RD/MC/160

Evidence regarding land south of the Cambridge Biomedical Campus

Part 3 - Flood Modelling and Drainage Strategy Report

Extension to Bio-Medical Campus, Cambridge

Flood Modelling and Drainage Strategy Report

On behalf of **Carter Jonas**

Project Ref: 36873/2001 | Rev: D | Date: October 2016

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Report Title: **Flood Modelling and Drainage Strategy Report**

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For and on behalf of Peter Brett Associates LLP				

Revision	Date	Description	Prepared	Reviewed	Approved
A	14/09/16	First Draft	CW/SF	AS	SD
B	13/10/16	Amended further to planning consultation.	SF	SD	SD
C	21/10/16	Hydrology and results update	CW	SD	SD
D	25/10/16	Appendices update	CW	SD	SD

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1 Background

1.1 Commission

1.1.1 Peter Brett & Associates LLP ("PBA") were commissioned by Cambridgeshire County Council via Carter Jonas to undertake work to provide an evidence base in support of site promotion and demonstrating deliverability for a proposed extension to the Bio-Medical Campus at Addenbrookes, Cambridge, promoted as a modification to the South Cambridgeshire Local Plan (provisional proposed modification PM/SC/8/A).

1.1.2 This report was prepared against an agreed brief and in accordance with PBA's Fee Proposal and Terms and Conditions. This report is not intended for and should not be relied on by any third party (i.e. parties other than the Client). PBA accepts no duty or responsibility (including in negligence) to any party other than the Client and disclaims all liability of any nature whatsoever to any such party in respect of this report.

1.2 Site of Interest

1.2.1 The site of interest is shown in Figure 1. It is proposed to be an extension to the Bio-Medical Campus at Addenbrookes, Cambridge. Masterplanning is at an early stage, however it is anticipated that the development will comprise in the order of 65% laboratory and 35% office development, of density 15,000sqft per acre, with associated car parking and infrastructure.

1.2.2 The published Environment Agency Flood Zone Map (Figure 1) shows that part of the site is within Flood Zone 3. The Environment Agency's flood zones were generated using simplistic JFLOW modelling and are known to have low accuracy in this area. The flood pathway shown does not relate well to the topography of the local area and does not follow the known drainage pathways. There is therefore low confidence in the Flood Zone Map for this site.

1.2.3 Surface water flooding has been investigated in the Cambridge and Milton Surface Water Management Plan (2011). The site lies just within the boundaries of the study area. Modelling of the 200 year (0.5% AEP) event indicates a surface water flow path through the site, with flooding depths of up to 0.5m on the site (Figure 2), affecting up to 50% of the site area. The same patterns of flooding are shown on the Environment Agency's surface water flood risk map. The location of the site close to the boundary of the SWMP study area gives some uncertainty in the results, as surface water flowing overland from the hills to the south is not taken account of. Taking these into account could further increase the risk of surface water flooding.

1.3 Scope of Study

1.3.1 The purpose of this study is to quantify surface water flood risk and surface water drainage for the purposes of establishing existing conditions at the site, informing site masterplanning and demonstrating in outline the viability of any required mitigation. This study does not constitute a Flood Risk Assessment but provides supporting information for the future evidence base in support of an application.

1.3.2 A previous scoping exercise (Flood Risk Constraints and Opportunities Report, PBA, January 2016) indicated that fluvial flood risk is less of a concern than suggested by published mapping. However, further work to appraise surface water risks is required.

1.3.3 The scope of work for this study is therefore as follows:

- Construct a 2D hydraulic model of the catchment terrain, estimating rainfall, surface water runoff and flows for the 100 year and 1000 year flood events, and mapping surface water flood areas for these events. This will quantify the surface water overland flow that is implied by the published surface water risk mapping.

- Undertake sensitivity testing of the impacts of climate change in accordance with the EA's new guidance document issued in February 2016.

- Estimate the increase in post-development surface water run-off, making general assumptions regarding impermeable surface coverings.

- Identifying, in concept, a combined scheme to address any required safe routing of surface water flooding through the site, and attenuation of run-off from the site. Amending the hydraulic model to demonstrate the viability of the scheme.

- Liaison to allow the masterplan to be developed with adequate space for water, and the integration of the above measures into the landscape proposals.

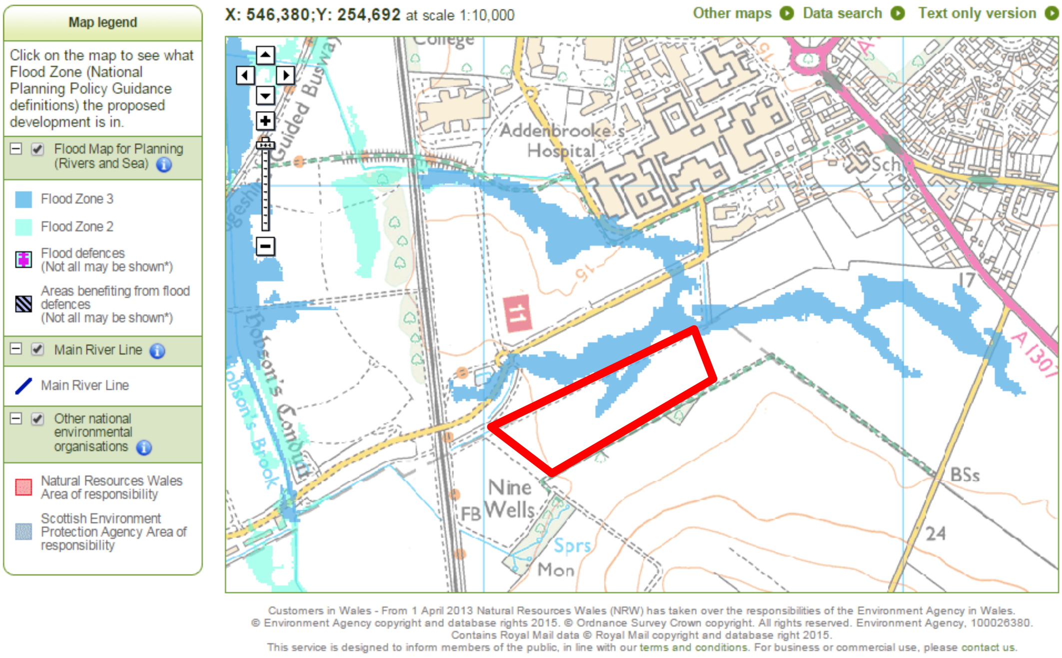

Figure 1: Environment Agency Flood Map for Planning.

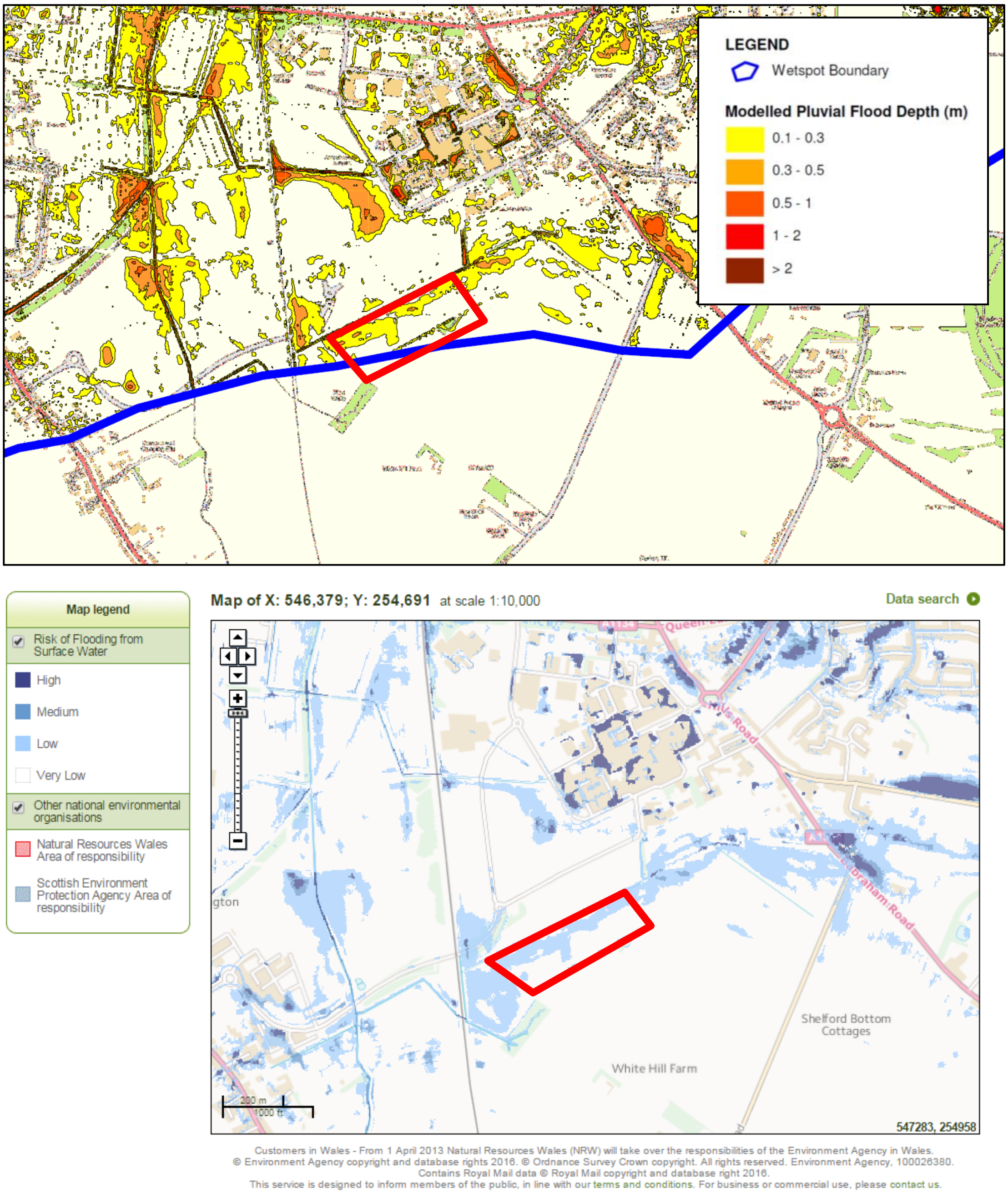

Figure 2: Extract from Cambridge SWMP, and Environment Agency surface water flood map.

2 Site Characteristics

2.1 Existing Drainage

2.1.1 The site currently drains northwards and westwards into a field ditch. This flows beneath the railway line, joining spring flows from Nine Wells Spring, into Hobson's Brook. This flows northwards through Cambridge via Vicar's Brook and Hobson's Conduit, into the River Cam (Figure 3). Nine Wells is a Local Nature Reserve. Hobson's Conduit and Vicar's Brook have City Wildlife Site Status, and Hobson's Conduit is a Scheduled Ancient Monument. They are managed by Cambridge City Council and Hobson's Conduit Trust, who hold title on the bed of the watercourse.

2.1.2 Hobson's Conduit and Vicar's Brook discharge surface water into the River Cam, including from new developments at the Cambridge BioMedical Campus, Addenbrookes, Glebe Farm and Clay Farm. The brooks are regularly monitored with an ongoing programme of flow and water quality monitoring to assess the impact of the residential and Bio-Medical developments. To the south-west of the site, the brook fed by Nine Wells is highly regarded as a chalk stream and is of significant heritage value. The brook feeds into the conduit which historically provides water to urban features in the city, the Botanical Gardens and college gardens, and provides a valuable wildlife corridor to the city.

