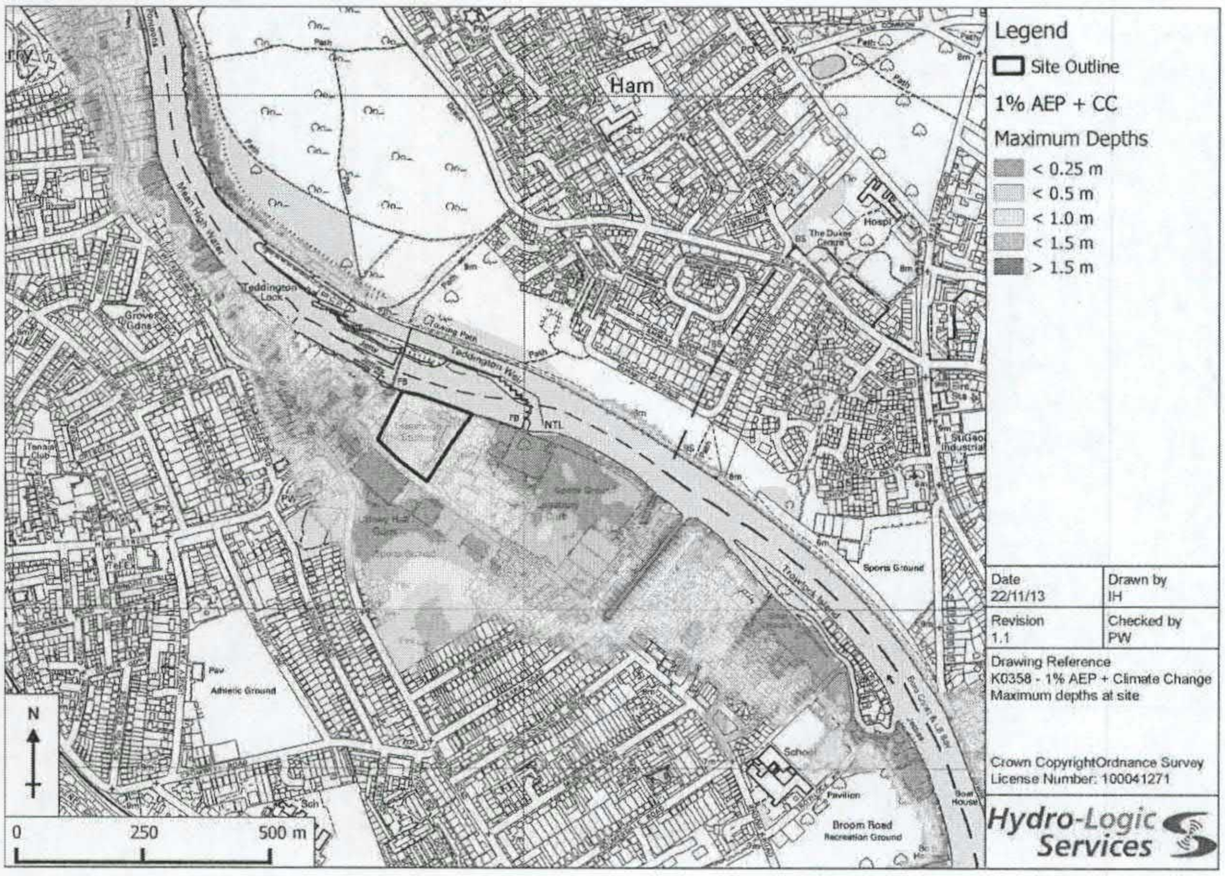
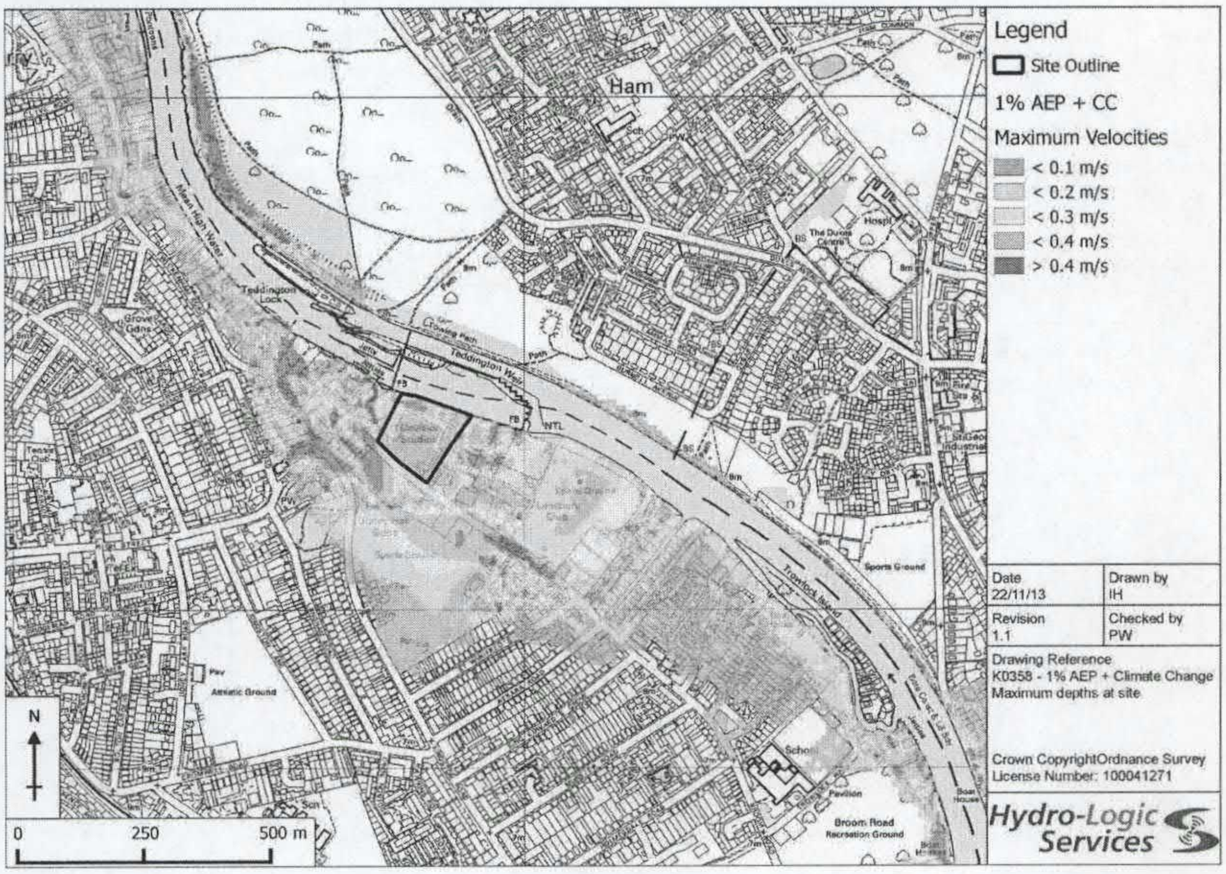


Figure 4-11 Maximum velocities: 1% CC



The proposed layout makes provision for an equivalent flow path as a culvert beneath the Piazza (Figure 4-13). The proposal is shown in plan, section and long section in Figure 4-14. It features twin channels, either side of Building D, with the following characteristics:

- The culverts will be formed from concrete channels.
- Each culvert will comprise 3 channels, each of 1.1 m width, giving a width of 3.3 m in each culvert and 6.6 m width overall.
- Where it flows under the Piazza for a length of 9 m, the top of the culvert will be formed from steel plate sections, bolted to the top of the side walls of the channels. The use of a steel plate means that it can be much thinner than (say) the deck for a conventional concrete culvert.
- This in turns means that there can be free flow through the culvert for water levels approaching 6.7 mAOD, the soffit of the culvert. This is well above the existing 1% flood fluvial flood level (6.38 mAOD – Table 3-5 of FRA, page 23).
- The approaches to the culvert would be protected by a flip-up barrier. In its normal (non-flood) condition, the barrier would be raised. It would be lowered under flood conditions to allow water to flow through the culverts.

In the FRA, the width of the existing opening at the Gatehouse is stated as 7 m. This is virtually matched by the 6.6 m width of the twin culverts. Furthermore, there is a further flow route of approximately 1 m width on the eastern side of the site (Figure 4-15). Thus, in very simple terms, the proposal offers a flow route that is of similar hydraulic width to the existing flow route. More detailed hydraulic consideration is not justified, given that it would be very challenging and given the fact that in major floods exceeding 6.41 mAOD (ie the existing ground level at the Gatehouse), any such flow routes are likely to be of limited significance.

Figure 4-12 The Gatehouse – a potential flow route across the site

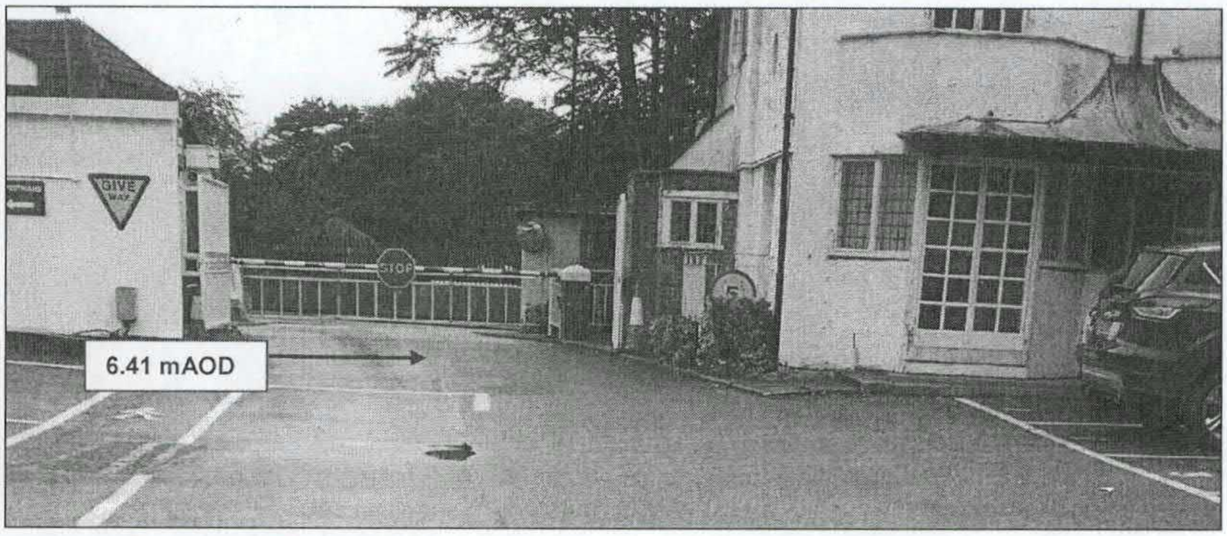


Figure 4-13 Proposed flow paths across the site

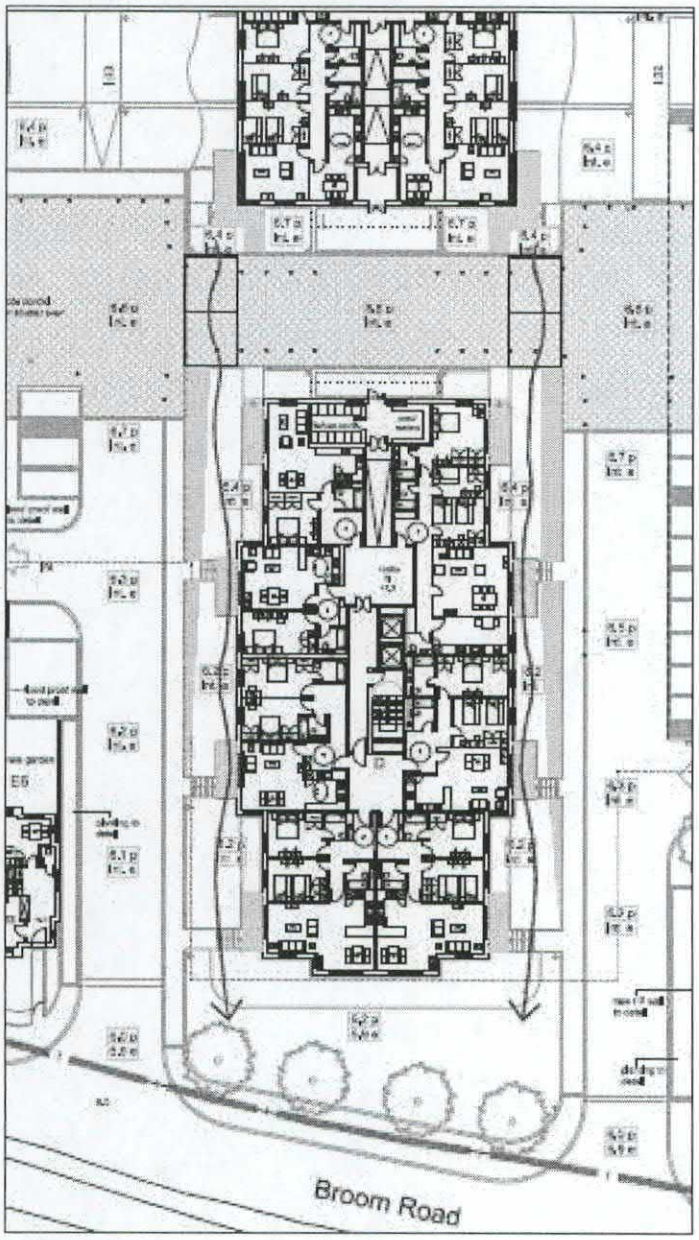


Figure 4-14 Sections through Piazza showing twin culverts (Drawing C0800 P3)

