Boyer

Submission Adur Local Plan Main Modifications

Response to Submission Adur Local Plan Main Modifications

Introduction

1.1 This note is a response on behalf of Hyde New Homes to the Submission Adur Local Plan Main Modifications published on the 15th June 2017.

1.2 Boyer, on behalf of Hyde New Homes, have made representations on the Adur Local Plan throughout the consultation process identifying those areas where the plan is in our view unsound, promoting the inclusion of the site known as New Salts Farm as a housing allocation and identifying how the Adur Local Plan can be amended so that it can be found sound.

1.3 These reps should be read in conjunction with all previous reps made throughout the Adur Local Plan consultation process and Examination in Public.

1.4 This note sets out where we consider the Main Modifications (MM) not to be sound and/or legally compliant and those changes necessary to make the Main Modification sound. Comments are also provided on the Addendum to the Sustainability Appraisal (SA) of the Adur Local Plan.

Addendum to Sustainability Appraisal

Intro

1.5 We would take this opportunity to reiterate that, as we have maintained throughout our reps to date, we have significant concerns about the inherent inconsistencies in the Sustainability Appraisals carried out to date which highlight that the whole process of preparing the Local Plan is unsound.

1.6 These identified inconsistencies have not been addressed in this latest Addendum and therefore we maintain that the Sustainability Appraisal is not sound, site selection has not been objective, there has not been an appropriate assessment of reasonable alternatives and therefore the proposed strategy in the Adur Local Plan is not the most appropriate strategy.

1.7 The Adur Local Plan therefore cannot be found sound as it is not based on a sound process of sustainability appraisal.

MM1

1.8 The OAN for Adur has increased from 5,820 dwellings over the plan period (in the Proposed Submission Adur Local Plan) to 6,825 dwellings in the Submission Adur Local Plan. This is an increase of 1,000 dwellings which is not an insignificant number. Despite this the increase has not been tested within the Sustainability Appraisal Addendum. The Council consider that the increase does not require an SA to properly test the increased OAN.

1.9 This is explained further at para 7.24 of the SA Addendum which states that the '*Adur Local Plan can only deliver a minimum 3,718 homes'* and that *'the level of housing proposed in the Adur Local Plan (a minimum of 3,718 homes) would still achieve a greater balance between the differing social, environmental and economic sustainability objectives than delivery of the full OAN'.*

1.10 It is not clear how the Council can draw a conclusion that an increase in the OAN of 1000 dwellings over the plan period does not warrant further consideration to understand whether or how they might seek to meet that need. We are of the view that in light of the significant shortfall, in order to plan positively to meet housing need in the district the Council should consider the allocation of additional sites for housing. It should also carry out a sustainability appraisal which considers additional reasonable alternatives and site options previously dismissed to properly understand the impact of not meeting housing need and ensure that the correct balance is struck in the Plan in regards to sustainable development.

1.11 It is also not clear how the Council can conclude that a greater balance is achieved between the different elements of sustainable development when only 54% of housing need is being met and when the impact of this significant shortfall in meeting housing need has never actually been tested and is clearly a greater negative in respect of the social and economic elements of sustainable development which would affect the overall outcome of the SA.

1.12 As we identified throughout our reps and in our Hearing Statements during the course of production of the Adur Local Plan the OAN has significantly increased. At no stage has an appropriate and thorough sustainability assessment of the increased OAN from 180 dpa (when the plan was first being prepared) to 291 dpa (for the Proposed Submission Version) been undertaken or appropriate consideration given to testing reasonable alternatives to deliver additional homes to meet the need.

1.13 This more recent increase from 291 to 325 dpa (when the Adur Local Plan was submitted) as noted above has never been tested. As a result the proposed spatial strategy set out in the Adur Local Plan has not been appropriately tested and is therefore not based on clear or reliable evidence which demonstrates why it is the most appropriate strategy based against reasonable alternatives.