2.2 Soils and Geology

2.2.1 The site is underlain by Chalk bedrock. British Geological Survey maps indicate that the site overlies the boundary between three Chalk types: the upper Zig Zag Formation, the middle Totternhoe Stone Member and the lower West Melbury Marly Chalk Formation (Figure 4). There are no superficial deposits. As the underlying Marly Chalk Formation is less permeable, rainwater percolates through the upper Chalk deposits before flowing through the Totternhoe Stone and emerging as a spring. The site overlies this spring line.

2.2.2 Boreholes are available via the British Geological Survey website. Two available boreholes immediately to the north of the site, in the Marly Chalk formation, show groundwater levels between 6 and 9 m below ground level, adjusted to 1.7 – 1.9 m below ground level for piezometric head. Given the complex site geology, on-site boreholes would likely be required at the planning application stage.

2.2.3 Groundwater monitoring of Nine Wells spring was undertaken for the Addenbrooke's Access Road project (referenced in the Clay Farm Development Groundwater Assessment Report, AECOM Environmental, May 2010). The initial findings showed that:

- The springs at Nine Wells are fed by groundwater draining from the north-east, through the Totternhoe Stone from the overlying chalk deposits.
- The groundwater levels respond to seasonal changes in rainfall and evaporation, with some limited response to heavier rainfall events superimposed upon the baseline (dampened by the presence of the surface discharge zone).
- Groundwater contained at depth may be confined and under pressure.

2.2.4 The Cranfield University Soilscapes Map describes soils at the site as "shallow lime-rick soils over chalk or limestone", freely draining and vulnerable to erosion.

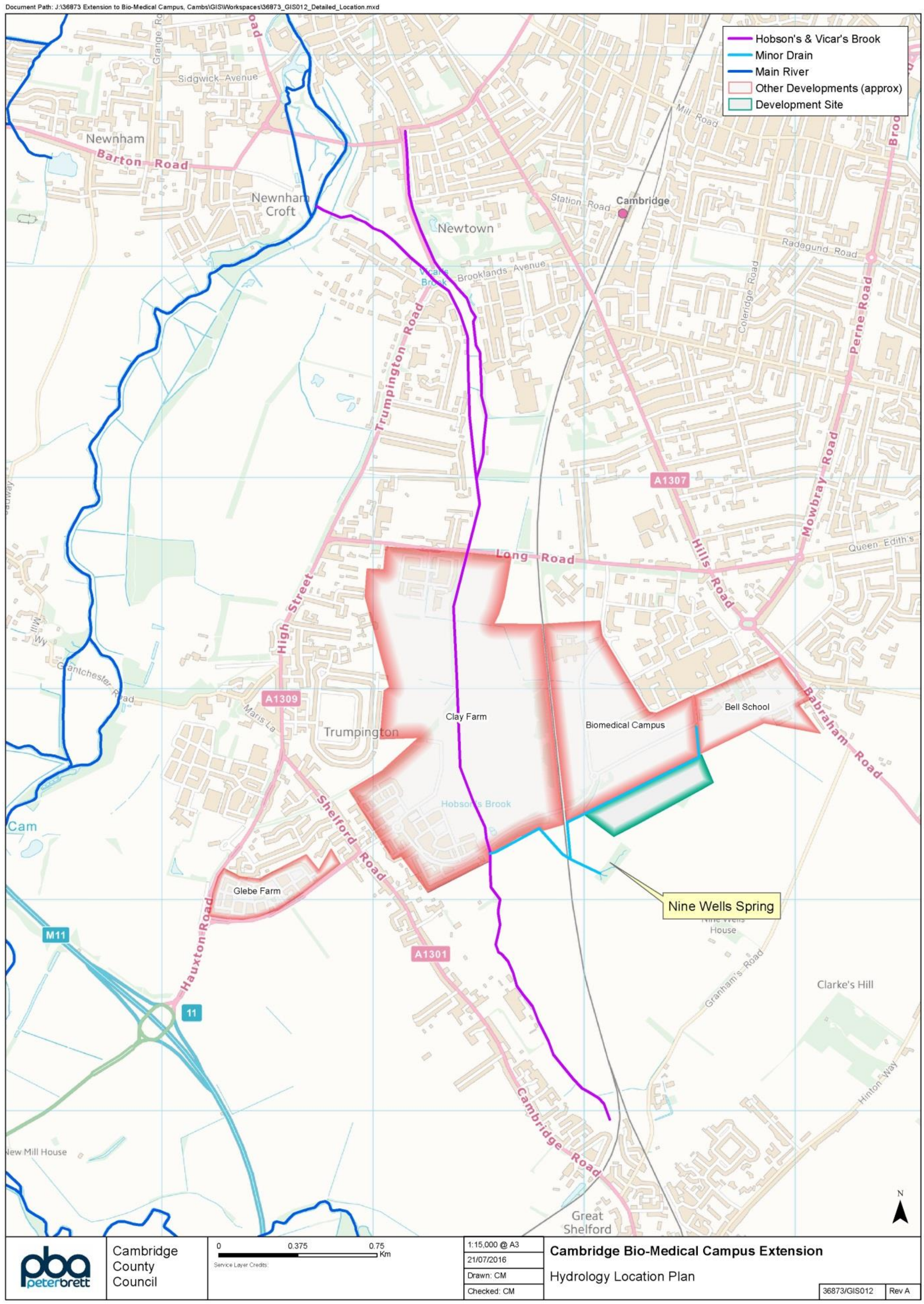

Figure 3: Detailed location plan.

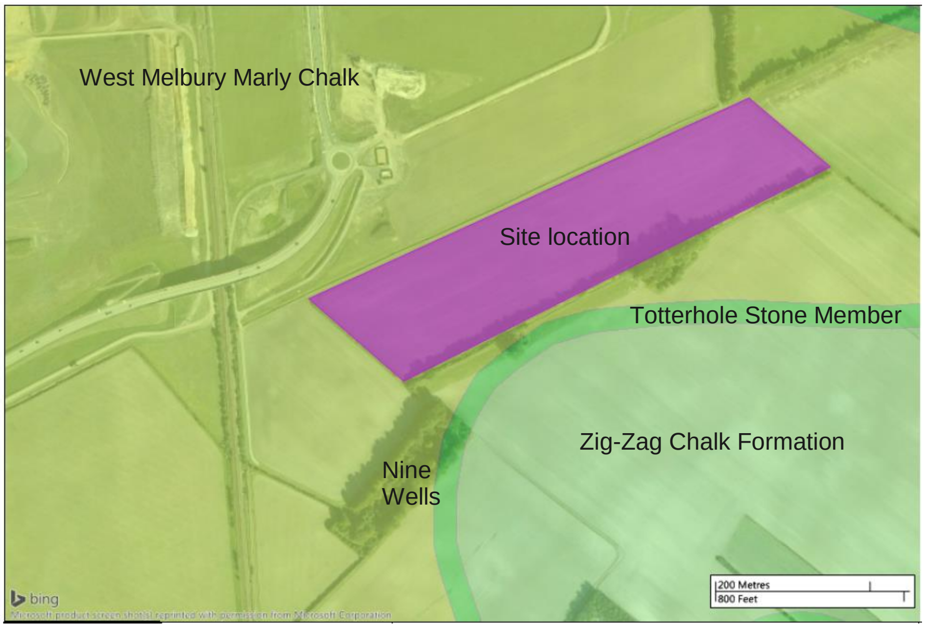

Figure 4: Geology plan.

2.3 Flood Zone Classification

2.3.1 As discussed earlier, the published Environment Agency Flood Zone Map (Figure 1) shows that part of the site is within Flood Zone 3. The Environment Agency's flood zones were generated using simplistic JFLOW modelling and are known to have low accuracy in this area. The flood pathway shown does not relate well to the topography of the local area and does not follow the known drainage pathways. There is therefore low confidence in the Flood Zone Map for this site.

2.3.2 The Flood Risk Assessment for the Cambridge Bio-Medical Campus to the north of the site investigated and discounted these Flood Zones, showing through numerical modelling that the Bio-Medical Campus site was not within Flood Zones 2 or 3. The model was audited and accepted as part of the planning process.

2.3.3 That model did not extend to the current site under consideration. However we anticipate that extension and investigation of fluvial flood risk to this site would provide a similar evidence base to demonstrate that the Flood Zone 2 and 3 extents are smaller or not present here. The primary risk is surface water run-off resulting in overland flow and this has been appraised in detail through additional modelling; this modelling provides assessment of the fluvial flood risk from the watercourses and demonstrates a re-classification as Flood Zone 1.

3 Surface Water Flood Modelling

3.1 Model Development and Scenario Testing

3.1.1 A linked 1D-2D model of the catchment and drain adjacent to the site was constructed using FloodModellerPro (FMP) software. Rainfall was estimated using the FEH13 data set. Rainfall losses were set at 82.5% following sensitivity testing against a ReFH2 flow analysis. Full details of model development and flow estimation are included in Appendix B and Appendix C The following scenarios were tested:

- Baseline “as existing” scenario:
  - 30 year, 100 year, 100 year plus climate change and 1000 year flood events.
  - Sensitivity to rainfall loss parameter.
- Post-development scenario:
  - 30 year, 100 year, 100 year plus climate change and 1000 year flood events.
  - 100 year event with 50% rainfall loss.

3.2 Baseline Scenario

3.2.1 Figure 5 shows the estimated flood depths across the catchment for the 100 year event. Flood waters drain in an easterly direction from the Gog Magog hills, with localised pockets of deep water becoming trapped adjacent to roads. This pattern matches the Environment Agency’s surface water flood maps well.

3.2.2 Flows within the development site enter from two directions:

a. From the north-east, flowing in a south-westerly direction across the site. This is the main flow path.

b. From the hills to the south, flowing in a northerly direction. These flows are trapped by a ridge of high ground / hedgerows immediately to the south of the site, with localised low spots allowing a few flow paths to develop across the site.

3.2.3 Figure 6 shows the estimated flood depths at the development site for the 30, 100, 100 + climate change and 1000 year events. The site is at risk of surface water flooding in all of these events.

3.2.4 Water depths in the ditch remain below the site bank level for the 100 year and 1000 year events. The ditch does not therefore overtop and cause fluvial flooding. This indicates that the site should be classified as Flood Zone 1 for flood zoning purposes and development allocation. However, the development will need to be protected against flooding from surface water overland flow across the site.

Sensitivity to Rainfall Loss Parameter

3.2.5 Figure 7 shows how the estimated flood depths at the development site vary according to rainfall percentage loss. If a lower rainfall loss is applied, flood depths across the site increase significantly. This is reflected in the peak flows in the ditch, which are of a magnitude higher for when the standard EA (39%) and Cambridge SWMP (50%) losses are applied, compared to this study’s estimate of 82.5%.

3.2.6 The significant flood depths for lower rainfall losses are due to a lack of watercourses in the catchment to catch and convey surface water runoff. This is a reflection of the highly permeable nature of the catchment, and is further physical evidence of the very low surface water runoff rates within the catchment.

3.2.7 The choice of rainfall loss parameter remains an area of high uncertainty in this study. While the 82.5% rainfall loss recommended here is supported by FEH flow estimate calculations, there is no gauged data for comparison. It is therefore recommended that a lower rainfall loss of 50% is used to sensitivity test mitigation measures as a precautionary measure.

3.3 Post-Development Scenario

3.3.1 For the post-development scenario, the model was amended to include a ditch around the perimeter of the site to intercept overland flows from surrounding areas and route them into the existing drain at the western corner of the site. Post-development run-off from the site was included at attenuated rates. See Appendix B Appendix B for further details.

3.3.2 Initial model tests indicated that the perimeter drains resulted in a small increase in peak flows downstream due to more efficient capture and conveyance of flood flows (approx. 0.4 m^3/s). Therefore flapped orifice units with 0.12 m^2 bore area were used to simulate a limited outflow from the perimeter drains to ensure no increase in peak flows downstream (Figures 8 and 9).