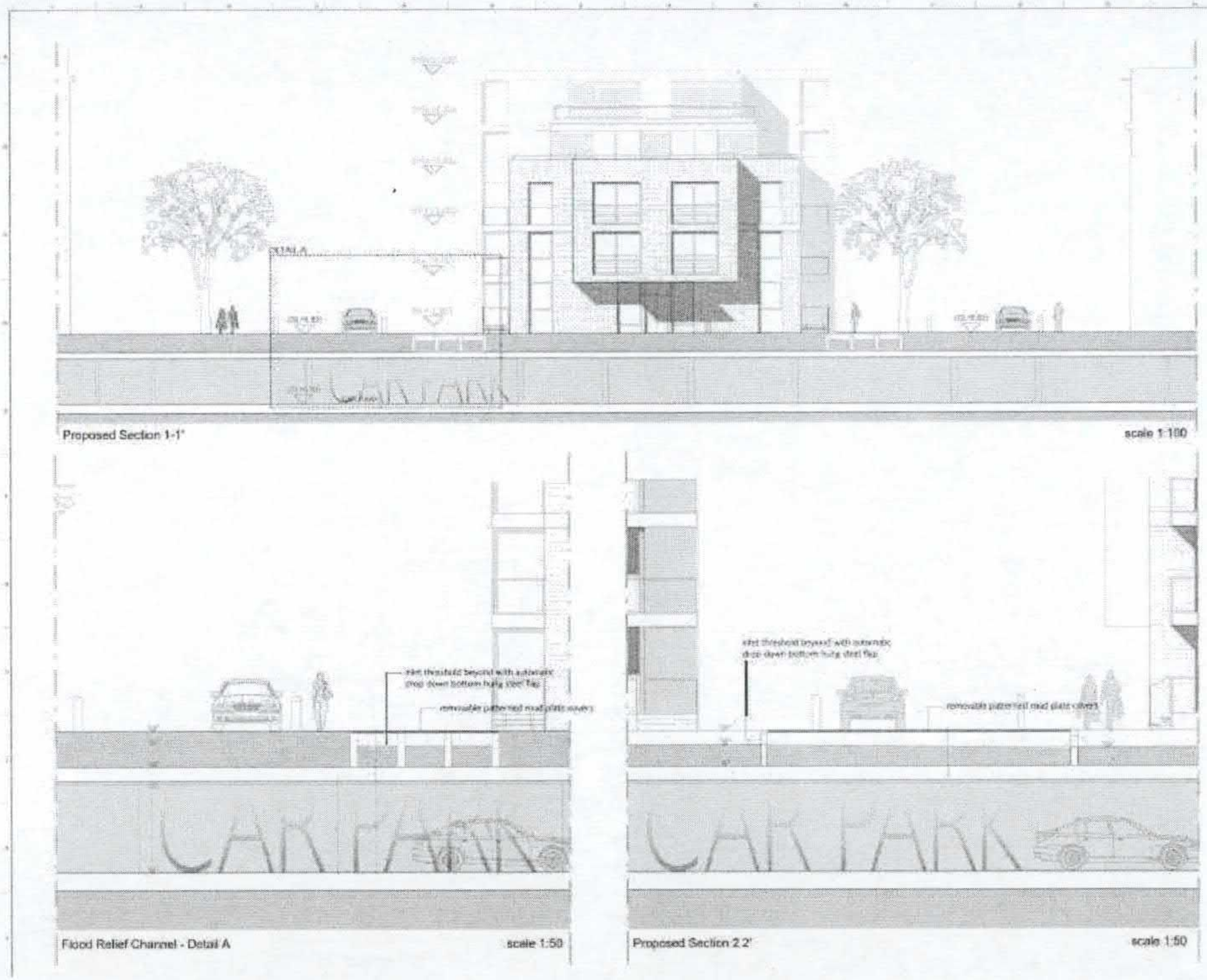
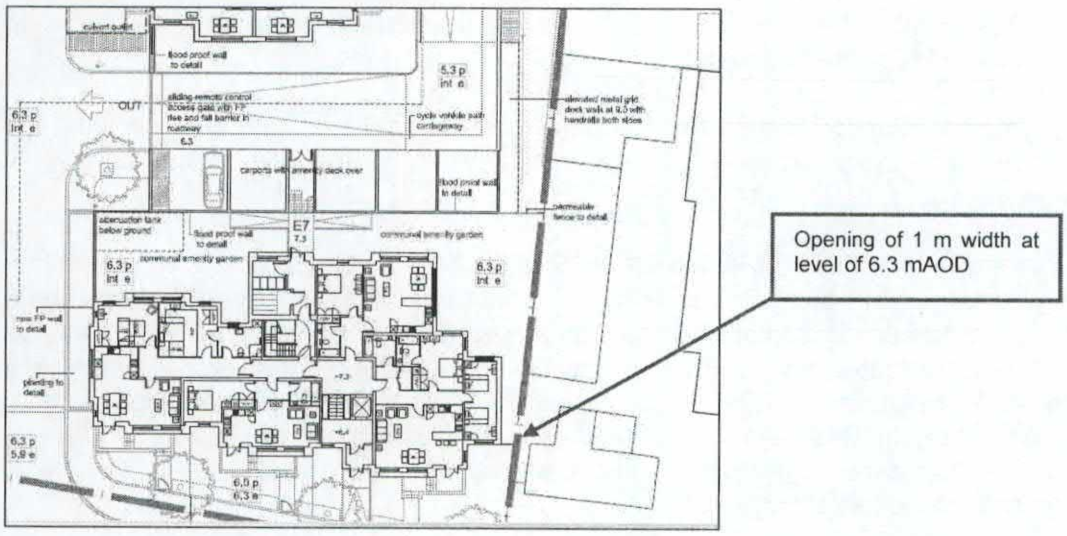


Figure 4-15 Flow path on Eastern boundary of site



4.3.2 Flood Plain storage

(a) General

It is a requirement under NPPF that there should be no loss of flood plain storage resulting from a development. The flood plain is defined by the 1% CC extent. The flood storage available on the site has been evaluated for the existing layout, by obtaining the floodable area at increments of 0.1 m up to 7.0 mAOD. This has then been compared with the floodable areas for the proposed development footprint. In general and in order to satisfy the "level-for-level" principle, the floodable area for the proposed development should exceed that for the existing layout for all level increments.

(b) Flood storage for existing site

The existing flood storage calculation has been based on the survey undertaken by Marchfield Surveys Ltd and shown on Drawing 2371 Rev A, dated April 2011 [sic]. On the plan, it is stated that the levels are related to EA BM (STN02) shown on survey value 5.238 m.

A site visit was undertaken on 3rd October 2013 to check if the buildings shown on the plan were likely to be able to resist water ingress. An extract from the survey plan is shown in Figure 4-16 along with photographs taken on that day. It was possible to confirm that the bold building boundary shown on the survey drawing is appropriate as a basis for flood storage computation as it demarcates the walls at **ground level**.

The visit also confirmed the extent of the multi-storey car park which would clearly be permeable and contribute to flood storage.

The doors and access points to all buildings were reviewed and a selection is shown in Figure 4-17 and Figure 4-18. Whilst some would not currently be classed as "flood resistant", they were all amenable to protection by the current occupiers, in order to exclude flood water.

The existing contours for the site are shown in Figure 4-19, along with the buildings that do not contribute to flood storage. Note that the multi-storey car park on the eastern boundary of the site is assumed to contribute fully to flood storage. These contours are based on the DTM; however, the DTM elevations were found to be unreliable along the boundary to the Thames. For this reason, the flood storage has been reviewed separately for the "riverside" and "development side" of the tidal defence.

The flood storage calculation for the "development side" is shown in Table 4-3, showing that there is 11,202 m^3 of storage on the site on the development side of the defences, of which **3,251 m^3** is below a level of 6.1 mAOD.

The storage on the riverside of the defences is topographically quite simple, being level and of reasonably uniform cross section. Given the unreliability of the DTM and the challenge of interpolating contours from limited spot elevation data from the topographic survey, a manual approach has been adopted for evaluating existing flood storage. The storage has been evaluated for three sections, A, B and C as shown in Figure 4-20. The resultant storage volumes are presented in Table 4-4 which shows that there is **498 m^3** of storage to a level of 6.1 mAOD on the "riverside" – giving a total flood storage volume of **3,749 m^3** (3,251 + 498) for the site as a whole to this elevation.