1.14 For reasons expressed throughout our reps and summarised above, we maintain that the Adur Local Plan is not based on a sound process of sustainability appraisal and testing of reasonable alternatives and therefore does not represent the most appropriate strategy and the Adur Local Plan cannot be found sound.

1.15 Given the increase in the OAN from 180 to 325 we consider that a full reassessment of site options must be carried out to ensure that, in light of the significant housing shortfall, appropriate consideration has been given to all reasonable alternatives and that development options including those sites previously dismissed have been appropriately considered and consideration given to how constraints may be positively resolved or mitigated (in line with the requirements of the NPPF).

1.16 Until this work has been carried out in consultation with the relevant landowners, we do not agree that the sustainability appraisal is sound. The approach taken by the Council is inherently flawed and is not a sound process of sustainability appraisal, as all reasonable alternatives have not been considered and the council has not worked positively to seek to meet its housing need.

1.17 In order for the plan to be found sound it needs to be based upon a sound process of sustainability appraisal. Therefore a full and proper sustainability appraisal must be carried out in light of the increased OAN which reconsiders those sites options previously dismissed, taking account of the most up to date evidence, and working positively and proactively to seek to meet housing need and strike the most appropriate balance between the 3 elements of sustainability. Currently, this is not being achieved and the significant shortfall in meeting housing needs, when compared against the slight benefit in maintaining additional countryside and green gap designations, is not an appropriate balance in our view. We are of the view that additional sites have been demonstrated to be available, developable and deliverable and capable of allocation for additional housing without constraint. These additional sites should be allocated within the Adur Local Plan in order for it to be found sound.

MM7 and MM20

1.18 These MM's introduce some flexibility to the site allocation boundaries. In respect of MM7 this flexibility, which has potential to impact on the Green Gap and Countryside designations, has not been subject to any further SA. The Proposed Submission Adur Local Plan SA states that '*amendment 5* [the indicative built up area boundary] *cannot be assessed at this stage as it will be unclear until the planning application stage exactly where the boundary will lie and therefore no meaningful assessment can currently be undertaken*'.

1.19 There is an inconsistency in approach here, which is carried forward in the Main Modifications, as no further SA testing is undertaken, in that the built up area boundary to the proposed site allocation at New Monks Farm is considered by the Council to be something which can be appropriately determined and assessed at planning application stage. Whereas the built up area boundaries in other parts of the district, and specifically those relating to the omission sites, are fixed.

1.20 Given the potential impact that adjustments to the New Monks Farm boundary could have on the Green Gap (as identified in the Council's evidence base) versus the limited or similar impact of other sites, there is an inconsistency in approach to assessment and interpretation of evidence for allocated and omission sites.

1.21 We have maintained throughout our reps an objection to the flawed landscape evidence base and inconsistent approach through the plan preparation process which has meant that there is no clear justification why some sites have been allocated and others omitted and how the boundaries of the Green Gap have been established.

1.22 In respect of MM20 the flexibility of the site allocation boundary in this instance has been tested in the SA Addendum which concludes that the modification would not have a detrimental impact on sustainability objectives. We are of the view that given the sensitivity of this site allocation and its position within the Green Gap there is potential for a significant impact on sustainability objectives particularly in respect of landscape and countryside especially given that the policy has also been amended to refer to a minimum employment floorspace which gives rise to potential for an increase in overall floor area and hence built form at the site. This again demonstrates the inconsistency which the council have applied in interpretation of their own evidence base.

MM27 & MM28

Site Options Appraisals

1.23 New Salts Farm, as explained in previous reps, crosses two of the Site Options Appraisal areas, Site Option 6 (Land North West of Hasler Estate) and Site Option 7 (Land North East of Hasler Estate).

Site Option 6

1.24 The Site Options Appraisals in the SA Addendum states in relation to Site Option 6: *'the exclusion of this site from the Local Green Gap would have no implications for the assessment of this site as it was not considered that it made a significant contribution to the setting, character, structure and environmental quality of the countryside/district'.* It goes on to say *'the main reason for this site not being allocated in the Local Plan relates to flood risk issues. Therefore, this modification to the Local Green Gap does not change the main conclusions of the site options appraisal and the site should continue to be omitted from the Local Plan'*.