3.3.3 The size of the perimeter ditch was iteratively amended to provide sufficient storage for the 100 year plus climate change event, with a freeboard. This could be provided as a series of stepped weirs along the length of a widened ditch. The masterplan has made allowance for this feature, though the detailed design will look to combine this with the wetland feature for on-site surface water attenuation for the development run-off to provide a more integrated approach with the landscape objectives. The interceptor ditch will also strengthen both the landscape buffer and amenity function, deterring encroachment of users towards the nature reserve. Other off-site measures will also be considered at detailed design stage to better manage and control surface water overland flow within the landowners ownership, and further reduce the impact downstream.

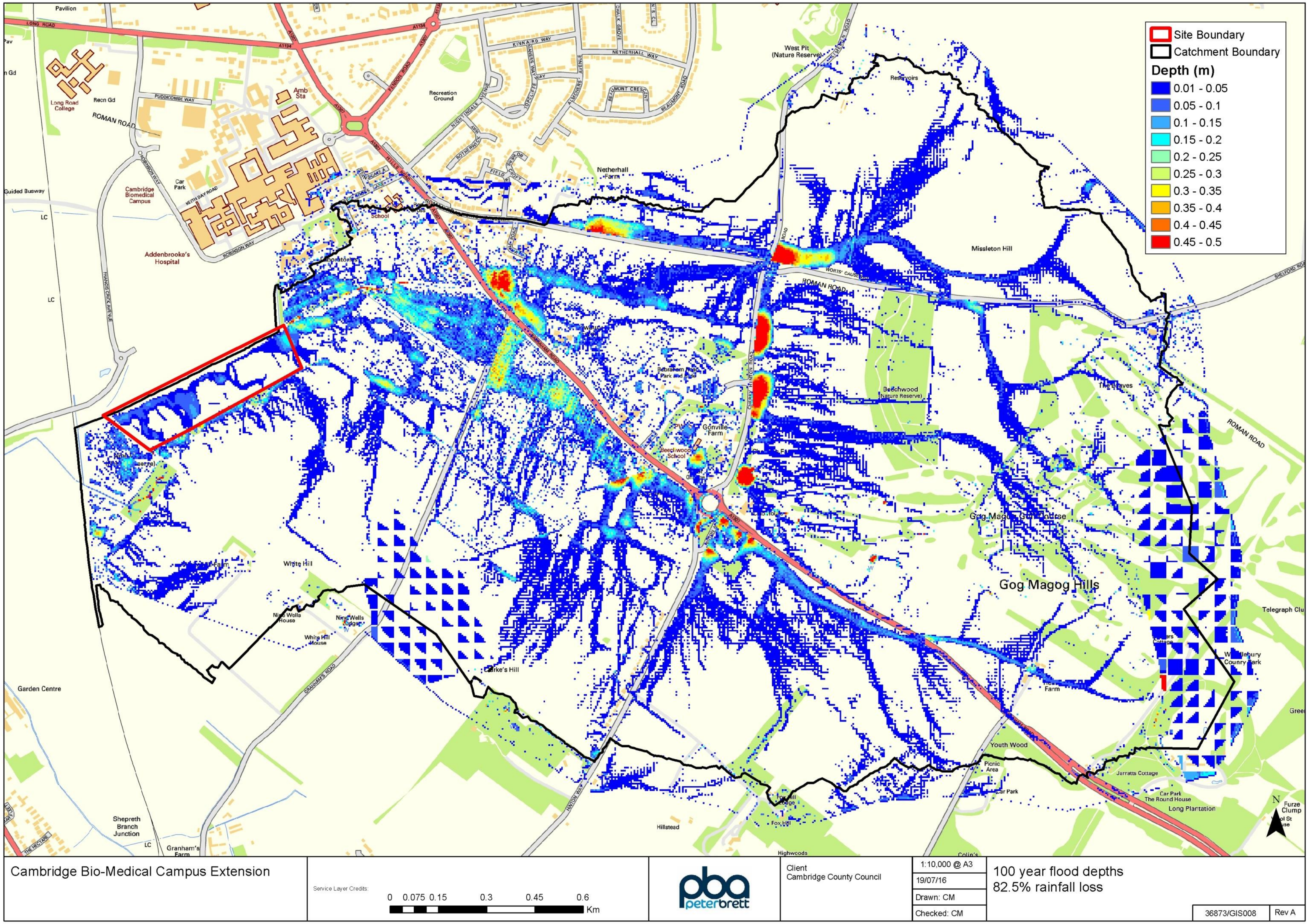

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Figure 5: 100 year flood depths, Baseline scenario, 82.5% rainfall loss.

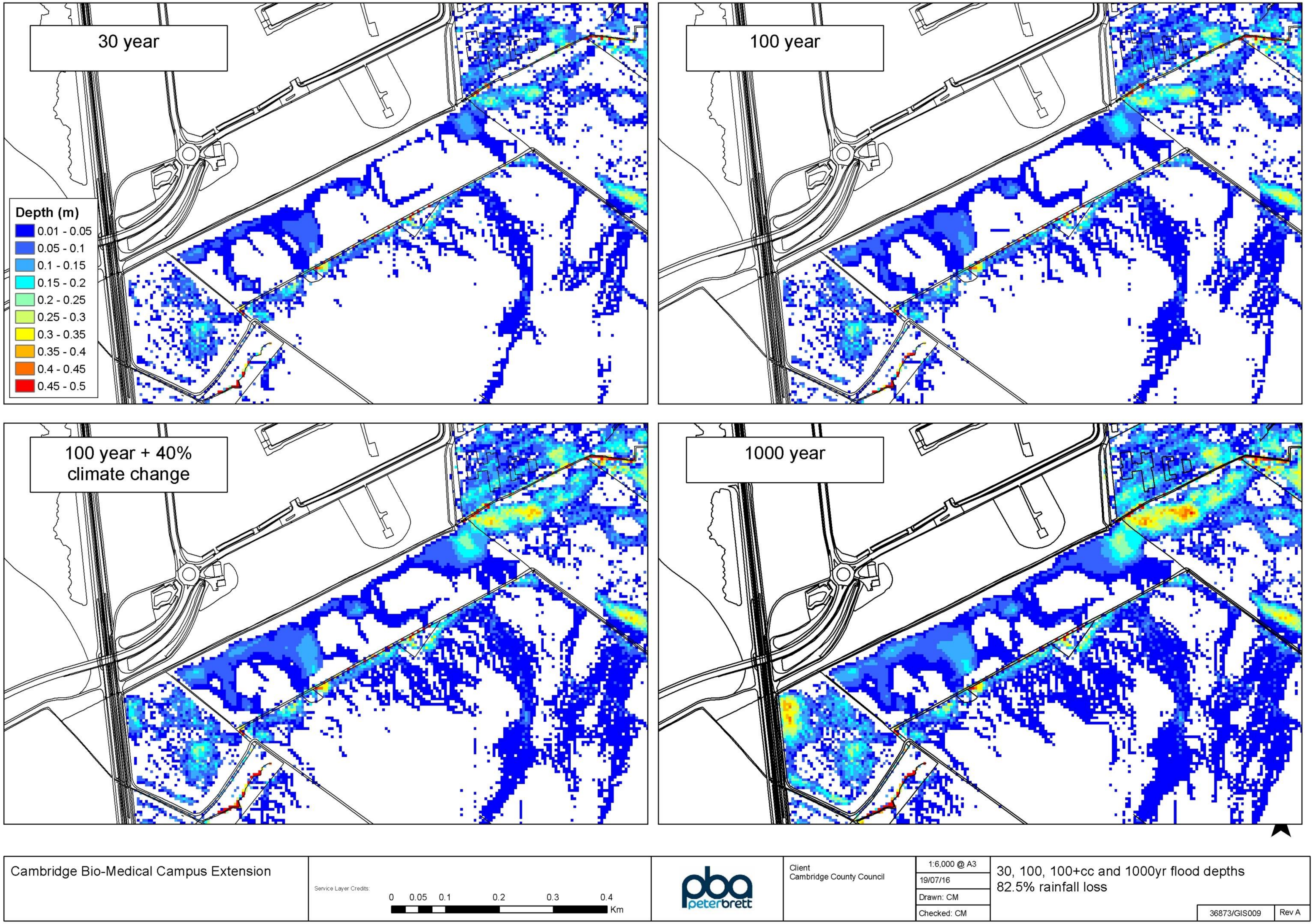

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Figure 6: Baseline scenario: 30 to 1000 year flood depths, 82.5% rainfall loss.

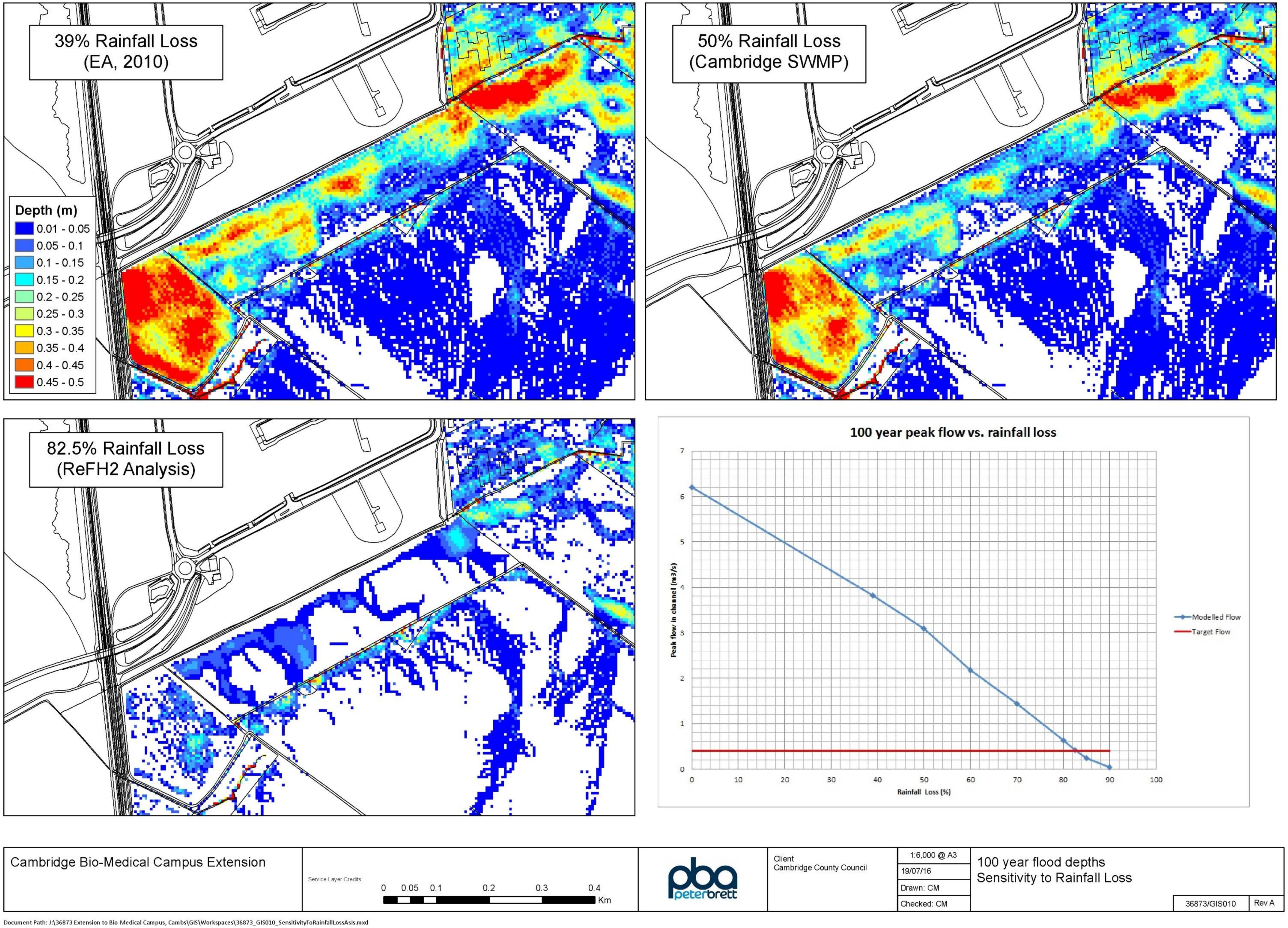


Figure 7: Sensitivity of baseline flood depths to rainfall loss parameter.