Figure 4-16 Review of buildings on eastern margin

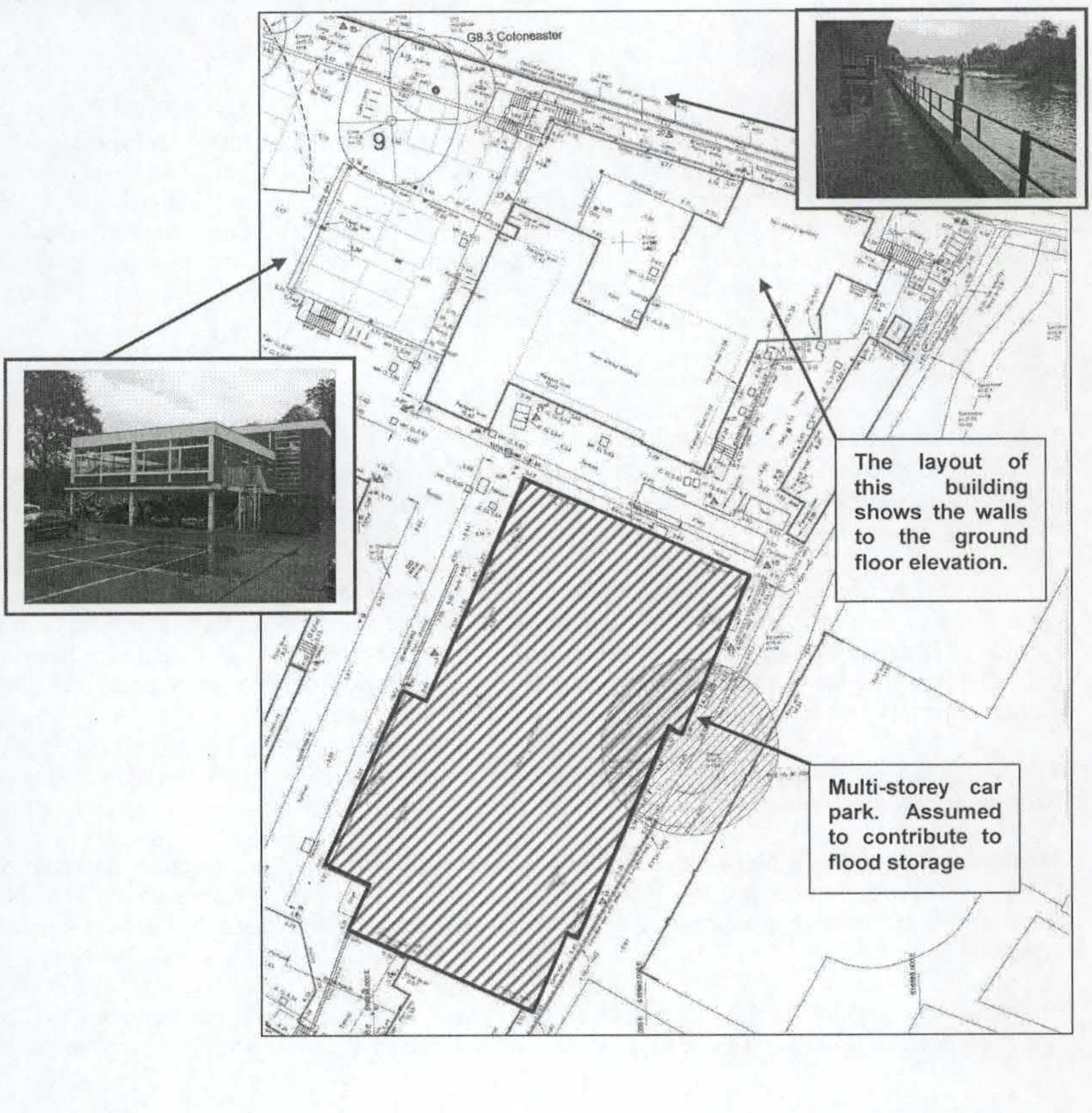


Figure 4-17 Selection of doorways and access points to buildings on the site

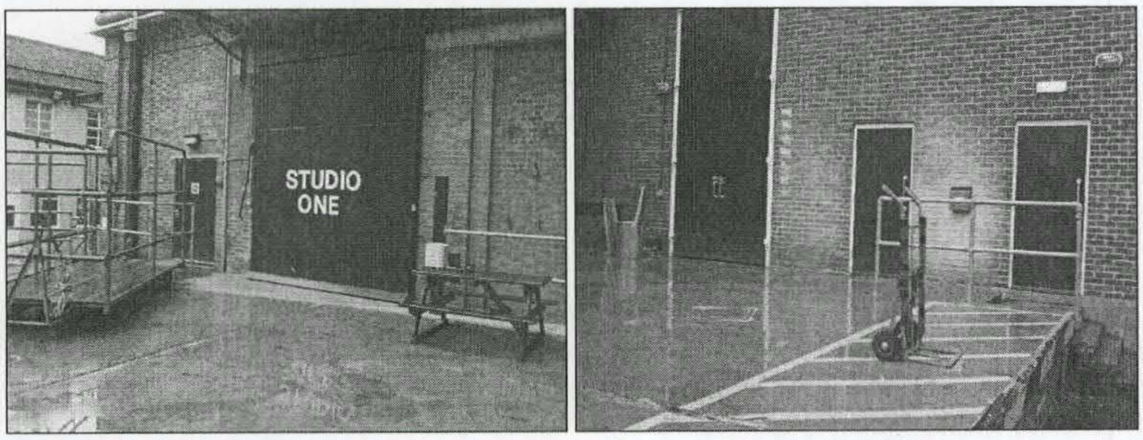


Figure 4-18 Selection of doorways and access points to buildings on the site

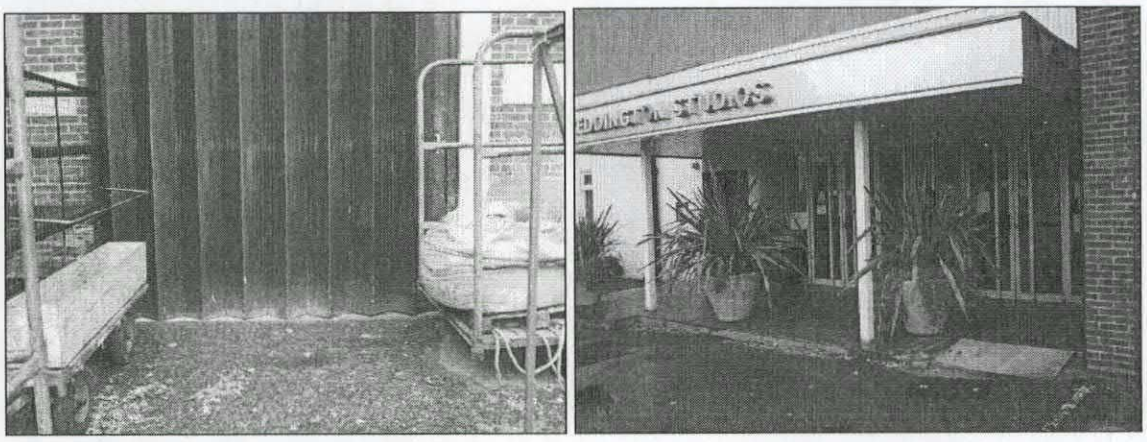


Table 4-3 Flood Storage for existing site: "Development side"

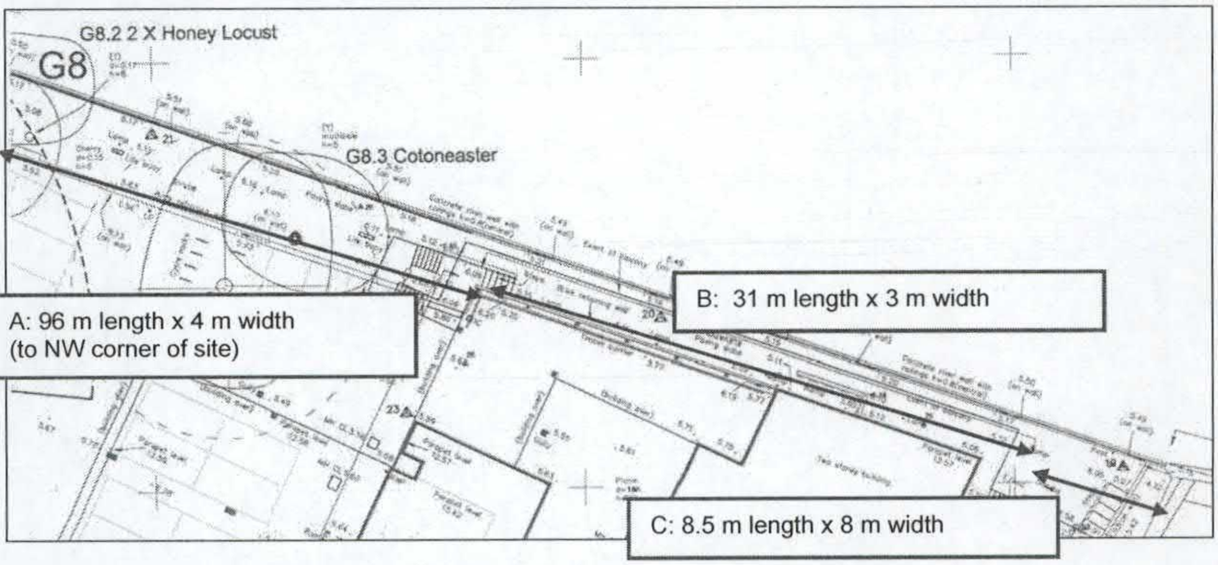
Lower (mAOD)	Upper (mAOD)	Depth (m)	Σ Area (m^2)	Area (m^2)	Σ Volume (m^3)
6.9	7	0.1	9,148	0	11,211
6.8	6.9	0.1	9,148	0	10,296
6.7	6.8	0.1	9,148	0	9,381
6.6	6.7	0.1	9,148	0	8,466
6.5	6.6	0.1	9,148	8	7,551
6.4	6.5	0.1	9,140	164	6,637
6.3	6.4	0.1	8,976	290	5,731
6.2	6.3	0.1	8,686	634	4,848
6.1	6.2	0.1	8,052	906	4,011
6	**6.1**	**0.1**	**7,146**	**804**	**3,251**
5.9	6	0.1	6,342	618	2,577
5.8	5.9	0.1	5,724	886	1,974
5.7	5.8	0.1	4,839	844	1,445
5.6	5.7	0.1	3,995	993	1,004
5.5	5.6	0.1	3,002	1,329	654
5.4	5.5	0.1	1,673	0	420
5.3	5.4	0.1	1,673	748	253
5.2	5.3	0.1	925	501	123
5.1	5.2	0.1	424	164	55
5	5.1	0.1	260	96	21
4.9	5.0	0.1	164	0	

Table 4-4 Flood Storage for existing site: "Riverside"

Lower (mAOD)	Upper (mAOD)	Depth (m)	Σ Area (m^2)	Area (m^2)	Σ Volume (m^3)	Comment
	To 7.0	0.1	544	0	987	
	To 6.1	**0.1**	**544**	**0**	**498**	
5.4	5.5	0.1	544	32	171	Soil c0.3 m high, 1 m wide
5.3	5.4	0.1	512	32	118	Soil c0.3 m high, 1 m wide
5.2	5.3	0.1	480	32	69	Soil c0.3 m high, 1 m wide
5.1	5.2	0.1	448	448	22	

Figure 4-19 Contours for existing site

Figure 4-20 Flood storage: "Riverside" assumptions



(c) *Flood storage for proposed development*

The proposed levels for the development have resulted from iterative discussions between the principal architect, landscape architect and flood risk consultant. They are shown in Figure 4-21 (see also the landscape layout in Appendix F). The proposed layout has been developed in order to satisfy key storage requirements, namely:

- Preserving the existing flood plain storage volume up to the crest level of the existing defences (i.e. up to 6.1 mAOD) on both the "development" side and the "riverside".
- Preserving, on a level-for-level basis and volumetric basis, the flood plain storage above 6.1 mAOD;

For the development side of the defences, Table 4-5 demonstrates that the proposed layout satisfies the requirements of both volumetric and level-for-level. At the critical elevation of 6.1 mAOD, the table shows that there will be an increase in available flood storage of **329 m^3** (3,580 – 3,251).

For the riverside of the defences, the relative simple geometry makes it clear that with a base area of 558 m^2 for the riverside walkway, the volume is also increased as shown in Table 4-6. For the critical elevation of 6.1 mAOD, the volume increases by **72 m^3** (569 – 498). This gives an overall increase in flood storage at that elevation of **401 m^3**.

Recent revisions to the alignment of the defences have meant that it is necessary to make use of flood storage provision beneath Block C (Figure 4-21). An area of 300 m^2, will be utilised for this, with its base at 4.0 mAOD (Figure 4-22). This will be dedicated for flood storage, with access only for maintenance purposes. It will fill via openings at an elevation of 5.6 mAOD; which is the level of bounding garden, but lower than the level of the existing defences at 6.1 mAOD. It will drain via a 300 mm flapped outfall to the Thames with invert at 3.8 mAOD.

Provision has been made for a further void with plan area of 250 m^2 beneath Block A (Figure 4-21). This will have a base level of 6.1 mAOD, extending to at least 6.5 mAOD and provides "fine tuning" to satisfy the specific requirements of level for level compensation at these elevations (Figure 4-22). The void will fill and drain via two openings, protected by grilles, of nominal width 1 m. In order to improve internal accessibility for maintenance, the soffit of the void may be increased to 6.8 mAOD; the 0.7 m depth providing a slightly more accessible space.

Flood water will enter the voids under Buildings A and C via openings in each void. The openings are shown on drawings D0099 P4, D0003 P5, D0201 P4 & D0203 P5 and have been designed such that their width is over 20% of the external wall to the void. Extracts from D0003 P5 are shown in Figure 4-23. The height would be 0.6 m. Under normal conditions, the opening will be protected by a barrier; this will restrict any entry into the void.

Under flood conditions with floodwater likely to overtop the tidal defence wall at 6.1 mAOD and flow into the garden, the barriers will be lowered allowing water to flow into the voids.

The mechanism for this will be a "flip down barrier". In its normal position, the barrier will prevent entry of water (or people) into the void from the garden. Under flood conditions, the barriers would be lowered, hydraulically, to allow water to flow into the void. Provision must be made in the design for these barriers to be lowered manually.

Although some of the drawings have shown that the void under Buildings A and C is sub-divided into sub voids, in discussions with the Environment Agency, it was agreed that a single void would be most practical. Drawings also show a "Lock"; this is to facilitate entry into the void should this be required.

Table 4-5 Flood Storage for proposed site: "development side"

Level (mAOD)	Existing Area (m^2)	Existing Inc Area (m2)	Existing Vol (m3)	Proposed Total Area (m^2)	Proposed Inc Area (m2)	Inc. Area increase?	Proposed Volume (m^3)	Difference (m^3)	Volumetric increase?
7	9,148	0	11,211	10,803	15	Yes	12,084	874	Yes
6.9	9,148	0	10,296	10,788	0	Yes	11,005	709	Yes
6.8	9,148	0	9,381	10,788	1,078	Yes	9,926	545	Yes
6.7	9,148	0	8,466	9,710	227	Yes	8,901	435	Yes
6.6	9,148	8	7,551	9,483	137	Yes	7,941	390	Yes
6.5	9,140	164	6,637	9,346	189	Yes	7,000	363	Yes
6.4	8,976	290	5,731	9,157	440	Yes	6,075	344	Yes
6.3	8,686	634	4,848	8,717	646	Yes	5,181	333	Yes
6.2	8,052	906	4,011	8,071	916	Yes	4,342	330	Yes
6.1	**7,146**	**804**	**3,251**	**7,155**	**822**	**n/a**	**3,580**	**329**	**Yes**
6	6,342	618	2,577	6,333	906	n/a	2,906	329	n/a
5.9	5,724	886	1,974	5,427	19	n/a	2,318	344	n/a
5.8	4,839	844	1,445	5,408	306	n/a	1,776	331	n/a
5.7	3,995	993	1,004	5,102	78	n/a	1,251	247	n/a
5.6	3,002	1,329	654	5,024	4,723	n/a	744	90	n/a
5.5	1,673	0	420	301	0	n/a	478	58	n/a
5.4	1,673	748	253	301	0	n/a	448	195	n/a
5.3	925	501	123	301	1	n/a	418	295	n/a
5.2	424	164	55	300	0	n/a	388	332	n/a
5.1	260	96	21	300	0	n/a	358	337	n/a
5	164	0		300	0				

First 4 columns from Table 4-3

Table 4-6 Flood Storage for proposed site: "Riverside"

Level (mAOD)	**Σ Area (m^2)**	**Σ Volume (m^3)**	**Proposed volume (m^3)**	**OK?**	**Comment**
To 7.0	544	987	1,092	Yes	
To 6.1	544	498	569	Yes	Increase in area due to steps to 6.1
To 5.5	544	171	225	Yes	Base of walkway at 5.1 (area of 558 m^2)

First 3 columns from Table 4-4

The cross-sections that are shown in Figure 4-24 confirm that the gradation of levels within the site is such that all storage areas can fill and drain freely. The location of the cross sections is shown in Figure 4-21. Parts of the site adjacent to Broom Road will drain down towards the road whilst parts of the site adjacent to the river will drain through flap valves through the tidal defences.

The analysis presented above has demonstrated that the proposed development will lead to an increase in available both floodable area and flood storage over all elevations. However, there is further minor intervention into flood storage areas that needs to be reviewed. These are of such a small scale that it is not appropriate to consider them as part of the calculations already completed. They are therefore addressed separately below.