1.25 In respect of the part of New Salts Farm that lies within Site Option 6 we welcome the recognition that this part of the site does not make a significant contribution to the setting, character, structure and environmental quality of the countryside. The assessment acknowledges that the only reason for the site not being allocated relates to flood risk issues. In this respect, as set out in more detail below, and as we have submitted throughout our reps, we are of the view that the identified flood risk constraints at the site are capable of being mitigated and this is therefore not a constraint that cannot be overcome.

1.26 Given the conclusions of the SA Addendum, and the context of significant shortfall in meeting housing need, we are of the view that the part of New Salts Farm which falls within Site Option 6 must be allocated for housing development for the plan to be found sound. Otherwise there is a clear demonstration here that the Council have not made every effort to meet their housing need and the plan has not been positively prepared.

1.27 It is clear that the landscape impacts of development on the part of New Salts Farm within Site Option 6 would be less than those impacts identified as a result of the site allocation at New Monks Farm when considering the two site appraisals alongside each other. Therefore if this site is not allocated there is further demonstration of the inconsistency in approach in the sustainability appraisal and its unsoundness.

Site Option 7

1.28 The Site Option 7 Appraisal states that *'the majority of this site still remains within the Local Green Gap'*. It is a point of clarity that the whole of this site option remains in the Local Green Gap.

1.29 The appraisal goes on to say that it '*provides a valuable 'slice of green' separating the urban areas to the south from the buildings of Shoreham Airport'* and concludes that the overall conclusions of the site appraisal would not be impacted.

1.30 We are of the view that the boundary for the Green Gap should not include the part of Site Option 7 to the west of New Salts Farm Road (i.e. New Salts Farm omission site). We have demonstrated throughout our reps to date that the site is capable of being developed for housing whilst maintaining the land essential to prevent coalescence and maintain the 'valuable slice of green' separating the airport from urban areas to the south. This was demonstrated in our most recent submission which included an amended indicative site plan (copied below). We therefore disagree with the conclusion within the SA Addendum and consider that the site is capable of being allocated for development.

1.31 In addition, given the Council's approach to an indicative built up area boundary, in relation to New Monks Farm for example, there is no reason why a similar approach could not be taken in respect of New Salts Farm. This would enable the council to meet a greater proportion of its housing need whilst also protecting the countryside and Green Gap by carrying out a full assessment of impacts at planning application stage.

Revised Illustrative Masterplan New Salts Farm

1.32 The update to the Site Option Appraisal continues the inconsistent approach to sustainability appraisal which we have set out has been carried out to date by the Council. The SA and strategic site selection process has not been objective or based on appropriate criteria. It has been inconsistent and based on flawed evidence and approach and this has not been rectified in the recent SA Addendum.

Flood Risk

1.33 Flood Risk is another constraint identified in relation to New Salts Farm and one which the Council have continued to cite as a reason for omitting the site from allocation in the latest Site Option Appraisals within the Sustainability Appraisal Addendum.

1.34 However on behalf of Hyde New Homes, we have provided extensive evidence to demonstrate that the identified surface and groundwater flood risk at the site are capable of being overcome for the lifetime of the development without giving rise to flood risk elsewhere. To date no site specific contrary information has been provided to us to suggest that the proposed strategy would not work.

1.35 Hyde New Homes have sought to work with Adur, WSCC and the Environment Agency (EA) to provide requested information and respond to queries raised over the proposed flood risk strategy at New Salts Farm.

1.36 During this process the EA have confirmed that they have no objection in principle to the allocation of the site for residential use (REP-006-002).

1.37 WSCC however advised at the end of March 2017 that they did not intend to engage any further with our client until the Inspectors decision on the status of New Salts Farm was released.