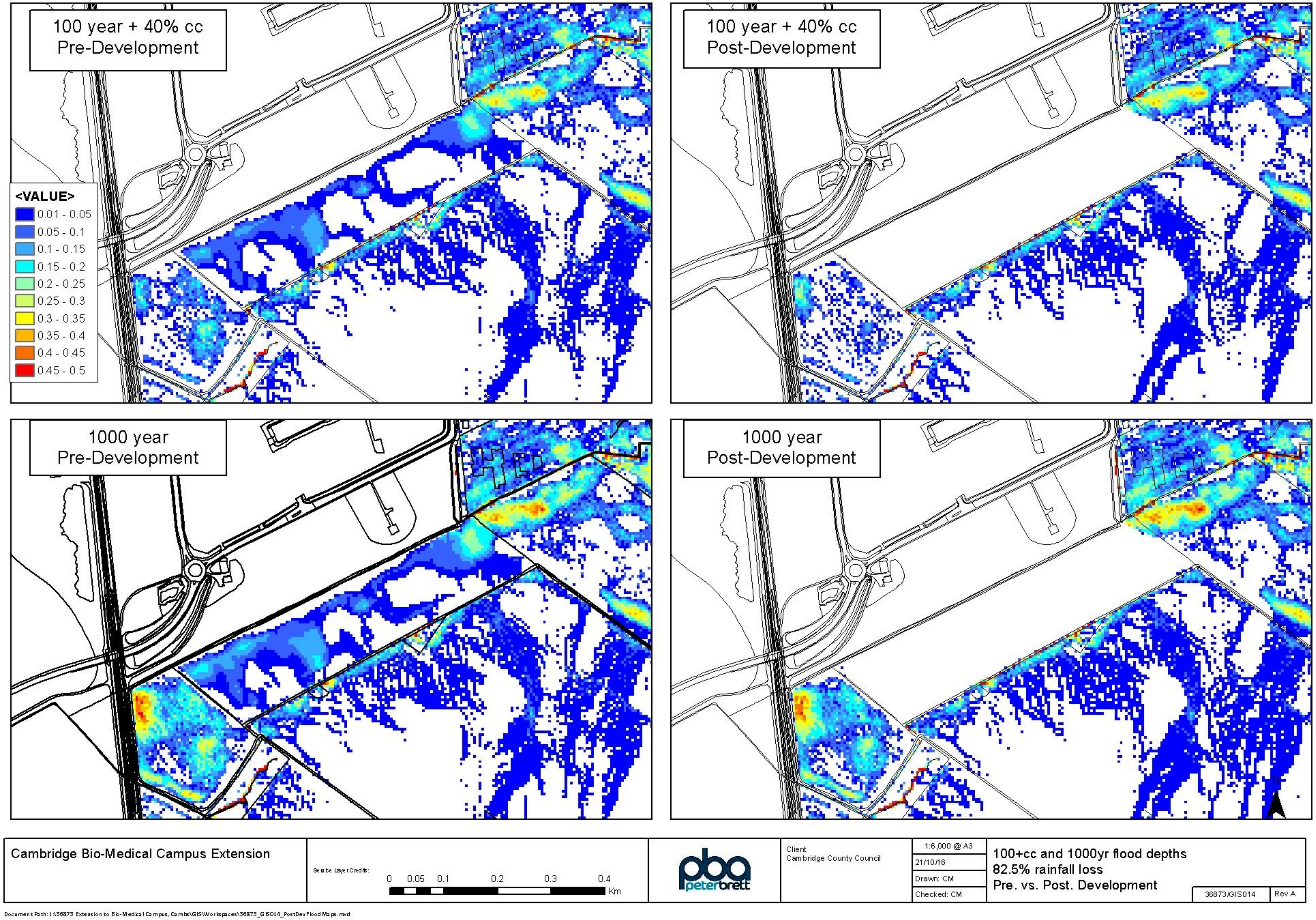


Figure 8: Pre- and Post- Development Flood Extents (100 year plus climate change and 1000 year events). Differences in the 0.01 – 0.05 m depth band are within model tolerances and uncertainties.

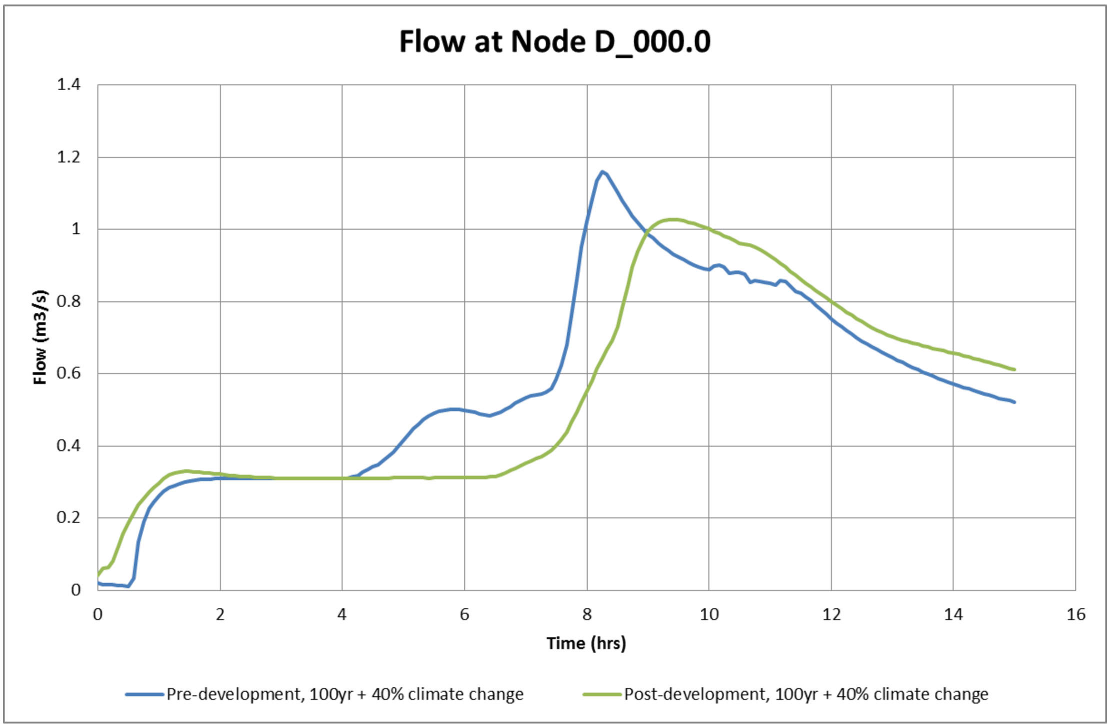


Figure 9: Pre- and post- development hydrographs for node D_000.0 (ditch at the corner of the railway line), for the 100 year plus 40% climate change event.

4 Surface Water Drainage

4.1 Ground Conditions

4.1.1 Initial geotechnical desk study assessment (Appendix D) indicates that the Nine Wells spring line is located to the south of the site where the 'Totterhole Stone Member' and 'Zig Zag Chalk' meets the less permeable 'West Melbury Marly Chalk' which underlies the proposed development. As a result, it is thought that the groundwater underlying the proposed site does not contribute to the Nine Wells spring line.

4.1.2 Initial soakaway testing indicates a low rate of infiltration to groundwater. Reliance on infiltration measures alone will not be possible.

4.1.3 It is anticipated that surface water from the development will drain to attenuation facilities that will discharge, controlled at agreed limited rates, to the field ditches. Although the potential infiltration is low, sustainable drainage features will be used to replicate the existing limited recharge to the ground, and provide appropriate trains of water quality control. There is opportunity in the masterplan to provide these attenuation facilities in the form of open ponds, swales, rain gardens and urban realm water features, urban rills and bio-retention areas of the on-site roads. Underground storage tanks are not proposed, and an integrated approach with the landscape strategy is favoured.

4.2 Site Constraints

4.2.1 A review of the existing drainage regime on site has been considered as well as the constraints encountered on the neighbouring Cambridge Bio-Medical Campus site to the north.

4.2.2 Initial soakaway testing indicates a low rate of infiltration, which suggests that a limiting discharge rate of 2 l/s/ha may be quite conservative. Restricting runoff to this discharge rate, with low potential to infiltrate, requires a large volume of attenuation relative to the proposed developed impermeable area.

4.2.3 It is proposed that runoff from the site outfalls to the existing ditch that runs along the northern site boundary. The bed level of this ditch is approx. 1.2m below the adjacent plot level which may restrict the depth of attenuation features, thereby increasing the required land area. This will also restrict the depth of any other gravity drainage networks associated with the proposed development, and techniques to convey flow as close to the surface as possible will be required.

4.2.4 As the rate of infiltration is currently quite low, it is thought the increased impermeable area associated with the proposed development will not significantly reduce the recharge rate of the underlying water table. The proposed developable area is approximately 65% of the entire plot area, which shall allow a significant proportion of the site to be utilised as permeable green infrastructure. As the site is not connected to the Nine Wells aquifer, any minor residual impact on recharge will not impact on the spring flows.

4.2.5 The existing topography of the site is generally quite flat. There is a high point to the south-east corner, and the site generally falls east-west from 15.6m to 14.15m at a gradient of approx. 1 in 400. As a result, the siting of attenuation features should be quite flexible.

4.3 Proposed Strategy

4.3.1 A maximum allowable discharge rate has been adopted based on the Cambridge Biomedical Campus Phase 2 FRA which set out a rate of 2 l/s/ha, based on consultation with the EA and Cambridge City Council. This rate has been adopted at the neighbouring Cambridge Bio-Medical Campus site and the wider catchment area.

4.3.2 This limiting discharge rate will be applied to rainfall events up to the 1 in 100 year plus climate change scenario. This is below the existing greenfield runoff rates, estimated at 4.5 l/s/ha in a 1 in 100 year rainfall event, based on the IH 124 method and therefore demonstrates the site can provide a positive betterment to catchment run-off.

4.3.3 It is critical that SuDS are integrated into all areas of the development, in order to prevent any detrimental impact downstream and particularly to Nine Wells. The SuDS selected for the proposed development should be chosen based on the principles set out in the SuDS Manual (CIRIA C753) and the CCC SuDS Design and Adoption Guide, relative to the site constraints discussed above.

4.3.4 A key principle of both of the above guidance documents is the 'SuDS Management Train'. This promotes the management of runoff as close to source as possible with a series of treatment processes on site before discharging to a regional pond or to an existing watercourse.

4.3.5 To control runoff at source, rainwater harvesting can be employed to capture roof runoff and re-use in buildings. This should be considered further as part of the detailed design stage. Runoff can also be controlled at source by infiltrating directly into the ground, however, this is considered unfeasible at this stage based on the known ground conditions and initial soakaway testing.

4.3.6 The next stage of treatment considers features such as filter strips, swales, canals and rills, and bio-retention systems, which will again be inhibited in terms of infiltration potential but can provide siltation and pollution control, whilst promoting bio-diversity and the visual appearance of the development. Due to the shallow depth of the existing outfall ditch, some of these features will be more practical than deeper positive drainage networks.

4.3.7 It is proposed that on-plot roads will drain directly to bio-retention swales with bio-retention gardens and rills used to attenuate/convey flows from roofs and other hardstanding areas.