Firstly, the Landscape Plan on which Figure 4-21 has been based also shows some soil embankments adjacent to the walls of the principal Blocks where they project in into the gardens. These are purely for landscape purposes and serve no flood related purpose. These soil embankments will further reduce the available flood storage. A separate calculation has been made to show that the development still satisfies the flood storage requirements. The location of the soil embankments is shown in Figure 4-25 along with their base elevations. The embankments will have a width of 0.9 m, a slope of 1 in 3 and a height therefore of approximately 0.3 m.

A separate calculation has therefore been undertaken to compare the available flood area at each elevation, with the area taken up by the soil embankments. These are reviewed separately below, for embankments above and below the level of the existing defences at 6.1 mAOD in the penultimate column of Table 4-7.

- For the embankments with their base at 5.6 mAOD, all storage will be below 6.1 mAOD and there is only a need for a volumetric provision. The volume of the embankments is easily calculated from their combined length of 315 m and the cross-sectional area of 0.135 m^2. This gives a storage of 42.5 m^3 that can easily be accommodated with in the available storage provision up to 6.1 mAOD of 317 m^3.
- For other embankments, the demonstration of level-for-level compliance is demonstrated in Table 4-7 for all elevations above 6.1 mAOD.

The total volume of these soil embankments is given by the product of the combined length (484 m) and cross-sectional area (0.135 m^2). This volumetric loss of 65 m^3 is within the available storage from Table 4-5.

Figure 4-21 Proposed levels for flood storage calculation

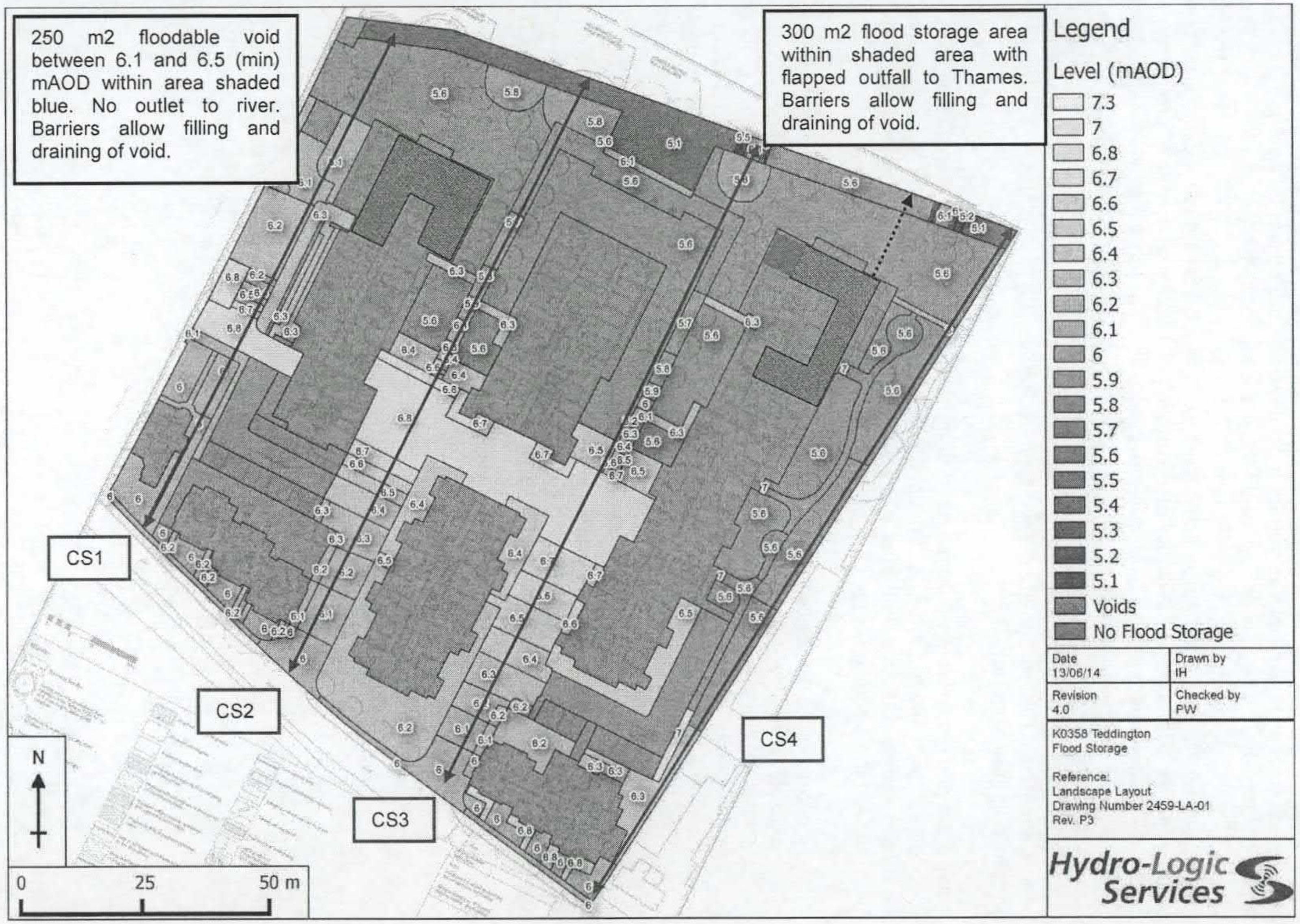
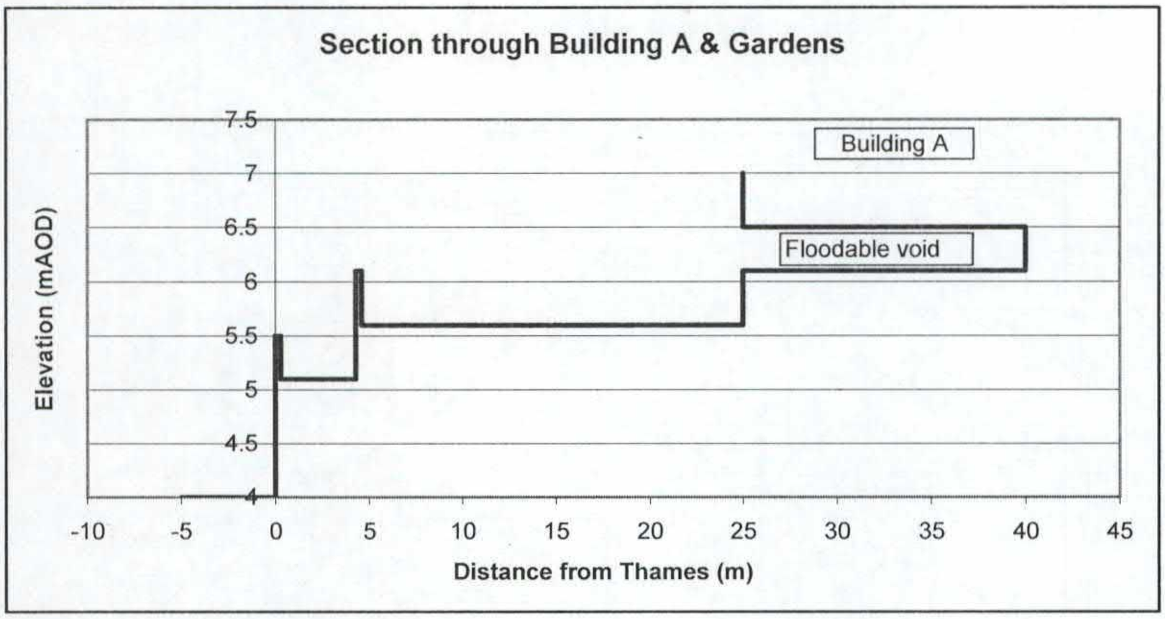


Figure 4-22 Cross sections showing floodable voids



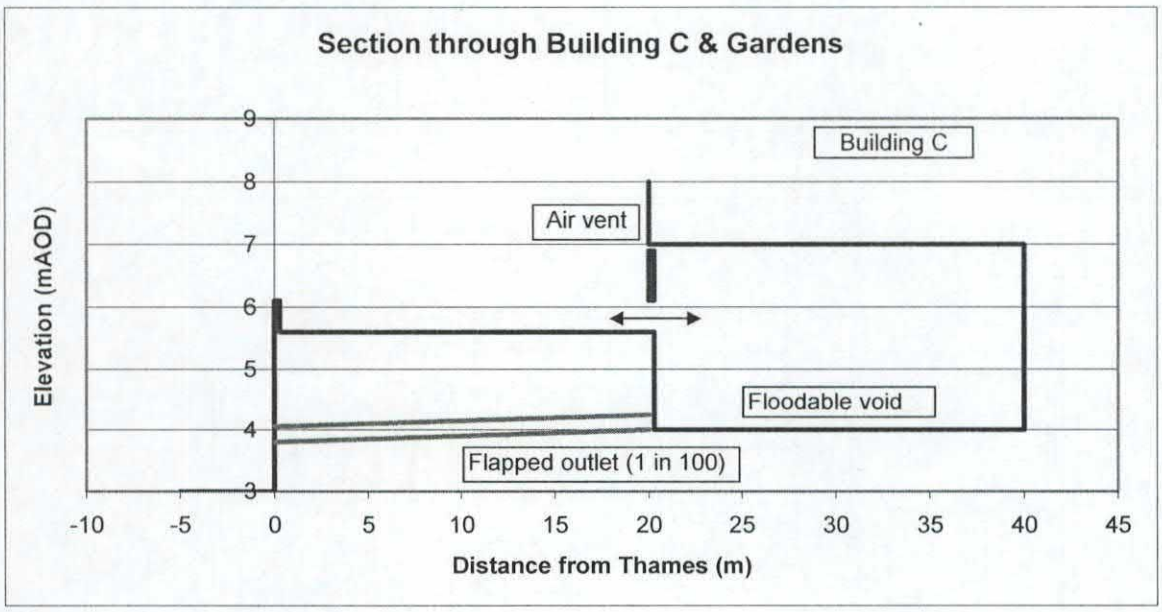
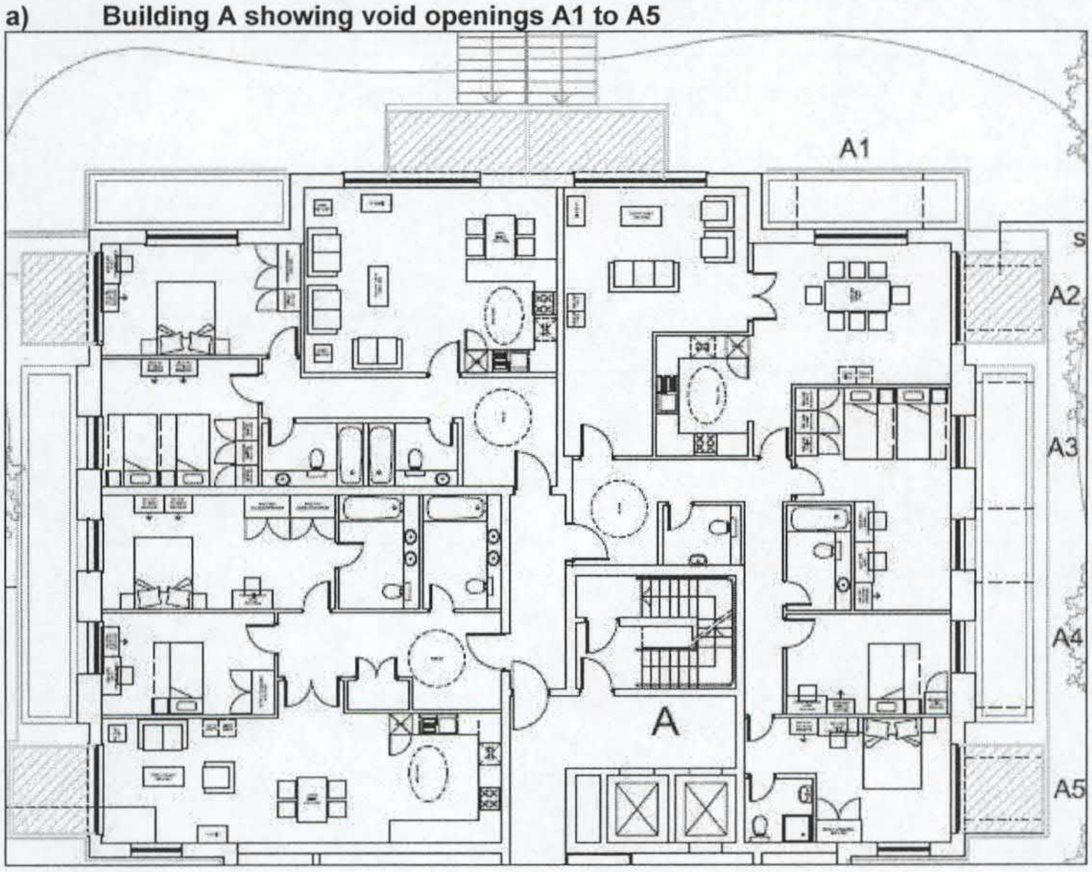