1.38 In respect of those outstanding concerns of WSCC and Adur these related to Modelling and ground water levels.

1.39 In May 2017 WSCC provided some further detail to explain their conclusions in respect of groundwater levels which sought to explain their comments in the letter dated 21st March 2017 (REP-023-03) and ongoing concerns with the flood strategy proposed (copied at Appendix 1).

1.40 This data has been reviewed by Tully De'ath on behalf of Hyde New Homes and a full response is provided in the letter at Appendix 2 to this report and has been circulated to WSCC. In summary we do not consider that the results of the two boreholes provided by WSCC (which are 1.6km and 2.8km from the New Salts Farm site) accurately represent the conditions found on the site which are influenced by local ground conditions. We consider that the data collected on site is more representative of the true situation and provides an accurate assessment of groundwater levels and the fluctuations which arise based on local ground conditions, tides and rainfall. Further, and as is suggested as necessary by WSCC in their note at point 8, the hydraulic calculations carried out at the site for an extreme storm event take full account of the recorded groundwater levels and provide for further allowance for higher seasonal groundwater levels and to take account of effects of sea level rise.

1.41 In respect of modelling WSCC's letter dated 21st March 2017 suggested the modelling carried out to support the flood risk strategy at New Salts Farm was not consistent with their understanding of the Lancing Brooks network based on updated survey work across New Monks Farm. The evidence referenced was not provided to us at the time of the letter. Following an FOI request we understand that this refers to a Flood Risk Assessment (FRA) prepared in support of a planning application for the New Monks Farm site. We have now had sight of a copy of the FRA which was provided by Adur on confirmation that they had received a planning application for the site.

1.42 We would note that in March 2017 when WSCC advised us that they had sight of contrary evidence, we can reasonably assume that the FRA was only in draft form. Further the document does not form part of the evidence base for the Local Plan and therefore the reliance on this document to suggest the flood mitigation strategy at New Salts Farm site is not adequate is questionable. The FRA has we understand only just been formally submitted to Adur and it would seem pre-emptive for WSCC to make judgements on that information before it had been finalised or been in the public domain and subject to scrutiny.

1.43 Having now had an opportunity to review the FRA for New Monks Farm we would highlight the following in respect of the specific point made by WSCC that they had more recent evidence to suggest that the modelling for New Salts Farm was inconsistent with their understanding of the Lancing Brooks Network:

- New Monks Farm do not appear to have undertaken a detailed hydraulic model of the ditches as has been carried out to inform the FRA and drainage strategy for the New Salts Farm site. The model created for New Salts Farm demonstrates that the predominate flow route on the Monks Farm Ditch is south north due to the constriction caused by the culverts under the railway and the slack gradients across the New Monks Farm site. This is contrary to the New Monks Farm Assessment results.
- WSCC appear to have taken the New Monks Farm testing as correct despite this being based on a less detailed assessment to that carried out for New Salts Farm site and despite it being, to our knowledge, in draft form when such assumptions were made.
- It is unclear why such a high level approach has been taken as more accurate than the detailed hydraulic model prepared for New Salts Farm site and we would maintain that the hydraulic model is a more appropriate approach for a site such as this which is relatively flat and where water can flow both ways.
- The New Monks Farm FRA proposes to allow flows into the New Salts Farm ditch system to match the existing flows. However the method of calculating the existing flows has used a simplistic approach rather than the detailed modelling undertaken at New Salts Farm and the flows calculated by New Monks Farm appear to far exceed the greenfield run-off rates that New Salts Farm are being asked to comply with by WSCC. Nevertheless if the flows were to be restricted to match the existing as the New Monks Farm FRA proposes then the overall impact has been considered within the modelling carried out for the New Salts Farm site and would not impact on the New Salts Farm FRA and drainage strategy.