4.3.8 As discussed above, the main constraint is the volume of attenuation required. Open attenuation features such as local and/or regional ponds are considered preferable to underground storage units which are difficult to maintain, less visible and therefore more difficult to monitor in terms of flood risk and do not promote environmental or social betterment.

4.3.9 It is therefore proposed that a large attenuation pond is located at the low lying western site boundary to provide most of the attenuation volume required. This will be supplemented by a long attenuation/conveyance ditch feature located along the northern plot boundary as well as some local ponds within the built development area, where feasible. The ditch will provide attenuation whilst also providing an outfall point that will help to ensure that drainage runs from buildings/hardstanding are kept short and therefore shallow. These features are shown on the proposed site masterplan.

4.3.10 The masterplan makes adequate space for water. The proposed attenuation features have been sized based on the assumed impermeable area associated with the initial development proposals and will need to be refined as part of a more detailed surface water management design. The masterplan currently allows a minimum of 5,750 m^3 surface water storage to mitigate the surface water run-off from the site.

4.3.11 It should be noted that the above features do not consider off-plot overland flows which are discussed in the Flood Risk section above; combining these measures at detailed design stage will provide a more integrated approach and will maximise the aquatic habitat that can be created.

5 Limitations, Conclusions and Recommendations

5.1 Limitations

5.1.1 This study is limited to the consideration of surface water run-off, from both off-site and on-site sources. Flood risk from fluvial sources is considered manageable, and the modelling has indicated that ditch capacity is not exceeded. Further review of flows and levels in the ditch should be undertaken at the Flood Risk Assessment stage to confirm the Flood Zone classification of the site.

5.1.2 There is an unknown risk of flooding from groundwater sources, though this is thought to be a manageable risk. Further work will be required (e.g. borehole water level monitoring) to evaluate this residual risk and evaluate mitigation measures.

5.1.3 Sensitivity testing undertaken in the surface water flood risk modelling for this study has shown a high sensitivity to percentage rainfall loss. This uncertainty has been allowed for as freeboard in the storage volume estimates.

5.1.4 This study has considered the site at an initial masterplanning stage. The development plans continue to evolve and different site usages and impermeable coverage may require alternative surface water management solutions to be developed.

5.2 Conclusions and Recommendations

5.2.1 This study has shown that surface water run-off from adjacent areas is a source of potential flood risk at the site. Flooding was predicted to occur in the 30 year event and all more extreme flood events. Flood enters the site from the south and eastern boundaries, and flows in a south-westerly direction across the site.

5.2.2 The flood risk can be mitigated by construction of a perimeter ditch to catch the surface water run-off and convey it to the main drainage network. Mitigation measures including flow control and an appropriate storage volume will be required to prevent any detrimental impact on water levels and flows downstream. Storage could be provided in the form of attenuation ponds, online weirs with widened ditches, or in combination with the on-site surface water drainage system. Off-site measure to contain overland flow might also be considered with the landowners landholdings.

5.2.3 Additional surface water run-off will be generated by the impermeable surfaces of the proposed development. A maximum allowable discharge of 2 l/s/ha is suggested, in line with adjacent developments and consultation with Cambridge City Council. A large storage pond in the lowest western corner of the site is suggested, with water conveyed to the pond via a conveyance ditch, swales, rills and rain gardens that further supplement the storage. A management train of SuDS measures is strongly recommended to be integrated into the development, to promote water quality as a primary driver and mitigate the impacts of water quality, and quantity of the receptor watercourse from the Nine Wells Spring.

Appendix B Hydraulic Model Construction

B.1 Data Sources

B.1.1 The following data was used to assess catchment boundaries and construct the hydraulic model:

- Site topographic and watercourse survey completed by Survey Solutions in June 2016. Due to Network Rail access constraints, it was not possible to survey the drain adjacent to the rail line or the culvert underneath the railway. The survey is included in Appendix E
- 1m and 2m resolution LiDAR data, provided by the Environment Agency. The catchment coverage is incomplete (Figure 10).
- Ordnance Survey Terrain 50 (OST50) data, downloaded from the Ordnance Survey. This data was used to supplement the LiDAR data in areas of no LiDAR coverage.

B.1.2 The lack of survey for the drain adjacent to the Network Rail line leads to uncertainty in the results. Assumptions have been made in the modelling regarding drain capacity. Site inspection indicated that this drain is in poor condition and overgrown, and maintenance is recommended. However the culvert underneath the railway is large and unlikely to cause a backwater effect on flows upstream.

B.1.3 The terrain data was combined into one digital elevation model (DEM) by overlaying OST50, 2m lidar, 1m lidar and the site survey respectively (Figure 11). The data was checked for consistency and errors; while there was some evidence of poor cleaning of structures (e.g. Addenbrookes site), these areas lay outside the catchment boundary.

B.2 Rainfall and Flow Estimates

B.2.1 The catchment boundary was estimated using the combined DEM. This indicates a catchment area of approximately 5 km^2, draining areas of the Gog Magog hills to the south-east. This catchment area is significantly larger than that modelled in the Cambridge SWMP.

B.2.2 Rainfall was estimated for the catchment using both FEH99 (FEH Cd-Rom) and FEH13 (FEH webservice) data. The FEH13 data was released in October 2015 and represents an improved dataset based on up-to-date rainfall observations and more detailed spatial resolution. The FEH13 rainfall estimates were larger than the FEH99 data for storm durations greater than 4 hours. As the critical storm duration for the catchment was found to be 6 hours, the more conservative FEH13 rainfall estimates were adopted.

B.2.3 Infiltration rates were estimated by comparing flows estimated by the model against a standard FEH flow estimate. This method was used in the Cambridge SWMP in preference to a standard UK-wide adjustment factor. After considering a number of flow estimation methods, the ReFH2 method was selected. The resulting peak flow estimates using this method are low, but consistent with the highly permeable catchment nature. In order to match these flows, rainfall inputs were scaled by 82.5%. This scaling factor is consistent with the catchment's very low Standard Percentage Runoff of 2.66.

B.2.4 Full details of the rainfall and flow calculations are included in Appendix C .