Figure 4-23 Location of openings to floodable voids

a) Building A showing void openings A1 to A5



b) Building C showing void openings C1 to C3

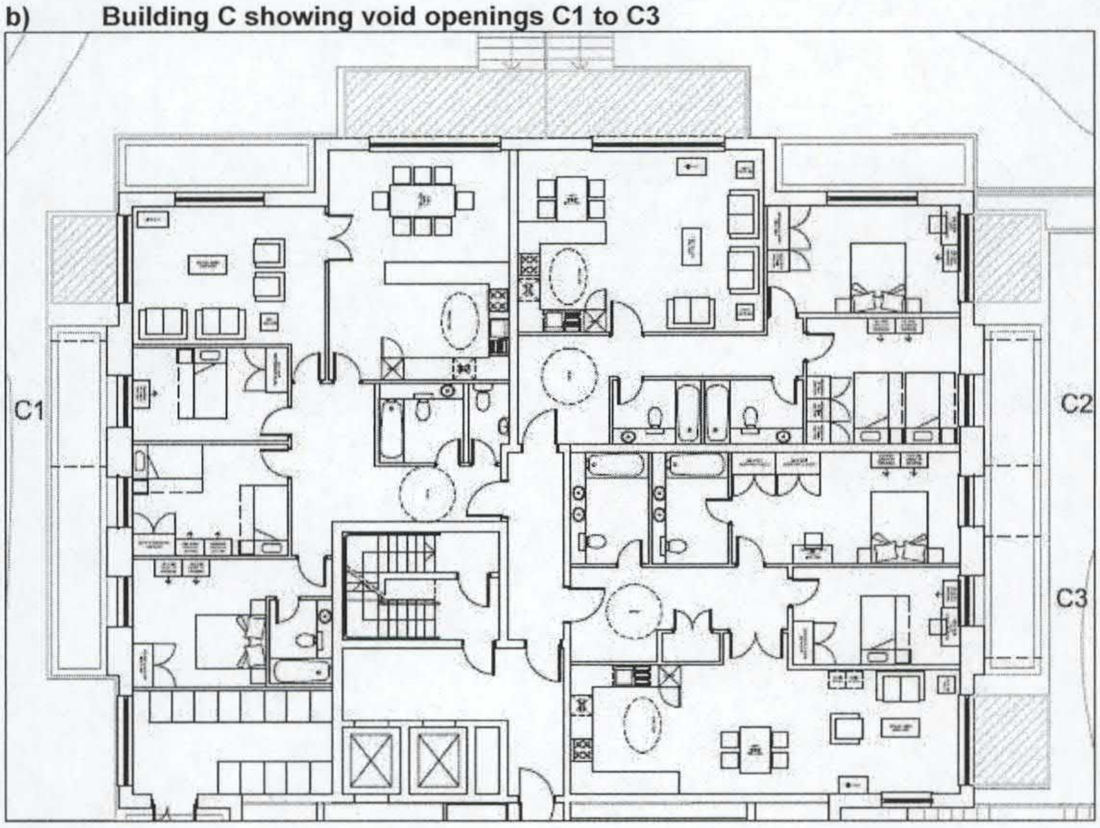


Figure 4-24 Cross sections through the site

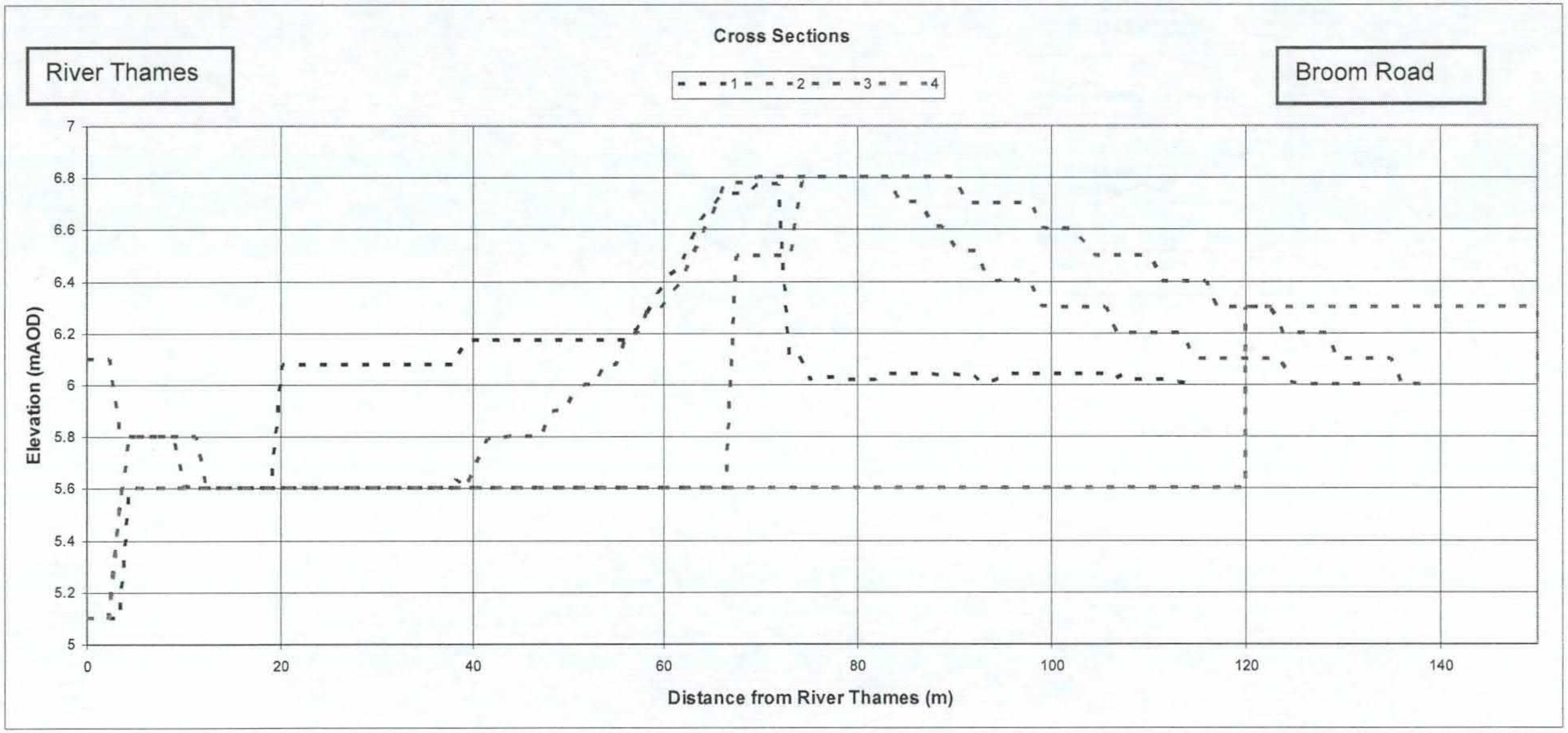
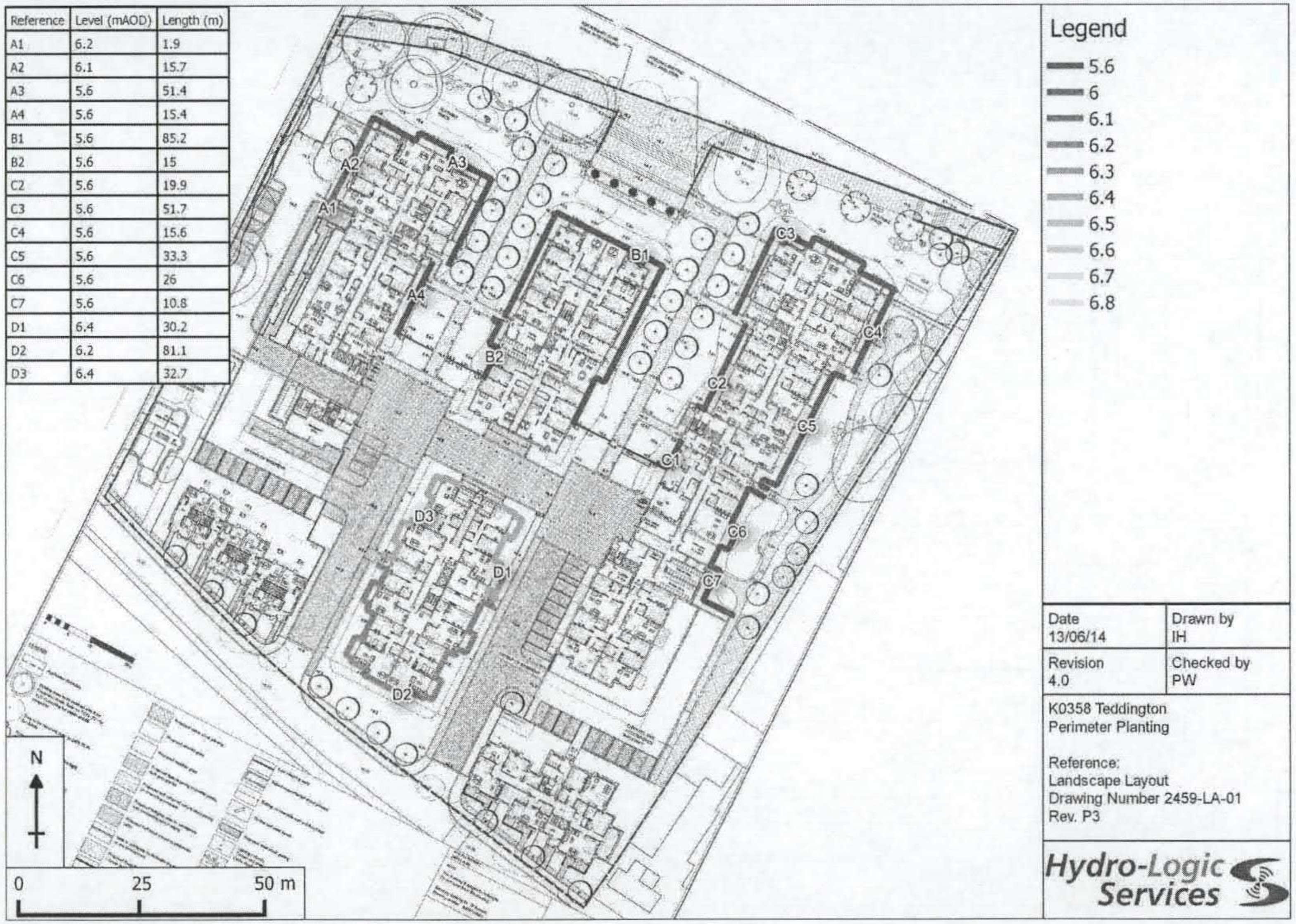


Figure 4-25 Soil embankments around main buildings



Reference	Level (mAOD)	Length (m)
A1	6.2	1.9
A2	6.1	15.7
A3	5.6	51.4
A4	5.6	15.4
B1	5.6	85.2
B2	5.6	15
C2	5.6	19.9
C3	5.6	51.7
C4	5.6	15.6
C5	5.6	33.3
C6	5.6	26
C7	5.6	10.8
D1	6.4	30.2
D2	6.2	81.1
D3	6.4	32.7

Table 4-7 Provision of flood storage for soil embankments

Level (mAOD)	Available Volume (m3)	Length at 5.6 mAOD (m)	Length at 6.2 mAOD (m)	Length at 6.4 mAOD (m)	Length at 6.5 mAOD (m)	Length at 6.7 mAOD (m)	Plan area of embankment (m2)	Volume of embankment (m3)	Level for Level satisfied?
Length of embankment (m)		315	83	63	13	11			
6.8	545					3.3	3.3	0.7	Yes
6.7	435				1.3	9.9	11.2	1.1	Yes
6.6	390			6.3	3.9		10.2	2.0	Yes
6.5	363			18.9	11.7		30.6	4.8	Yes
6.4	344		8.3	56.7			65	4.5	Yes
6.3	333		24.9				24.9	5.0	Yes
6.2	330		74.7				74.7	3.7	Yes

(d) Summary

It has been shown in this Section that the proposed development complies with the requirements of volumetric and level-for-level flood plain compensation, both in relation to the "riverside" and "development side" of the flood defences.

The analysis has also demonstrated that the volume of minor interventions, relating to soil embankments for landscape purposes (65 m^3) can be accommodated within the contingency.

There is in fact a net increase in flood storage across all elevations. After allowing for the two minor interventions, this is around 250 m^3 at 6.1 mAOD increasing to around 800 m^3 at 7.0 mAOD, providing contingency in the event of further landscape revisions.

4.3.3 Runoff

(a) General

It is a requirement under NPPF that there should be no increase in peak rates of runoff arising from the development. In the SFRA, there is a target that developers should achieve a 50% reduction of the pre-development rate of runoff for the 1 in 100 (1%) storm, whilst the London Plan seeks a reduction to Greenfield rates. Since the proposed development involves a substantial reduction of the impermeable area on the site, the development leads to a profound reduction in runoff.

In this Section, the existing rates of runoff are compared with post-development rates and scope for further reduction, using a SUDS methodology is discussed.

(b) Runoff from the existing site

The existing site is essentially 100% impermeable, comprising roofs, car parking and other hard standing. The 1% peak rate of runoff for the site has been evaluated using the Marshall & Bayliss method (Institute of Hydrology, 1994) and is 12.985 l/s/ha (Table 4-8) – equivalent to **24.0 l/s** for a site area of 1.86 ha. The mean greenfield rate of runoff has also been evaluated and is 1.522 l/s/ha (Table 4-9). The 1% greenfield rate of runoff is 4.854 l/s/ha – equivalent to **9.0 l/s** for the site. The 1% peak rate of runoff is thus almost 3 times the greenfield rate of runoff.