1.44 In addition, we wish to highlight the following points:

- The New Monks Farm FRA is reliant on recorded data which covers a period of 1 year in length. The disadvantage of this approach is that it is unlikely that a 1 in 100 year event would have occurred in the monitoring period. In respect of New Salts Farm WSCC have been reluctant to accept that the data collected over a period of monitoring (in excess of 1 year on parts of the site) is representative of what occurs on the site. The New Salts Farm FRA has therefore focussed on predicted risk and had regard to design standards for a 1 in 100 year event. This is a more onerous approach than that at New Monks Farm, and requires a higher design standard to be achieved.
- In general throughout discussions with WSCC and Adur over the last year we have been required to carry out more detailed assessments and provide a higher level of information than

that which appears to have been prepared for the New Monks Farm FRA. Which we presume has been through a similar level of discussions with statutory authorities given that the site has been allocated despite identified flood risk constraints. New Salts Farm has provided more detail in respect of drainage design, details and calculations, hydraulic modelling of ditches and more details of flood resilience and resistance to the units. This again highlights the continuing inconsistencies in Adur's approach to assessing site options and that they have not positively sought resolutions to identified constraints.

1.45 Adur have maintained in their latest site options appraisal that New Salts Farm is not allocated due to flood risk issues. However this assumption is based on an inconsistent appraisal of how such constraints may be overcome and on information which does not form part of the Local Plan evidence base and was, to our knowledge, in draft form. Further it has not taken account of the evidence provided in respect of New Salts Farm to date which identifies how flood risk can be mitigated at the site.

1.46 On behalf of Hyde New Homes, we have responded to all requests for information and data provided in respect of flood risk issues at New Salts Farm and have clearly demonstrated that there is a solution to flood risk constraints which would enable the site to be developed to deliver additional homes. In light of this, Adur must reassess at least the New Salts Farm omission site, to ensure that they have made every effort to meet housing need and planned positively giving consideration to how constraints identified in site options appraisal may be overcome given the significant shortfall in meeting housing need within the district.

1.47 The proposed FRA and drainage strategy was issued to BRE to carry out an independent review. This review (provided with our reps on 19th April 2017) confirmed that flood risk can be managed on the site using the approaches set out by Hyde New Homes and their consultants.

1.48 The evidence which Hyde New Homes have provided to support the development of the site has not been given any further consideration in the sustainability appraisal through the course of developing the Local Plan which further highlights the negative approach of the council. We submit that flood risk is no longer a constraint to development that cannot be mitigated and this should be reconsidered within the Site Options Appraisals.

1.49 To provide a coordinated response in respect of flood risk, picking up on all ongoing correspondence with the EA, WSCC and Adur to date, and to address the BRE Peer Review of the proposals commissioned by Hyde and submitted with our reps on the 19th April 2017, we have prepared an updated Flood Risk Assessment and Drainage Strategy which is at Appendix 3 to this report.

1.50 This evidence submitted to date, on behalf of Hyde New Homes, confirms that the surface and groundwater issues identified at the New Salts Farm site can be mitigated and that Flood Risk is not a constraint which would prevent development of the site.

Conclusion

1.51 We consider that in light of the proposed amendments to the Local Green Gap boundary and the higher OAN alongside the significant evidence provided on behalf of our client in respect of flood risk mitigation, a full reassessment of at least the New Salts Farm omission site, should be carried out in order for the plan to be based on a sound process of sustainability appraisals. Further that additional reasonable alternatives which seek to meet the identified OAN must be carried out to ensure that the Local Plan represents the most appropriate strategy.

Adur Local Plan Main Modifications

Intro

1.52 We are of the view that the Adur Local Plan remains unsound despite the Main Modifications proposed and that a number of the Main Modifications are unsound (for reasons set out below).

MM1

1.53 MM1 of the Inspectors Preliminary Findings (ID/7) states '*up-date document with regard to new end-date of 2032; household projections; OAN; and housing supply figures'.*

1.54 We are of the view that this MM is unsound as it is not positively prepared, given the Adur Local Plan does not make every effort to contribute towards its OAN, is not justified, as consideration has not been given to all reasonable alternatives to meet its OAN, and is not consistent with the NPPF specifically paras 14, 17, 47,152, 156 and 157 as it has not struck the right balance in terms of sustainable development.