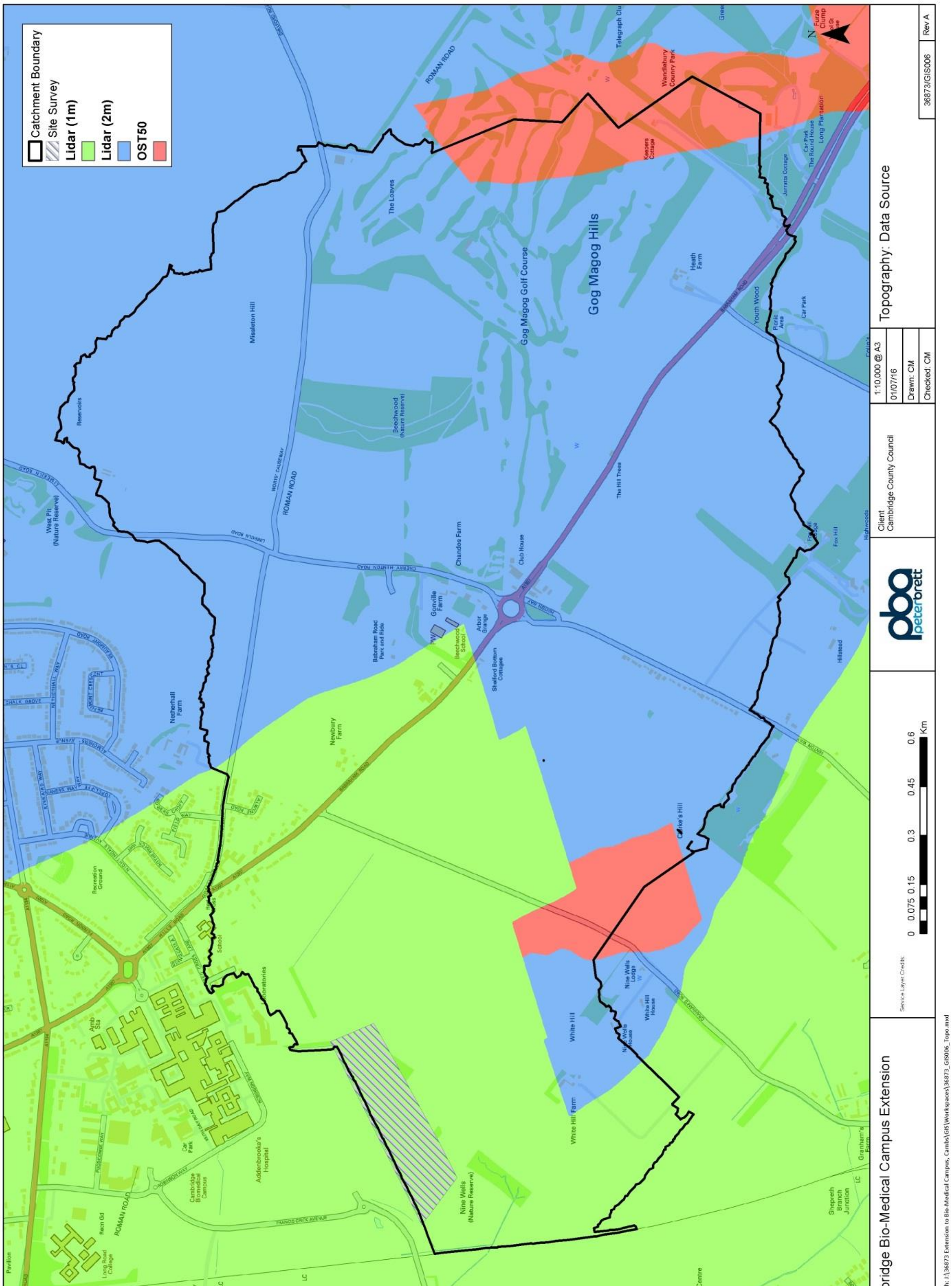


Figure 10: Data Sources for Digital Elevation Model

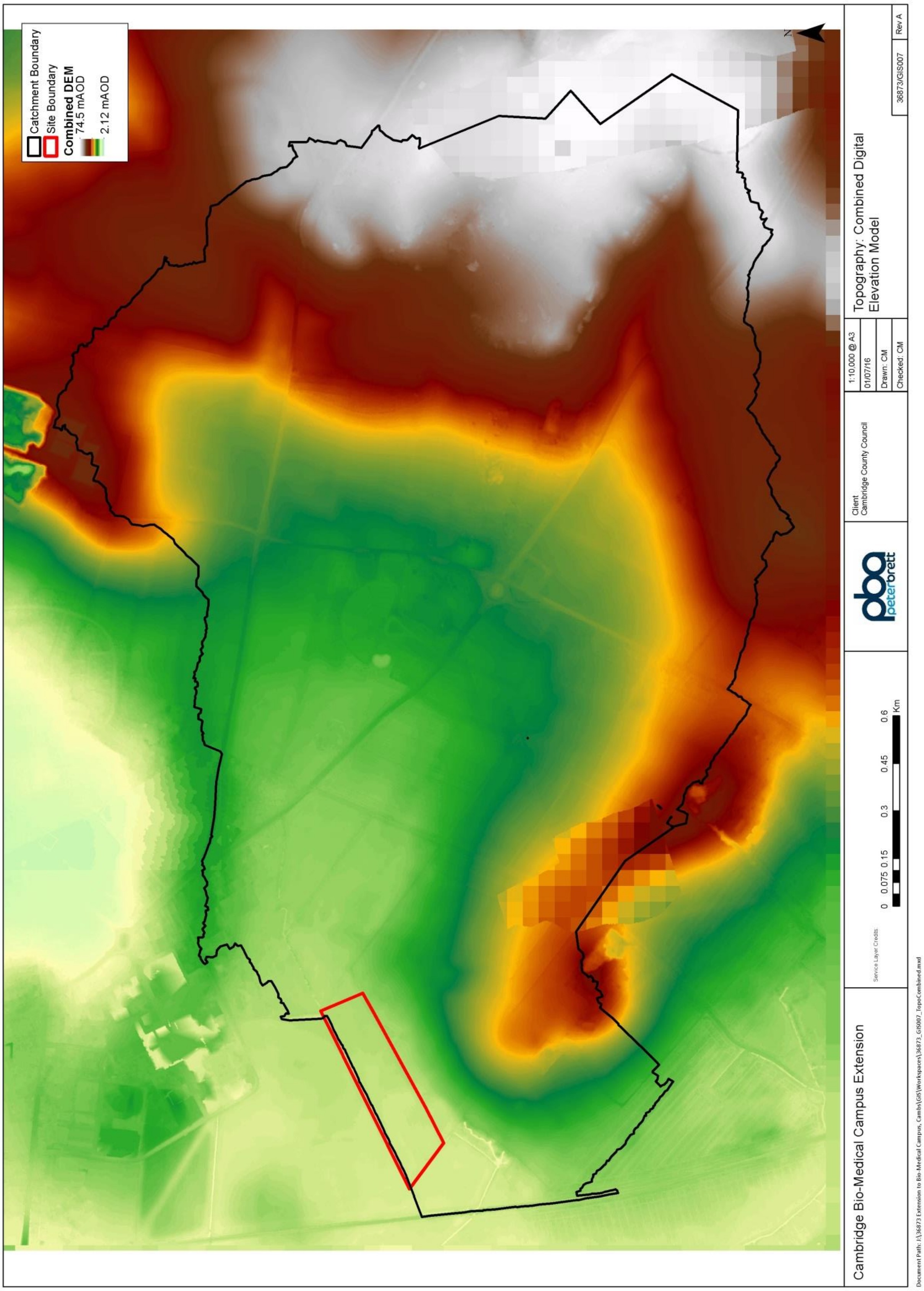


Figure 11: Combined Digital Elevation Model

B.3 Hydraulic Modelling

B.3.1 Due to the size of the catchment to be modelled, a 1D-2D linked hydraulic modelling approach was used. This allows a medium grid size to be used across the catchment (5m) while also ensuring the full storage capacity and hydraulics of the drain are captured. The model was constructed using FloodModellerPro (previously ISIS) software.

B.3.2 The 1D model was constructed based on the available watercourse survey. The model extents are shown in Figure 12. Cross-section data was reviewed and assigned panel markers and Manning’s roughness values (0.03, reflecting a well-maintained channel). Two culverts were included to represent the cycle path crossings with spills in parallel for overtopping. A constant ‘sweetener’ flow of 0.02 m^3/s was used with a Preissman slot to prevent the channel from drying out. A normal depth boundary of 0.001 was applied to the downstream cross-section.

B.3.3 The 2D model was constructed as a regular 5m grid based on the combined DEM. The model extents are shown in Figure 12. A uniform Manning’s roughness value of 0.1 was applied, reflecting the mostly rural land use in the catchment and following the Environment Agency’s standard guidance for roughness values. Buildings were not represented in the domain due to their sparsity. The 2D model was linked to the 1D model using level boundaries based on the DEM. A normal depth boundary was applied to the active area boundary to allow water flowing away from the catchment to leave.

B.3.4 The model ran stably for all simulations, based on a time step of 2 seconds, using the ADI solver. Mass balance and error files were checked and no areas of concern found.

B.4 Post-Development Amendments

B.4.1 The 1D hydraulic model was amended to include new drainage channels around the perimeter of the site. The intention of these drains is to collect surface water run-off entering the site from surrounding land and direct it into the drainage network to avoid overland flow across the site itself. The drains could also carry outflows from attenuation ponds, depending on their position in the site. Details of the model amendments are shown in Figure 13.

B.4.2 It is assumed that flows from the development plot will be attenuated to a maximum rate of 2 l/s/ha. For the 9.167 ha plot, a post-development peak runoff of 0.02 m^3/s was therefore estimated. This was input as a constant flow into the new drainage channels.

B.4.3 Initial model tests indicated that the perimeter drains resulted in an increase in peak flows downstream due to more efficient capture and conveyance of flood flows. Therefore orifice units were used to limit outflows from the perimeter drains to ensure no increase in peak flows downstream. The size of the drainage ditches was enlargened to provide sufficient storage for the attenuated flows.

B.5 Climate Change and Sensitivity Tests

B.5.1 Climate change was modelled for the 100 year event for both the baseline and post-development scenarios. Climate change was allowed for using the “upper end” estimate of 40% increase in rainfall, following the latest national guidance.

B.5.2 Both the baseline and post-development scenarios were tested for sensitivity to Manning’s roughness and to rainfall losses, using the 100 year event. Manning’s roughness was altered by ±0.02 for both channel and floodplain sections to reflect possible seasonal changes in vegetation cover. Rainfall losses were simulated at 39% and 50% to reflect standard parameters used in other national and local surface water flood mapping.

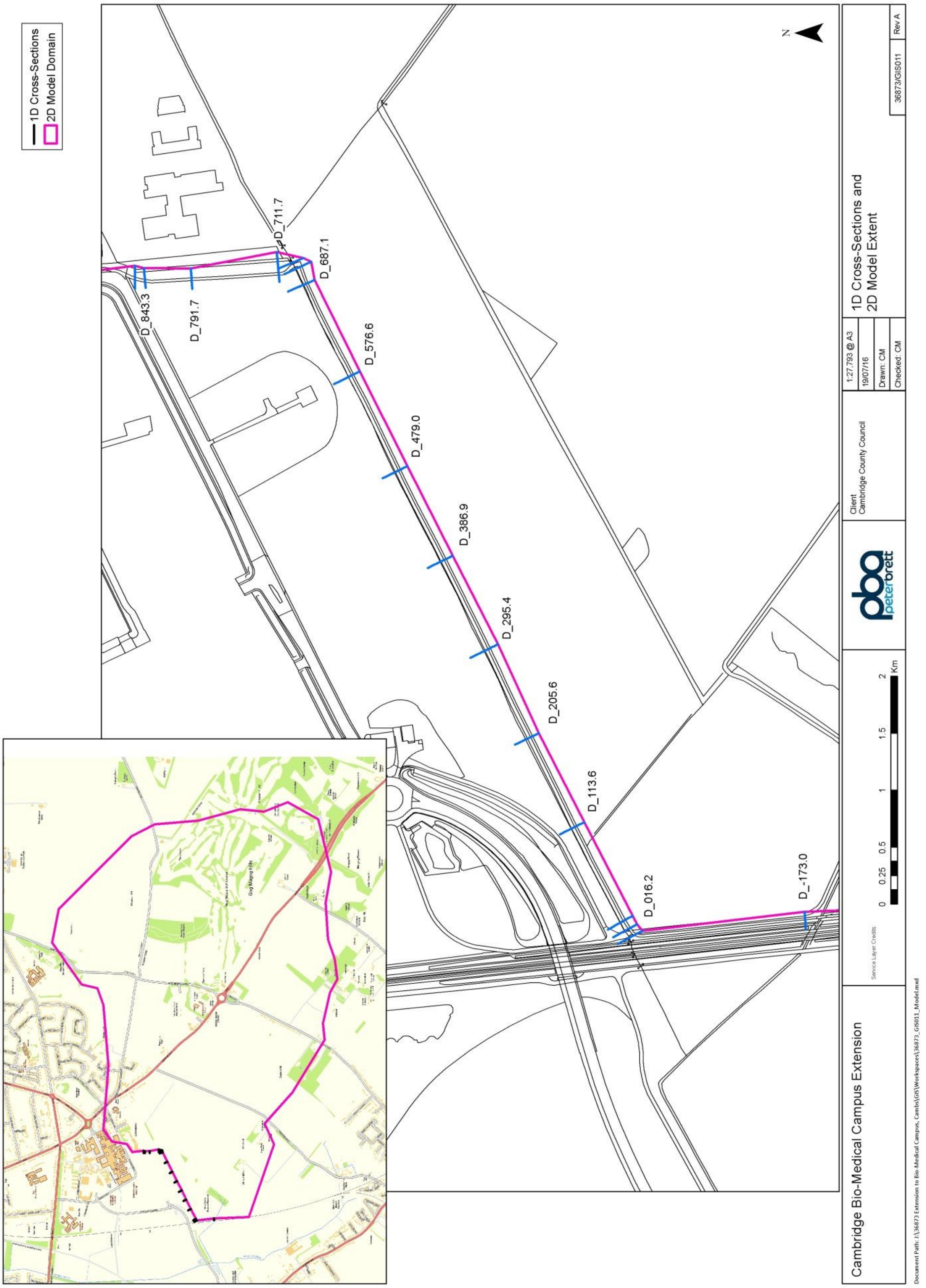


Figure 12: Baseline Model Structure

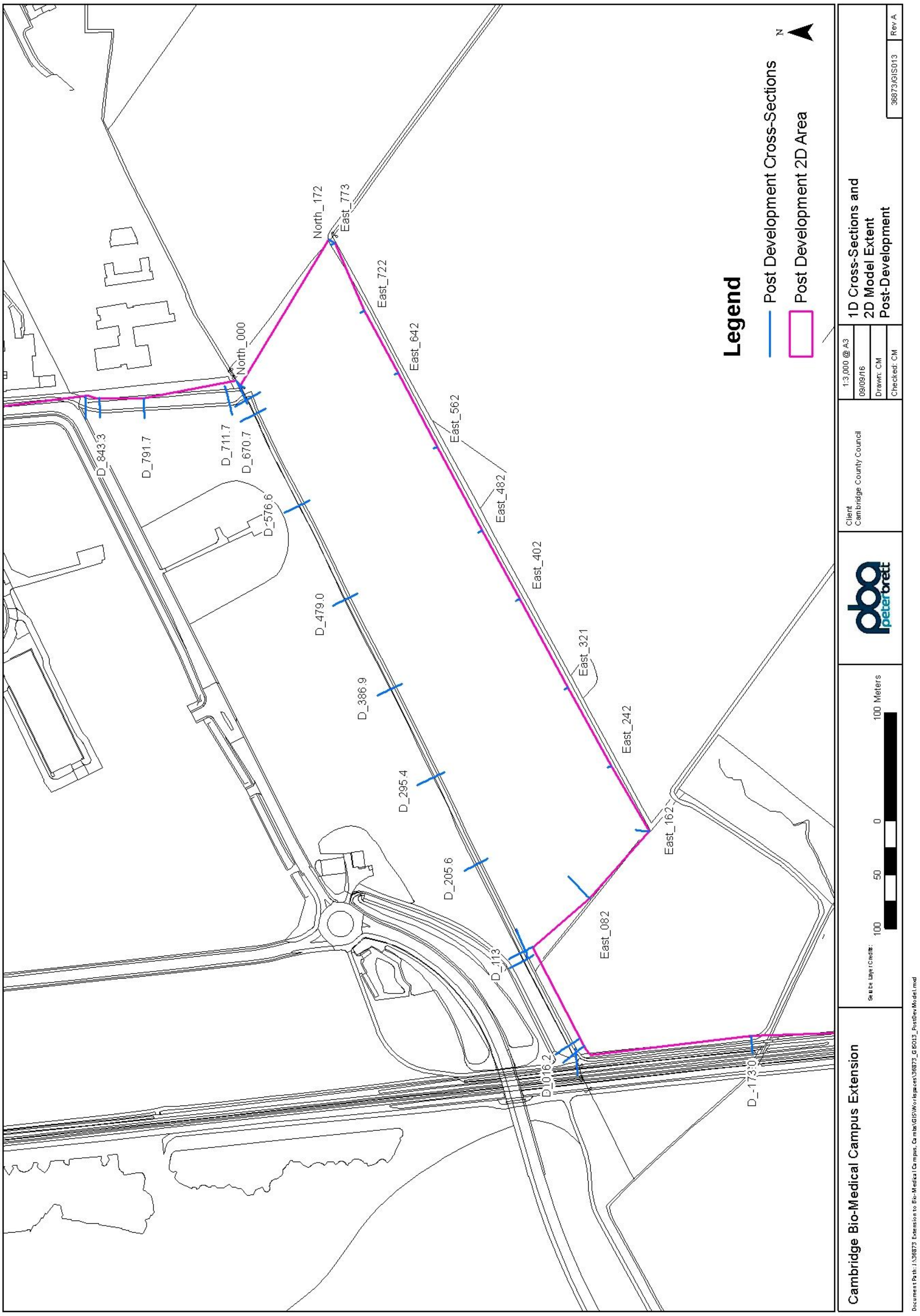


Figure 13: Post-Development Model Structure

Appendix C Rainfall and Flow Calculations

C.1 Rainfall Estimates

C.1.1 Rainfall was estimated for the point TL 46316 54451 using both the FEH99 (FEH CD-Rom) and the FEH13 (FEH web service) data sets. The data are compared in Table 1 showing that the FEH13 data gives higher rainfall estimates for all return periods for storm durations for 4 hours and longer.