These calculations assume that the site lies in the WRAP (Winter Rain Acceptance Potential from the Flood Studies Report (1975)) class 2 which gives a SOIL index of 0.3 as shown in Table 4-9. This is somewhat higher than the SPRHOST value of 0.19 (19%) in Table 3-1.

In Section 3.4, it has been shown that the site drains either to sewers in Broom Road, operated by Thames Water, or discharges to the Thames at the north-west corner of the site. This stormwater tank may achieve some attenuation of runoff, though the size of the flapped outfall (Figure 3-13) would not appear to provide much hydraulic control.

Table 4-8 Evaluation of peak rate of runoff

Runoff estimation for site

Date: 15/11/2013

Methodology based on Marshall D.C.W. & Bayliss A.C., 1994. Flood estimation for small catchments, IH Report No. 124, Institute of Hydrology, Wallingford and Hall, Hockin & Ellis

AREA	0.0186
SAAR4170	600
SOIL	0.30
URBAN-pre	1
URBAN-post	0.22
CWI	95
REGION	Thames/Southern

	Runoff (l/s/ha)			**Runoff (l/s)**		
Return period (years)	**Greenfield**	**pre-devt**	**post-devt**	**Greenfield**	**pre-devt**	**post-devt**
QBAR	1.522	7.321	2.364	2.830	13.618	4.397
2		7.744	2.178		14.404	4.051
5		9.988	3.070		18.578	5.711
10		10.830	3.771		20.144	7.014
20		11.481	4.488		21.355	8.347
25		11.622	4.729		21.618	8.795
50		12.165	5.395		22.626	10.035
100		**12.895**	**6.325**		**23.985**	**11.765**
200		13.400	7.196		24.924	13.385
250		13.526	7.469		25.158	13.892
500		13.849	8.298		25.759	15.434
1000		14.415	9.303		26.812	17.303

Note: Post development runoff refers to the "critical" impermeable proportion of 22% to achieve a 50% reduction for the 1% peak rate of runoff.

Table 4-9 Evaluation of greenfield rate of runoff

QBAR estimation - generally used for small (>0.5km^2 <25km^2) rural catchments
Calculates mean annual flood for a rural catchment drained by well defined water course
Higher return periods can be calculated using the appropriate *flood studies report* regional growth curve

AREA	0.5	Catchment area in km^2
SAAR4170	600	Standard Average Annual Rainfall based on the period 1941 to 1970
S_1		Proportion of soil type within catchment (e.g. If 70% of catchment is Soil type 3 then S_3 = 0.7)
S_2	1	
S_3		
S_4		
S_5		
SOIL	0.30	SOIL = $S_1\,0.15 + S_2\,0.3 + S_3\,0.4 + S_4\,0.45 + S_5\,0.5$
QBAR-r	0.08	$QBAR_{rural} = 0.00108\ AREA^{0.89} SAAR^{1.17} SOIL^{2.17}$ = Cumecs
QBAR-r	1.52	l/s/ha

Marshall D.C.W. & Bayliss A.C., 1994. Flood estimation for small catchments, *IH Report No. 124*, Institute of Hydrology, Wallingford,

(c) Drainage strategy

The approximate breakdown of the site into impermeable and permeable areas is shown in Table 4-10, based on Figure 4-26 and the Design & Access Statement. This shows that 39% of the site area would be classed as impermeable. The analysis in Table 4-8 shows that in order to achieve a 50% reduction in peak runoff, the impermeable proportion would need to be below 22%. This indicates that some mitigation measures are required if the LBRT requirement is to be met and even moreso if peak rates of runoff are to be reduced to the greenfield rate.

Table 4-10 Surface cover for the developed site

Building	Area	Area	Soakaway	Main tank	Small tank
	(m^2)	(%)			
Block A: roof	1,002		1,002		
Block B: roof	825		825		
Block C: roof	1,665		1,000	665	
Block D: roof	739		175	564	
Block E7: roof	385				385
Town House (x6): roof	281		281		
F (weir Cottage):roof	118		118		
Sub-total	**5,015**	**27%**	**3,401**	**1,229**	**385**
Other imp. areas	2,172	12%		2,172	
Total imp. area	**7,187**	**39%**	**3,401**	**3,401**	**385**
Permeable area	11,413	61%			
Total site area	**18,600**	**100%**			

The review of SUDS selection is presented in Table 4-11 in relation to the developed site. This shows that there is limited potential for most of the measures due to limitations of area or compatibility with the development concept. However, the measures which may have some scope, shown in bold text in the Table, are Infiltration, Detention and Source Control.

Table 4-11 SUDS Selection Review

SUDS Group	Technique	Comment
Retention	Pond/Storage	Not compatible with development concept
Wetland	Pond/Channel	Not compatible with development concept
Infiltration	Trench/Basin/**Soakaway**	Subject to the results of Site Investigation, and exposure of natural geology, **soakaways appropriate for roof drainage**.
Filtration	Sand/vegetated strips	Insufficient area to accommodate these features
Detention	**Basin/Tanks**	**Tank for attenuation of roof drainage**
Open channels	Swales	Insufficient area to accommodate these features
Source control	**Permeable paving, green roof, rainwater harvesting**	**Scope for source control**

Based on Table 5.9 (CIRIA, 2007)

The strategy for dealing with stormwater on different parts of the site is shown in Figure 4-27 and uses the areas presented in Table 4-10. In broad terms, this will have the following components:

- A stormwater attenuation tank with an outfall to the River Thames for the balance of roof runoff on Buildings C and D plus other impermeable surfaces. ;
- A small stormwater attenuation tank beneath the Affordable Housing with outfall to the Thames Water sewer.
- Three large Soakaways for dealing with a portion (1,000 m^2 per soakaway) of the runoff from the Blocks A, B and C;
- A small soakaway for roof runoff from Weir Cottage and the Town Houses
- Although the use of green roofs may form part of the final design, all roofs have been treated as impermeable for the purposes of this modelling.

Whilst it is currently envisaged that the main tank could be built beneath Block B, the SI may reveal that the volume occupied by the existing tank in the north west corner of the site is suitable, in part, or in full, thereby reducing the volume of any tank under Block B.

The drainage strategy presented in this Section has been informed by output from the MicroDrainage software, in evaluation of critical storage requirements. The results from a series of simulation runs are shown in Table 4-12; supporting material is included in Appendix G . The key assumptions for these simulations are as follows:

- Point rainfalls have been obtained from the FEH CD-ROM v3 (CEH, 2009). These have been augmented by 30% to allow for climate change in line with the requirements of NPPF.
- Factor of safety of 1.5, in recognition of the minor consequences of any design exceedance.
- A runoff coefficient of 100% has been adopted for a nominal contributing area of 1,000 m^2.
- The infiltration coefficient of 0.05 m/h has been used, based on values for loam and sandy loam in Table 4.7 in CIRIA (2007).
- A porosity of 0.95 would be appropriate for a soakaway filled with crated modules.
- The peak outflow has been set as the 1 in 100 year peak greenfield runoff of 9 l/s.
- Hydraulic control is to be achieved using a hydrobrake.
- No green roofs have been assumed.

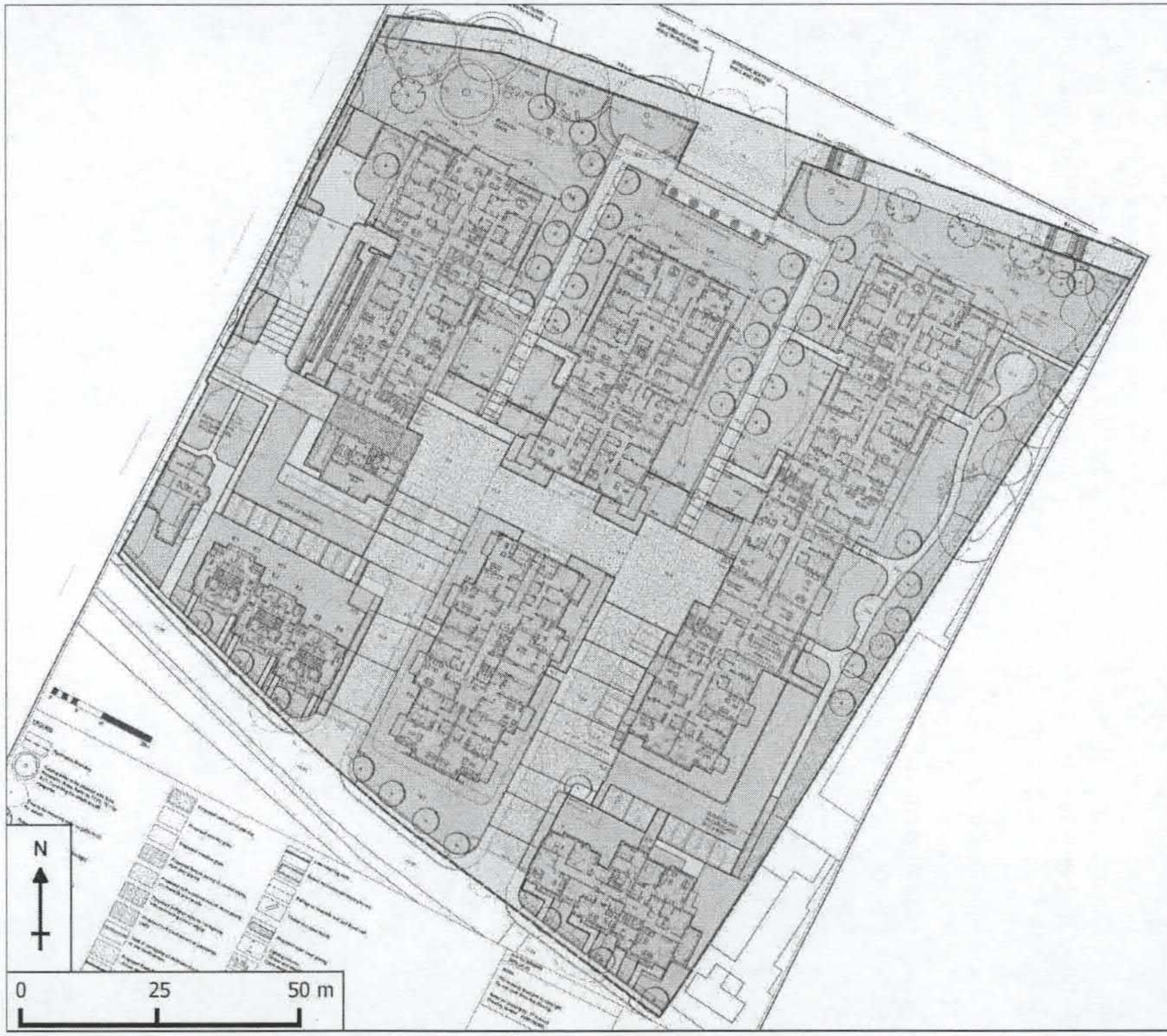
In broad terms, this shows that the proposed devices are able to attenuate storm runoff to the greenfield rate. The strategy and results are reviewed further in the next Section.

Table 4-12 Summary results of MicroDrainage simulation runs

Device	Impermeable Area (m^2)	Storage plan area (m^2)	Storage depth /max water depth (m)	Peak 1%CC outflow (l/s)	Required storage volume (m^3)
Main_tank	3,412	250	1/0.863	4.9	**205**
Secondary_tank	385	20	1/0.709	4	**13.5**
Main_soakaway	1,000	90	0.8/0.744	N/A	**63.6**
Small_soakaway	399	20	1.5/1.411	N/A	**26.8**

Results are presented in Appendix G to support these results

Figure 4-26 Surface cover for proposed development



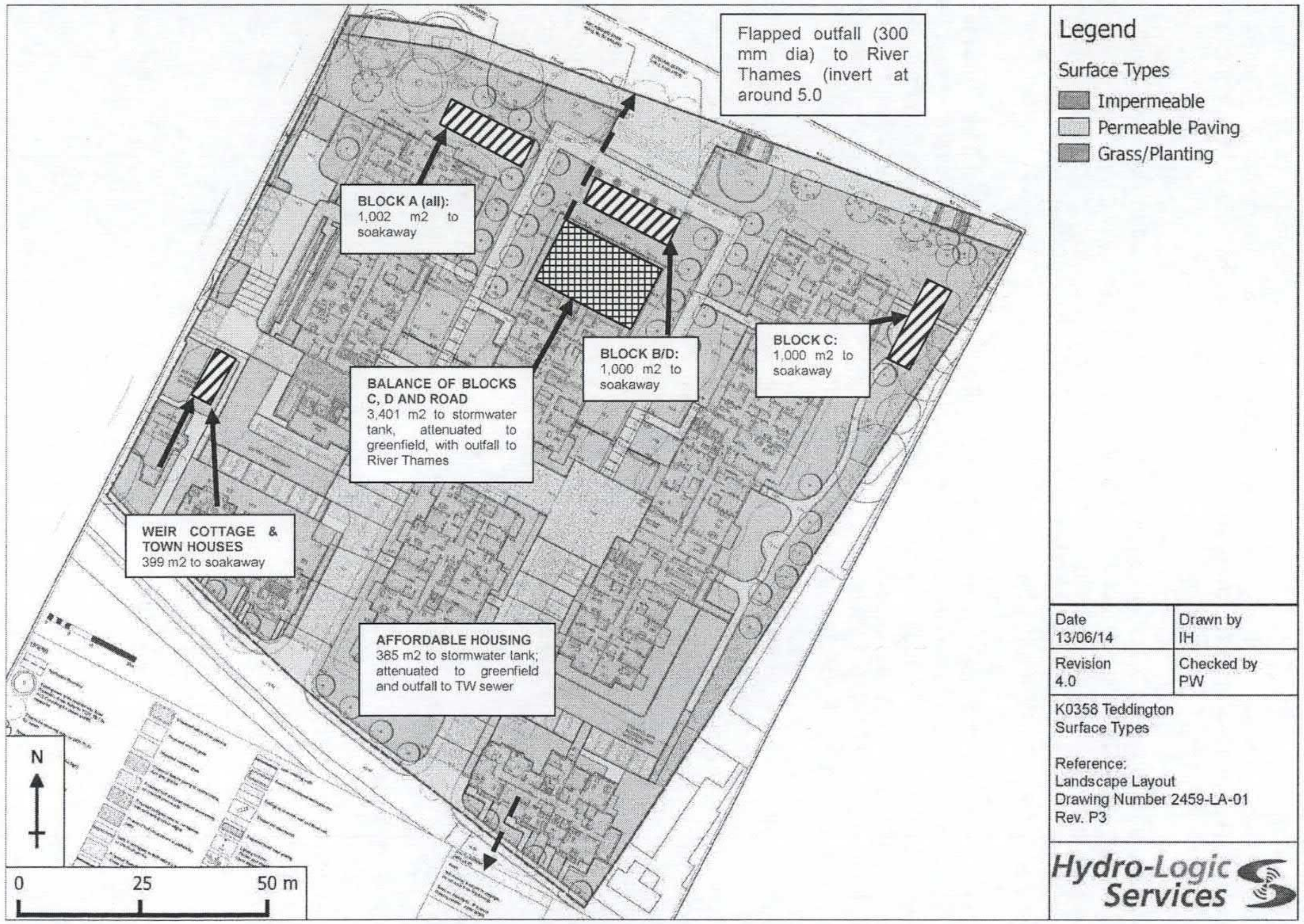
Date 13/06/14	Drawn by IH
Revision 4.0	Checked by PW

K0358 Teddington
Surface Types

Reference:
Landscape Layout
Drawing Number 2459-LA-01
Rev. P3

Hydro-Logic Services

Figure 4-27 Drainage Strategy



(d) Discussion – stormwater tanks

The drainage strategy has included simulation for a main tank beneath Block B with outfall to the River Thames and a secondary tank under the Affordable Housing Block with outfall to the Thames Water sewer. The tanks have been sized such that their combined peak outflow is less than the peak rate of greenfield runoff of 9 l/s. Since these peak outflows from each tank are unlikely to coincide precisely in time (due to different critical storm durations), there is a degree of conservatism in this analysis.

This strategy represents a substantial **reduction in area** draining to the Thames Water sewer. The outflow is also **attenuated** in the proposed stormwater tank adjacent to the Affordable Housing.

There is a residual risk, notably from storms in excess of the design capacity (1 in 100 years plus CC). Should the tanks surcharge, then there is ample storage with the lower garden area to accommodate any surplus. The available storage of over 3,000 m^3 up to the 6.1 mAOD defence level is equivalent to 161 mm of rainfall over the entire site – which is well in excess of the 1 in 100 year 24-hour rainfall.

It is proposed that the main tank outfall to the Thames via a flapped outfall. This introduces the possibility of tidelocking. For the normal tidal regime, this is unlikely to be a problem, as the mean high water springs (Section 3.3.2) is 4.3 mAOD for Richmond and slightly higher for Teddington. With the base of the tank at 5.6 mAOD, there would be a positive head under these conditions. This would also apply for the 5% AEP level from the Product 4 data which is 5.5 mAOD (Node 2.01) which is below the base of the tank.

Under "extreme tidal conditions" with the 1% tidal level in excess of 6.1 mAOD (the crest level of the defences), there is greater probability of tidelocking. However, the duration is likely to be short (a few hours at most) because of these levels would only sustained around high water. Furthermore, any excess could be retained within an enlarged tank – by providing storage up to around 6.9 mAOD. This would reduce any impacts of tidelocking by retaining storage and increase the chances of a positive head.

For "extreme fluvial conditions", high river levels may be sustained over a much longer period. However, under these conditions, the garden areas would be flooded in any event, so the practical consequences of tidelocking of the stormwater drainage system are limited.

In any event, it is recommended that the consequences of tidelocking be investigated more thoroughly during the detailed design phase.

There is an existing stormwater tank at the north western corner of the site and which outfalls to the River Thames via flapped gate. This tank does not form part of the drainage strategy as no information is available regarding its size and other features. Its potential can also be evaluated when the results of SI are available.

(e) Discussion - soakaways

The scope for soakaways is thought to be good on the basis that the underlying geology is likely to feature alluvium and river terrace deposits and which should be characterised by high rates of infiltration. Clearly, any made-ground would need to be excavated to expose and test the medium into which infiltration would take place. This would be an essential component of any site Investigation, following which the trial soakaway designs presented herein could be refined. These investigations will also clarify any requirement for separating the soakaway (eg using geotextile) from any residual man-made ground around it.

It is reported that the groundwater level on the site is around 2 mAOD and therefore similar to the normal level of the Thames downstream of the Teddington Weir, when not subject to high tides or high fluvial flow. The soakaways would require a nominal depth of 0.8 m and could be located in a range of about 4.2 to 5.0 mAOD, thereby providing around 0.6 m cover.

Table 4-13 Summary of Anticipated Geology (Campbell Reith, 2013)

Strata	Depth to Base (m bgl)	Depth to base (m AOD)	Thickness (m)	Typical Description
Made Ground	1 to 2b	4 to 5	1 to 2[b]	A mixture of cohesive and granular man-made soils associated with historic development of the site.
Alluvium[a]	2 to 3b	4 to 3	1 to 2[b]	Soft clay and silt, with bands of loose sand, gravel. Often contains bands of soft organic rich clay and peat.
River Terrace Deposits	5 to 6	1	3	Kempton Park Gravel (Medium dense gravel and sand. Can be clayey in part)
London Clay	65	-60	60	Stiff fissured grey clay, becoming very stiff at depth. Weathers near surface to an orange-brown colour and firm consistency.

a - where present
b - based on historic SI, held in CampbellReith GIS system, and located 300m to the north of the site. Actual values may vary.

On the basis of the soil information, a design value of 0.05 m/hr (1.39 x 10^{-5} m/s) has been adopted for infiltration. Since it has not been possible to conduct infiltration tests at the site, the simulation runs for the main stormwater tank have been extended by assuming that the roof area of 3,000 m^2 proposed to drain to soakaways, be directed into the stormwater tank. The resultant storage volume of 458 m^3 and design depth of 1.93 m are viable in the proposed location under Block B. This gives considerable flexibility to the strategy. Similarly, it may be possible to construct larger soakaways, subject to other site considerations and thereby reduce the size of the main stormwater tank.

The smaller soakaway that receives runoff from Weir Cottage and Town Houses may also be refined following Site Investigation.

Furthermore, we have shown that in the even that soakaways are not viable, the attenuation tank under Building B can be increased in size to attenuate **all of the runoff** that is proposed to drain to the three soakaways. This is regarded as a flexible and robust strategy, that can be refined following the relevant Site Investigations.

(f) Summary

1. This Section has presented a drainage strategy for the proposed development. It incorporates the following components:

- 3 large soakaways and 1 small soakaway for dealing with roof runoff
- Large stormwater tank for attenuating runoff from roofs and the piazza, located at the northern edge of Block B and outfalling to the Thames;
- Small stormwater tank for attenuating runoff from the roof of the Affordable Housing Block

2. Simulation results from the MicroDrainage software show that this combination of devices can attenuate the 1 in 100 year CC storm to the 1 in 100 year **greenfield runoff of 9 l/s**. This is 38% of the pre-development rate of 24 l/s.

3. The strategy affords considerable flexibility. In the event that soakaways are found to be unsuitable, then simulation runs have shown that a single, large attenuation tank can attenuate runoff from the areas shown as draining to soakaways (i.e. 3,000 m^2). It is recommended that the balance between infiltration and attenuation measures be resolved during detailed design when the results of SI are available.

4. The SI will also reveal the extent to which the existing tank, or void in the north-west corner of the site can contribute to the drainage strategy.

5. Storms in excess of the design event may lead to surcharging; however, this can easily be accommodated in the flood storage area of the gardens.

6. The main stormwater attenuation tank may be subject to tidelocking. However, the practical consequences of this are small, due in part to the elevation of the tank and the flood storage provision. Further work may be carried out at the detailed design stage on tidelocking.

4.4 Residual Risks (B8a, B8b)

Residual risks are the risks remaining after applying the sequential approach and taking action to control risk. Residual risks need to be considered as part of all site specific flood risk assessments.

Flood risk to people and property associated with the development can be managed but it can never be completely removed; a residual risk will remain after flood management or mitigation measures have been put in place. Examples of residual flood risk from the PPS25 Practice Guide include:

- the failure of flood management infrastructure such as a breach of a raised flood defence, blockage of a surface water conveyance system, failure of a flap-valve, overtopping of an upstream storage area, or failure of a pumped drainage system; or
- a severe flood event that exceeds a flood management design standard, such as a flood that overtops a raised flood defence, or an intense rainfall event which the piped drainage cannot cope with.

These residual risks are reviewed in this Section.

4.4.1 Failure of flood management infrastructure

The most significant flood management infrastructure affecting the site is the Thames Barrier. Its importance underpins and has driven the TE2100 Management Strategy. Relevant issues pertaining to its operation and risk of failure are beyond the scope of this FRA and are the responsibility of the Environment Agency. The levels that may be consequent on any such failure are considered in Section 3.5.2 alongside general event exceedance.

The existing tidal defences have been described previously. The crest level of the defences is nominally at 6.1 mAOD which is well below the reference flood level of 7.0 mAOD and which has been used to inform the design and layout. Furthermore, the return period of any exceedance of the existing defences is approximately 5% (1 in 20). The consequence of failure of the defences within the site is thus likely to be similar to overtopping of the defences. The occurrence of overtopping events is included within the Emergency Plan (Appendix B). The consequences of breaching as opposed to overtopping are not considered to require special consideration.

The remaining features at risk are not specifically "flood management" infrastructure, but may be considered more appropriately as part of the drainage systems. These include flap valves from the (existing) storm water detention tank and (proposed) flap valve from the flood storage area behind and below the existing defences at 6.1 mAOD, soakaways, rainwater harvesting tank etc. The inspection of these features should form part of the regular and routine inspection of the site.

4.4.2 Event exceedance

As has been noted above, event exceedance, as defined by overtopping of existing tidal defences, is likely on several occasions during the lifetime of the development. It has accordingly been necessary to accommodate such exceedances in the Emergency Plan (Appendix B). Exceedance of the reference flood level (of 7.0 mAOD) has a low probability during the lifetime of the development, though the consequences are clearly serious. The principal residential accommodation in Blocks A, B, C, D plus the Affordable Housing has been designed with a 300 mm freeboard providing protection to a level of 7.3 mAOD. The same standard has been used for the subterranean car parks. The Townhouses (Block E) will be designed to be "flood resistant" to a level of 7.0 mAOD. For floods in excess of this level, water should be allowed to enter the property to avoid the risk of structural damage. Solid floors and flood resilient walls to the round floor are recommended in order to provide a resilient form of construction. All residents have access to areas on site that are "safe" as they have access to higher floors from within all properties.

The level of the emergency access/egress route has been set at 6.8 mAOD, with a maximum depth of 0.2 m. Under extreme flood conditions, the depth may be greater than this. However, the access route is through the raised Piazza in the centre of the site and subsequently along the western boundary. Flow velocities are likely to be low along this entire route and so the hazard classification is likely to remain in the "Low Hazard" or "Danger for Some" categories (Table 4-1 and Table 4-2).

Event exceedance may also result from severe storms on the site that lead to surcharge of the storm drainage system. Given the elevated nature of the proposed development and that it has been designed to cope with a reference flood level of 7.0 mAOD, the consequences of extreme storms are unlikely to have any impact upon the residential property. Surcharged stormwater is likely to flow across the site and may be stored temporarily in the flood storage area, prior to draining under gravity to the Thames through the flap valve. This may cause some inconvenience but is unlikely to pose any significant hazard.

4.4.3 Maintenance

The key requirement in order to minimise residual risk is to ensure that regular inspection and maintenance takes place of drainage systems and infrastructure. This includes the following:

- Main stormwater attenuation tank, including the flap valve.
- Flood storage area beneath Block C, including the flap valve
- Soakaways
- Secondary stormwater tank for Affordable Housing Block
- Permeable paving to be regularly swept to maintain infiltration characteristics.
- Storm drainage system in general
- Five amphibious vehicles
- Flood protection systems, including demountable barriers, flood proof doors, car park barriers, non return valves etc.

4.5 Risks During Construction

The construction activities will involve demolition of existing buildings (excluding the Cottage), construction of new dwellings and associated landscaping. These will involve storage of waste materials, prior to being transferred off-site and storage of building materials and plant. Such storage may impact on flow paths across the site and flood storage, in the event of extreme flooding. The magnitude of these impacts cannot be ascertained at this stage as the construction schedule is not available. However, given the extent of buildings on the site and the likely requirement for reasonably low levels of material storage on site, it is most unlikely that there will be an adverse impact during the construction period. Construction activity may lead to wash off of silt and pollutants to the surface drainage system.