Rainfall Method	Storm duration (hrs)	Return period (yrs)					
		10	20	50	100	200	500
FEH99	1	23.91	30.04	40.36	50.36	62.79	83.99
FEH13	1	24.59	29.89	37.48	44.09	52.01	65.16
FEH99	2	28.17	34.94	46.16	56.87	70.02	92.13
FEH13	2	31.64	37.8	46.97	55.6	66.52	84.05
FEH99	4	33.19	40.63	52.79	64.23	78.09	101.06
FEH13	4	38.39	45.36	56.33	67.37	81.64	102.46
FEH99	6	36.53	44.38	57.09	68.96	83.23	106.68
FEH13	6	41.77	49.21	61.15	73.57	89.31	111.35

Table 1: Rainfall estimates.

C.1.2 Rainfall profiles were generated for both winter and summer events, applying an areal reduction factor for catchment area and seasonal correction factors. For the 100 year event, storms of 1, 2, 4, 6 and 8 hrs for both winter and summer events were tested in the model to identify the critical event. The 6 hour summer event was found to be critical. Climate change was evaluated by increasing rainfall estimates by +40%, following the latest DEFRA guidance.

C.2 Flow Estimates

C.2.1 Catchment descriptors were extracted from the FEH CD-Rom for the catchment that was the closest match to the catchment identified using LiDAR (TL 46600 54850). Area and DPLBAR were amended to reflect the amended catchment boundary, all other descriptors were unaltered. Key descriptors are listed in Table 2.

C.2.2 A number of methods were considered for estimating flows:

- FEH Rainfall-Runoff method: not used as known to overestimate.
- ReFH Rainfall-Runoff method: not used as unsuitable for permeable catchments.
- ReFH2 Rainfall-Runoff method: used.
- FEH “improved” statistical method (SC050050): used.

Descriptor	Value	Comments
AREA	4.8	Amended to 5.25 to match amended catchment boundary
BFIHOST	0.994	Extremely high – very permeable catchment
DPLBAR	2.13	Amended to 2.48 to match amended catchment boundary
DPSBAR	32.3	
FARL	1	No reservoir influences
FPEXT	0.0974	No significant floodplain extents
PROPWET	0.26	
SAAR	545	
SPRHOST	2.66	Extremely low – very permeable catchment
URBEXT1990	0.0026	Low- rural catchment
URBEXT2000	0.0031	Low- rural catchment

Table 2: Catchment Descriptors.

Statistical Method

C.2.3 QMED was estimated as 0.052 m^3/s using catchment descriptors. The possibility of using donor data from gauging stations on the neighbouring River Granta and River Cam was investigated but the catchments were found to be too dissimilar in terms of size and permeability.

C.2.4 A pooling group was generated using the Winfap-FEH v4.1 (May 2016) dataset. The initial pooling group was reviewed for discordance and outliers, and adjusted for a length of 500 years. The final pooling group is listed in Table 3. The pooling group remained strongly heterogeneous despite review, due to the subject site's small size and extremely high permeability, and this limits confidence in the results.

C.2.5 Following FEH guidance the Generalised Logistic (GL) method was applied to estimate the pooled flood frequency curve. Other distributions were reviewed and the GL method found to give the most conservative result. Peak flow estimates are summarised in Table 4.

ReFH2 Method

C.2.6 Catchment descriptors and FEH13 catchment data were downloaded from the FEH Web Service and imported into the ReFH2 software. Catchment area and DPLBAR were amended following Table B2. No other amendments to catchment descriptors were made.

C.2.7 Storm durations from 1 to 15 hours were tested, and the critical storm duration found to be 7.5 hours. Flow hydrographs were generated for a range of flood return periods using standard equations. The results are summarised in Table 4.

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27051 (Crimple @ Burn Bridge)	1.474	40	4.539	0.222	0.149	0.423
27073 (Brompton Beck @ Snainton Ings)	1.805	32	0.813	0.197	-0.022	0.991
45816 (Haddeo @ Upton)	1.867	19	3.456	0.324	0.434	0.737
26802 (Gypsey Race @ Kirby Grindalythe)	1.934	13	0.109	0.261	0.199	0.239
25019 (Leven @ Easby)	1.991	34	5.538	0.347	0.394	0.741
76011 (Coal Burn @ Coalburn)	2.033	35	1.84	0.169	0.333	1.201
28033 (Dove @ Hollinsclough)	2.118	33	4.666	0.266	0.415	0.628
20002 (West Peffer Burn @ Luffness)	2.414	41	3.299	0.292	0.015	1.388
27010 (Hodge Beck @ Bransdale Weir)	2.431	41	9.42	0.224	0.293	0.202
44008 (South Winterbourne @ Winterbourne Steepleton)	2.505	33	0.42	0.395	0.332	1.08
203046 (Rathmore Burn @ Rathmore Bridge)	2.506	30	10.934	0.136	0.091	0.891
25011 (Langdon Beck @ Langdon)	2.518	26	15.878	0.241	0.326	1.244
36010 (Bumpstead Brook @ Broad Green)	2.518	45	6.759	0.418	0.228	1.719
25003 (Trout Beck @ Moor House)	2.755	39	15.164	0.176	0.291	0.624
206006 (Annalong @ Recorder)	2.767	48	15.33	0.189	0.052	1.525
26803 (Water Forlornes @ Driffield)	2.871	13	0.684	0.215	0.069	2.368
Total		522				
Weighted means		522		0.255	0.225	

Table 3: Final pooling group.

Flood return period (yrs)	Peak flow (m^3/s)	
	Statistical Method	ReFH2 Method
2	0.048	0.110
5	0.069	0.163
10	0.084	0.202
25	0.106	0.261
30	0.111	0.275
50	0.126	0.321
100	0.149	0.409
1000	0.255	0.865

Table 4: FEH Statistical and ReFH peak flow estimates.

Choice of Method

C.2.8 The ReFH2 flow estimates are two to three times larger than the FEH Statistical methods. This variation highlights the uncertainties associated with flow estimates for small highly permeable catchments. For a conservative approach, the ReFH2 flow estimates have been selected.

C.3 Infiltration Rates

C.3.1 There is not yet any national guidance on how to estimate loss rates for direct rainfall catchment-scale modelling. Rainfall losses include infiltration, evapotranspiration, and local storage of rain water. Previous studies include:

- The Environment Agency's Flood Map for Surface Water (FMfSW) modelling (2010) assumed generic losses of 39% for rural areas and 70% for urban areas nationwide.
- The Environment Agency's Updated Flood Map for Surface Water (uFMfSW) modelling (2013) used the ReFH rainfall loss model and soil moisture capacity parameters from the National Soil Resources Institute data for rural areas.
- The Cambridge SWMP applied a loss of 50% after comparing outflow hydrographs for Vicar's Brook against FEH statistical peak flow estimates.

C.3.2 The approach adopted in the Cambridge SWMP was used here. The study catchment here is extremely permeable and it is plausible that a very significant proportion of rainfall infiltrates into the catchment. The lack of drains mapped within the catchment supports this.

C.3.3 Losses varying from 0% to 90% were applied to the 100 year critical duration rainfall event and the resulting outflows in the drain assessed. The 100 year target peak flow of 0.409 m^3/s was met when a loss of 82.5% was applied. This loss is larger than those used by the EA and Cambridge SWMP, but is reflective of the extremely permeable nature of the catchment.

C.4 Additional Flows

C.4.1 Cambridge City Council have indicated that development to the south of Robinson Way discharges unattenuated surface water directly into the ditch. MicroDrainage software was used to estimate approximate peak runoff rates from this impermeable area (Table 5). These peak flows were input as a constant flow to the ditch, to avoid any issues of timing of flow hydrographs. This approach is overly conservative and therefore allows for uncertainties in magnitude and timing of surface water discharges from the Addenbrookes site.

Flood return period (yrs)	Peak flow (m^3/s)
30	0.16
100	0.22
100 + 40% climate change	0.31
1000	0.55

Table 5: Peak flow estimates for development south of Robinson Way

Appendix D Geotechnical Appraisal

Job Name: Cambridge Bio-Med Park Extension

Job No: 36873

Note No: GEO1 Rev 1

Date: September 2016

Prepared By: P. Webb / G.Bates

Subject: **Ground Conditions - Baseline Opportunities & Constraints**

Item	Subject
1.	**Scope** • Peter Brett Associates (PBA) have been commissioned by Carter Jonas (the Agent) on behalf of Cambridgeshire County Council (the Client) to provide a desk based assessment to establish the ground conditions, geology and hydrogeology of the site and immediate surroundings. • This note provides an assessment of surface and groundwater conditions in context of flood risk, potential drainage solutions for the development and the impact the development could have on the overall groundwater and surface water regime. • PBA will review geological and hydrogeological maps, historical maps, memoirs, published borehole and well records and other relevant information in order to establish a ground and groundwater model for the site and surroundings. • A conceptual site model will be developed that permits an assessment to be made of the possible impacts of development on groundwater and the Nine Wells Local Nature Reserve.
2.	**Introduction** • The Site is located to the south of Dame Mary Archer Way, Trumpington, Cambridgeshire and covers an area of approximately 7.8ha. • The Site is located to the south of Addenbrooke's Hospital, with the proposed development forming an extension to the current hospital site. • A planning application has been made to Cambridge City Council (Ref 16/0176/OUT) for the adjacent land immediately to the north of the subject Site. The pertinent publically available information for this scheme has been reviewed as part of this assessment. • The Site location is presented on Figure 1 below.

<table>
<tr><th>Item</th><th>Subject</th></tr>
<tr><td></td><td>


Figure 14 Site Location Plan
</td></tr>
<tr><td>3.</td><td>
Current Land Use

The Site currently comprises an open agricultural field, with footpaths along the eastern, southern and western boundaries. A cycle way, which is part of National Cycle Network Route 11, forms the northern boundary of the Site with a surface water drainage ditch beyond.
The helipad for East Anglian Air Ambulance lies approximately 30m to the north of the Site, with Dame Mary Archer Way a further 100m to the north.
Nine Wells Local Nature Reserve[1] is located approximately 40m to the south of the Site. The nature reserve contains a number of chalk springs that feed streams running north towards Cambridge.