In order to ensure that there are no adverse effects from the storage of materials during the construction phase, it should be confirmed that the flood storage areas and volumetric requirements identified in the FRA are satisfied for all stages of construction. As stated above, given the current extent of buildings on site, satisfying this condition should not be onerous.

The potential for impacts to occur as a result of storage of materials will be minimised by the following measures:

- Storage compounds (for the storage of construction materials or temporary stockpiling of material from demolished buildings) will be located away from the Thames and drains;
- Drums and barrel will be stored in a designated bunded safe area within a site compound; and
- All drums and barrels will be fitted with flow control taps and will be properly labelled.

The Construction Site Manager should also be in receipt of flood warnings for the Thames from the Environment Agency. This will allow removal of plant from the site or its relocation to areas outside those liable to flooding. Whilst flooding will be unhelpful to the construction process, the consequences may be managed by preparation and controlled dewatering following the flood.

The proposed development will also involve realignment of the existing defences. All such work would be undertaken in conjunction with the Environment Agency to ensure necessary approvals for design and constructional sequence. In particular, it will be necessary to ensure the integrity of the existing tidal defences throughout the period of construction. This will be achieved by maintaining the existing defences until any replacements are in place. Should there be any requirement for tying in new defences to existing alignments, this will be undertaken at times when there is essentially no risk of fluvial or tidal flooding. Engagement with Environment Agency staff will also be required to ensure that the new defences are compatible with the Environment Agency plans for possible raising by up to 0.8 m at some stage in the future.

4.6 Climate Change (C4a)

The general impacts of climate change on flood behaviour in England and Wales remain unclear. The FEH (Institute of Hydrology, 1999) describes a review of flood peak data to investigate possible trends. The analyses do not show that climate change has affected UK flood behaviour, but neither do they prove that it has not affected it. NPPF requires a consideration of the impacts of climate change on the flood risk for any proposed development. The suggested mechanism for this is to allow for increases of 10% in peak flows by 2025 and increases of 20% from 2025 to 2115. For precipitation, NPPF recommends a progressive increase reaching 30% by 2115. Climate change has been accounted for in accordance with these requirements by the Environment Agency by:

- Increasing river flows by 20%;
- Increasing rainfall depths by 30%;
- Use of appropriate tidal projections to inform model boundary conditions for combined model runs.

The climate change allowance that need to be used in FRAs are shown in Appendix C .

5. Summary and Recommendations

This Report presents an FRA for the proposed Teddington Riverside development on the site of the existing Teddington Studios. It has been informed by exchanges with the LBRT, The Environment Agency and Thames Water, with officials from each organisation providing valuable input, relevant data and feedback. The main findings are as follows, with cross referencing to the appropriate Section of the FRA shown in square brackets. By way of a summary, the LBRT requirements are presented in Table 5-1 with further cross-referencing to the FRA.

1. The proposed development is for a residential scheme which mapping provided by LBRT has confirmed is in **flood zone 3a** [Section 3.5]. Residential use has a vulnerability classification of "More Vulnerable". Accordingly, it is only acceptable in flood zone 3a if both the Sequential Test and the Exception Test have been satisfied.

2. **The Sequential Test** has been undertaken by CgMs Consulting, the findings of which have been discussed with staff from LBRT. The Sequential Test has shown that there are no other equivalent sites available for development. Subject to review by the Environment Agency, the Sequential Test is deemed to have been satisfied. [Section 2.3]

3. **The Exception Test** involves two components based on the sustainability credentials of the development and an acceptable FRA. Subject to this FRA being acceptable, the Exception Test is deemed to have been passed. [Section 2.3].

4. **Flood levels** at the site result from a complex interaction of fluvial and tidal factors and are subject to the operation of the Thames Barrier. The Environment Agency has provided the results of detailed hydraulic modelling which has provided the basis for adopting the reference flood level for the site. Whilst moderate floods have a large tidal component, the truly extreme floods are dominated by fluvial factors. The flood level for the site is 6.97 mAOD (nominally 7.0 mAOD) which corresponds to the 1% (1 in 100) fluvial extreme. [Section 3.3 and 3.5]. The Environment Agency has provided revised flood levels arising from the TE2100 modelling. These extreme levels are higher than the reference flood level for the early years of the development. These levels have not been used in this FRA for reasons that are outlined in Section 3.5.2.

5. **Other sources of flooding** have been reviewed in the FRA. The Environment Agency has indicated that the risk of groundwater flooding is unlikely. The elevated position of the site relative to the surrounding area means that it is not at risk from surface water flooding, nor sewer flooding. It is clear that the main risks are from some combination of fluvial and tidal flooding.[Section 3.3]

6. **Finished floor levels** for the main residential blocks plus the Affordable Housing have been set at 7.3 mAOD, which includes a 300 mm contingency, as recommended in the LBRT SFRA. Floor levels for the Townhouses are set at 6.2 mAOD but flood resistance and resilience measures will be provided to a level of 7.1 mAOD. This is 300 mm above the flood levels provided for the flood plain at this location, namely 6.8 mAOD. Flood resistance and resilience measures are also recommended for Weir Cottage to at least this level. [Section 4.2.1].

7. **The Basement** is not for habitation, but is solely for car parking. The entry to and exit from the car park will be equipped with a removable flood barrier providing

protection to a flood level of 7.3 mAOD. Furthermore, the car park will be provided with drainage to deal with storm water or flood water that may enter the car park. There will be internal access via lifts and steps to the residential blocks. The basement car park will have sufficient space for all cars that use the surface car park; this to be achieved by valet parking. There should be no cars in the surface car parks during major flood events. [Section 4.2.3]

8. **Flow routes** through the site have been maintained at existing levels by the provision of a culvert that passes under the Piazza. This replicates the existing flow path in the vicinity of the Gatehouse. In practice, this flow route is likely to be active very infrequently, currently for levels higher than the 1% level. At such a level, the water level changes only slowly over time, so the hydraulics of the culvert are not considered to be limiting factors. Additional flow will be able to cross the site through a 1 m wide gap at a level of 6.3 mAOD on the eastern boundary of the site. [Section 4.3.1]

9. **Flood Storage** calculations have shown that the proposed layout satisfies the flood storage requirements on a level-for-level and volumetric basis above the existing tidal defences of 6.1 mAOD – for both the "riverside" and "development side" of the tidal defences. For levels behind and below the defences, the volumetric storage requirements are satisfied, including a provision for soil embankments adjacent to the main Buildings for landscaping purposes. The proposal makes use of flood storage beneath Block C, which is able to drain via a flapped outlet to the Thames. There is a further flood storage provision under Block A, specifically to "fine tune" the level for level storage compensation. As a consequence, there is in fact a small gain in flood plain storage of around 250 m^3 at 6.1 mAOD and 800 m^3 at 7.0 mAOD; after allowance for some contingency for soil embankments. [Section 4.3.2]

10. **Runoff rates** are substantially reduced from the existing levels as a result of the development, thereby reducing flood risk for surrounding areas. This is due to the replacement of the largely impermeable site with a mixture of roofs, permeable paving, grass and borders. The drainage strategy has been developed using the simulation mode, MicroDrainage. This has demonstrated that a combination of soakaways and stormwater attenuation tanks can be used to attenuate peak runoff for the 1%CC storm to the greenfield rate, which is 38% of the pre-development rate. This satisfies the LBRT condition of reductions of at least 50% from current levels and the London Plan to achieve greenfield rates of runoff. [Section 4.3.3c]

11. The drainage strategy has demonstrated the storage volumes required to achieve **Surface Water attenuation** in both soakaways and attenuation tanks. There is considerable flood storage available on site which can provide contingency in the event of failure of any drainage component or storms in excess of the design storm [Section . 4.3.3e].

12. Following removal of surface cover, but prior to any construction, **Site Investigations** should seek to establish the dimensions and the condition of the existing stormwater tank in the north-west corner of the site, with a view to it being used as part of the stormwater management infrastructure. The Site Investigation should seek to establish the nature and infiltration characteristics of the soil that underlies the impermeable surfaces and buildings. This will help to refine the surface water management strategy. Given that it was not possible to undertake field infiltration tests, the drainage strategy has demonstrated that the main soakaways can be replaced with an enlarged attenuation tank and still achieve the target reductions in runoff. It is thus clear that the drainage strategy is very flexible.

13. **Safe access** and egress is provided via Broom Road for moderate floods initially on foot and for deeper floods using suitable vehicles. For extreme floods, safe access and egress within the site is to be provided via raised walkways at an elevation of 6.8 mAOD. These provide safe internal access to the Piazza from where amphibious vehicles can be boarded. These will provide a "shuttle" service to safe areas wholly outside the flood zone. [Section 4.2.2 and Appendix B]

14. An **Emergency Plan** has been prepared in line with the LBRT requirements. This is included as Appendix B and describes the procedures for warning and evacuation of the site at times of imminent flooding. It is recommended that an annual drill be undertaken, ideally in association with the Environment Agency and LBRT.

15. The Emergency Plan provides benefits to the wider community including the Provision of emergency car parking; use of the proposed emergency access, use of the site as a refuge and provision of an access/egress link for the Lensbury Hotel [4.2.5]

16. The **Residual Risks** have been assessed and are considered to be minor. [Section 4.4] A maintenance programme of key drainage infrastructure should be put in place to ensure that residual risks are minimised. [Section 4.4.3]

17. There is a **16 m standoff** from the River Thames as required by the Environment Agency. This is achieved in part by realignment of the existing defences [Section 4.2.4]. Provision has been made in the design layout to satisfy the possible future need for **raising the existing defences** to a level of 6.9 mAOD, as indicated by the Environment Agency. This has ensured that there is relatively easy access to the bank and there are no pinch points that may restrict the ability of plant to access the river.

18. Flood risks during the **period of construction** have been assessed and, with the adoption of standard site management practice, they should be of no practical consequence. [Section 4.5]

19. A **statement of flood risk** should be provided to all residents that they can provide to their Insurance Company (or other organisations).

20. In summary, the proposed development will lead to an increase in flood storage on the site and a reduction in peak rates of runoff. The provision of elevated living accommodation with a range of access/egress routes will provide benefits to the local residents under flood conditions, as well as a refuge for future residents of the site.

Table 5-1 LBRT: Planning & Development Control Recommendations

Spatial planning recommendations		***Response in FRA and reference***
Important Considerations	Future development within Zone 3a High Probability can only be considered following application of the Sequential Test	*Section 2.3. Given that no equivalent and available sites were identified at lower risk of flooding, the Test is deemed to have been passed.*
Land Use (refer PPS25 Table D2)	Land use should be restricted to Water Compatible or Less Vulnerable development. More Vulnerable development may only be considered if Exception Test can be passed.	*Section 2.3. Subject to acceptance of the FRA, the Test is deemed to have been passed.*
Development control recommendations		
Detailed Flood Risk Assessment (FRA)	Required	*This Report.*
Floor level	Floor levels are to be situated a minimum of 300mm above the Q100 fluvial or Q200 tidal (whichever is greater) flood level, including climate change, assuming a breach of the river defences.	*Section 4.2.1. Most floor levels set at 7.3 mAOD, 300 mm above 1%CC fluvial extreme. Flood resistance and resilience provided where floor levels are lower (Town Houses and Weir Cottage).*
Site access & egress	Refer SFRA Appendix E. For residential property, dry access is to be provided above the Q100 fluvial or Q200 tidal (whichever is greater) flood level, including climate change, assuming a breach of the defences.	*Safe access routes presented in Section 4.2.2 and Appendix B . Agreed level of 6.8 mAOD in consultation with EA & LBRT.*
Basements	Self-contained residential basements and bedrooms at basement level should not be permitted. All basements, basement extensions and basement conversions should have internal access to higher floors.	*Basement car park is protected by "flip-up" flood barriers. The car park has internal pedestrian access to higher floors (Section 4.2.3).*
Site runoff	Implement SuDS to ensure that runoff from the site (post redevelopment), as a minimum, is not increased. A reduction in site runoff should be sought, aiming to achieve greenfield run-off rates, or reduce run-off rates by at least 50% over current levels.	*Reduction to greenfield (38%) of pre-development rate based on reduction of impermeable area, provision of permeable pavements, soakaways and attenuation tanks.*
Buffer Zone	A minimum buffer zone must be provided to 'top of bank' within sites immediately adjoining the River Thames. A 16m buffer will be sought along the River Thames. Advice must be sought from the Environment Agency at an early stage.	*16m buffer zone provided along with provision for raising defences.*
Other	Ensure that the proposed development does not result in an increase in the risk of flooding (from all sources) within adjoining properties. This may be achieved by ensuring (for example) that the existing building footprint is not increased, that overland flow routes are not truncated by buildings and/or infrastructure, or hydraulically linked compensatory flood storage is provided within the site (or upstream)	*Reduction in built footprint means that the proposed redevelopment results in increase in flood plain storage and reduction in runoff rates (Section 4.3.3). Demonstrated in Figure 4-24 that flood storage is hydraulically linked.*

Extract for "Zone 3a, Defended, Richmond"

6. References

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