</td></tr>
<tr><td>4.</td><td>
Site Reconnaissance

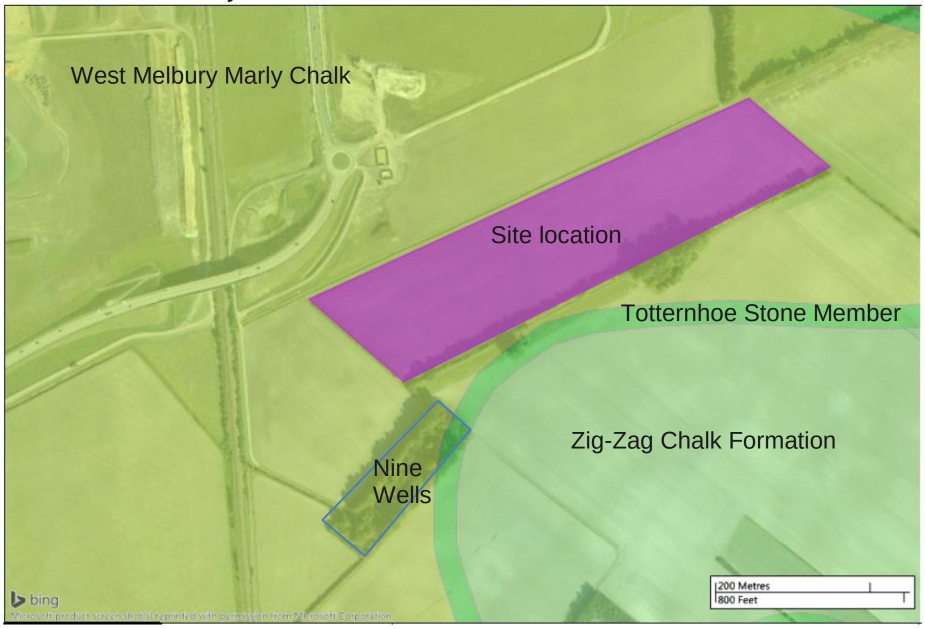
A site walkover was undertaken by a PBA Engineer on 25th May 2016. This was to confirm the current land use and make on-site observations of any ground or surface water features. Potential sources of contamination and potential receptors were also recorded and used to inform the conceptual model.
During the site walkover a utility cover for an unknown service was identified in the northern part of the Site, between the Cycle Path and field boundary.
The current land use was confirmed during the walkover, with crop (possibly peas) present on Site.
Tents or hides were noted on Site and in surrounding fields.
Water was present in the springs at Nine Wells.
The ditch along the northern boundary of the Site was dry.
A photographic record of the site reconnaissance is enclosed.

</td></tr>
<tr><td>5.</td><td>
Historical Land Use

1886: Site comprises an agricultural field. There is a drain north of site and a

</td></tr>
</table>

[1] http://lnr.cambridge.gov.uk/nature_reserve/nine-wells/ last accessed 02/06/16

Item	Subject
	pathway running along the south of the Site. The Great Eastern Railway line is located 100m west of site. Two springs are located 80m south of the Site. Nine Wells Monument is recorded 80m south of the site. Nine Wells House located to the south. Clunch Pit and Old Clunch Pit are located at Nine Wells House. • 1902-1903: No significant changes. • 1972-1974: Addenbrooke's Hospital has been constructed 300 m to the north. Pits to the south now recorded as disused. Development and expansion of Cambridge to the north. • 1983-1992: Site remains an open field. • In summary the Site has remained undeveloped and comprises an agricultural field.
6.	**Geology** • Based on the British Geological Survey (BGS) digital mapping[2], there are no superficial deposits recorded on the Site. • The bedrock geology beneath the Site comprises West Melbury Marly Chalk Formation. Experience from PBA ground investigations in this geological formation indicates that it generally consists of uncompact grey white SILT • The Totternhoe Stone Member and Zig-Zag Chalk Formation outcrops immediately south-east of the Site. These formations sit on higher topographically higher ground. Figure 2 shows the Site location in relation to the underlying geology. • There are two BGS borehole records within 500m of the Site: 1. There is an Environmental Agency borehole approximately 50m north of site (BGS reference TL45SE65). The borehole is recorded as 10m in depth, has a water strike level of 5.8m (below ground level) and a monitored rest water level of 1.77m below ground level (bgl). The strata is recorded as Topsoil to 0.45m underlain by sandy CLAY to 10m. 2. Approximately 200 metres north-west of the Site (BGS reference TL45SE64) is a borehole 10m deep, with a water strike level of 7.3m bgl and a monitored water level of 1.94m bgl. The recorded geology comprises 0.6m Topsoil underlain by CLAY to 10m.  **Figure 15 Site Location and Geology Plan**

[2] http://mapapps.bgs.ac.uk/geologyofbritain/home.html last accessed 02/06/16

Item	Subject
7.	**Groundwater** • The Site does not lie within a Source Protection Zone. The nearest Source Protection Zone is approximately 3km to the north of the Site. • The underlying Bedrock Geology (Chalk) is classified as a Principal Aquifer. • As shown on Figure 2; Nine Wells Local Nature Reserve lies on a spring line where the Totterhole Stone Member and Zig Zag Chalk meets the less permeable West Melbury Marly Chalk. • The hydrogeological map of the area[3] shows a “principal groundwater divide in Chalk” to the east of the Site. This is likely to correlate with the change in geology between the Zig Zag Chalk and West Melbury Marly Chalk. • The Site does not lie within a vulnerable water zone. Groundwater flow direction is unknown. However, it is likely to be to the west towards the Springs at Nine Wells and the stream fed by the springs. • Based on available BGS borehole logs, groundwater is likely to be present within the underlying West Melbury Marly Chalk at a depth of approximately 6 – 7m bgl.
8.	**Surface Water** • The spring at Nine Wells local nature reserve feeds the surface water stream to the west of the Site flowing north-west. The stream joins Hobson’s Brook 1 kilometre west of site. • There is a ditch at the northern boundary of the Site, the ditch was recorded as dry at the time of the walkover. • There is a ditch located 100m west of the Site following the route of the railway line. • Road drainage was recorded during the walkover to the west of the Site. There are two surface water abstractions located south-west of the Site at Nine Wells: 1. Water abstraction from a stream or ditch. 2. Water abstraction from stream at a rate of 23 m^3/day for spray irrigation. A flood modelling and drainage strategy report is being prepared by PBA, this report contains further information on the surface water regime at the Site.
9.	**Environmental Data** • Addenbrooke’s Hospital has a license for radioactive substances (for medical purposes.) • Cambridge University Hospital has a license for incineration of hazardous waste from Integrated Pollution and Control and from Local Authority Pollution Prevention and Controls. • There are no landfill sites recorded within 500 m of the Site. • Contemporary Trade Directory Entries exist for hospital operations 200m to the north of the Site associated with Addenbrooke’s Hospital.
10.	**Environmental Assessment – Conceptual Site Model** Based on the historical mapping, Site walkover and review of available environmental data, a risk assessment has been undertaken for the development based on existing Site conditions. The underlying principle is the evaluation of *pollutant linkages* in order to assess whether the presence of a source of contamination could potentially lead to harmful consequences. A pollutant linkage consists of the following three elements: a. A **source** of contamination or hazard that has the potential to cause harm or pollution.

[3] Hydrogeological maps of the United Kingdom, 1984. Sheet 14. Hydrogeological map of the area between Cambridge and Maidenhead including parts of hydrometric areas 33,38 and 39. 1,100,000 Scale, British Geological Survey 1984

Item	Subject
	b. A **pathway** for the hazard to move along / generate exposure. c. A **receptor** which is affected by the hazard. For each potential pollutant linkage identified the risk is estimated through consideration of the magnitude of the potential consequences and the likelihood or probability of an event occurring. **Identified On-Site Potential Sources** • Carbon dioxide ground gas generated by the underlying Chalk strata. The Environmental Health Officer for Cambridge City Council has identified this as a potential source for the planning application adjacent to the north of the Site. • Fertilizers and associated with the Site's agricultural land use. **Identified Off-Site Potential Sources** • Engine coolant and antifreeze used on the helipad to the north of the Site. • Radioactive substances and waste incineration from Addenbrooke's Hospital to the north of Site. **Identified Potential Pathways and Receptors** • Accumulation of carbon dioxide ground gas in future buildings impacting future occupants (Site users). • Leaching of herbicides, fertilizers and organic compounds (coolant and antifreeze) into the underlying Chalk Principal Aquifer. • Run off of fertilizers and herbicides into on-site and off-site surface water features (ditch along northern boundary of the Site). • Inhalation of dust and chemicals from incineration to the north of the Site by future site users. **Nine Wells** There are not considered to be any significant on-site sources of potential contamination that would impact Nine Wells Local Nature Reserve. The site is underlain by West Melbury Marly Chalk Formation. The primary source of water emerging at Nine Wells spring, is from the Tottenhoe Stone Member and Zig Zag Chalk Formation located to the south and east of the Site. Based on available BGS borehole logs, groundwater is likely to be present within the underlying West Melbury Marly Chalk at a depth of approximately 6 – 7m bgl. Based on this the groundwater at the Site and Nine Wells Local Nature Reserve are at different depths. Therefore if there were to be any impact on the groundwater at the Site, the hydrogeological regime at the Site is such that any impact on groundwater is unlikely to impact Nine Wells Local Nature Reserve. However, the potential exists for Nine Wells to be impacted by surface run off during construction phase if the works are not properly controlled.
11.	**Geotechnical Assessment** Following review of information obtained from the Envirocheck data included in the planning application[4] to the north of the Site, the following geotechnical assessment has been made: • The Site is not considered to be in a coal mining area.

[4] https://idox.cambridge.gov.uk/online-applications/applicationDetails.do?activeTab=documents&keyVal=O1VRDVDXK0X00

Item	Subject
	• The risk of a collapsible ground hazard is considered very low. • There is not considered to be a risk from running sand on-site. • There is not considered to be a risk of landslide hazards on-site. Based on PBA's experience of the Cambridge area we would agree with the conclusions of the above assessment. **Soil Re-use** • Reusing natural Chalk bedrock as engineered fill will require careful handling and control. It may not be possible to use chalk as subgrade on areas of hardstanding and imported fill may be required. **Cavities** • Based on a search of the PBA man-made and natural cavities database there are no recorded dissolution features located within 500 metres of the Site. **Foundations** • Shallow foundations are likely to be suitable for low to moderate foundations loads. Piled foundations may be considered for more heavily loaded structures. **Excavations** • Shallow excavations for foundations and drainage are anticipated to be stable in the short term. Close or continuous support will be required for any manned entry to excavations. **SuDS Potential** • Shallow infiltration drainage may be feasible in the underlying Chalk bedrock. In-situ was undertaken by PBA in the West Melbury Marley Chalk indicates that infiltration rates are likely to be in the order of 10^{-6} – 10^{-7} m/s. The results of the infiltration assessment are presented in Technical Note 36873/3501/GEO2.
12.	**Summary** • Based on historical and current land uses on-site, the potential risk associated with land contamination from past and current land use activity is considered to be generally low. However, there is considered to be an elevated risk from carbon dioxide ground gas originating from the chalk bedrock. The Local Authority Environmental Health Officer may require ground gas monitoring. • Based on the information reviewed in this assessment there is considered to be a low risk to groundwater and Nine Wells Local Nature Reserve. • There are not considered to be any significant geotechnical constraints on the Site. Reusing natural Chalk bedrock as engineered fill will require careful handling and control. It may not be possible to use chalk as subgrade on areas of hardstanding and imported fill may be required. • There are a number of potential opportunities at the Site: a. Infiltration drainage may be possible in the underlying chalk, if not, there are opportunities to use existing drainage networks. b. The Site does not lie within a source protection zone for groundwater. c. Soils on-site are likely to be classified as inert for off-site waste disposal, however, the final decision on waste classification is ultimately with the receiving landfill facility. • There are not considered to be any significant on-site sources of potential contamination that would impact Nine Wells Local Nature Reserve. The hydrogeological regime at the Site is such that any impact on groundwater is unlikely to impact Nine Wells Local Nature Reserve. However, the potential exists for Nine Wells to be impacted by surface run off during construction phase if the works are not properly controlled.

Appendix E Survey