1.55 The MM updates the relevant OAN and housing supply figures, however the updated OAN has not been properly tested in the Sustainability Appraisal (addressed earlier in this response).

1.56 The NPPF states that LPA's are required to positively seek opportunities to meet the development needs of their area unless adverse impacts significantly and demonstrably outweigh the benefits (para 14). Further LPA's should seek to achieve net gains across all 3 elements of sustainable development avoiding significant adverse impacts where possible and incorporate mitigation measures (para 152).

1.57 The NPPG (para 026 ref ID: 3-026-20140306) states that where there are insufficient sites to meet the OAN *'plan makers will need to revisit the assessment, for example changing the assumptions on the development potential on particular sites (including physical and policy constraints)'.*

1.58 We are sympathetic to the Council's position which makes it difficult to meet the full OAN in the district. However to date we do not agree that the Council have been positive and proactive or made every effort to meet or contribute towards housing need as far as is possible throughout the Local Plan production process. We maintain that further sites, such as New Salts Farm, are available, suitable and achievable with no outstanding constraints that cannot be overcome and should be considered deliverable and allocated to contribute towards meeting housing need in the District.

1.59 We maintain that the Plan in its current form cannot be found sound for the above reasons.

1.60 The allocation of New Salts Farm for housing development would assist in making the plan sound. Our client has provided a high level of evidence to date which demonstrates that the site is capable of delivering additional homes without constraint and with appropriate mitigation.

MM2

1.61 We welcome this MM and the Council's commitment to review or partially review the Adur Local Plan within 5 years. However we would seek further clarity in the proposed wording to reflect the fact that any review must reconsider sites within Adur as well as how any unmet need may be addressed elsewhere as part of a wider sub-regional exercise. This is necessary to ensure that the plan is sound i.e. is positively prepared and effective in meeting housing need.

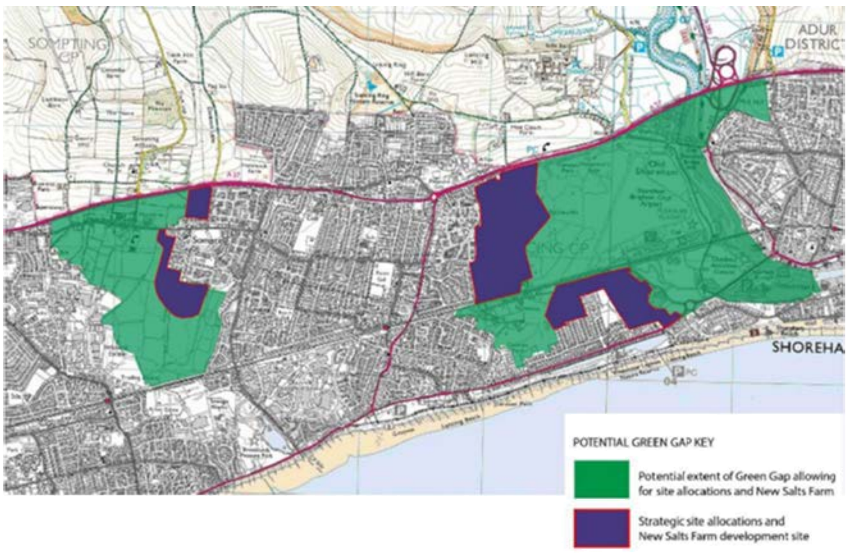
MM28

1.62 ID/7 in respect of MM28 stated *'amend green gaps boundary to include only land essential to prevent the coalescence of settlements. Amend policy wording and Policies Map accordingly. Those areas of land to be removed from the local gap designation will remain outside the settlement boundary.'*

1.63 The amendments proposed to the Policies Map and in MM28 remove 3 of the omission sites from the green gap boundary and part of the New Salts Farm omission site.

1.64 For reasons expressed in our response to the Council's homework in this regard (ALP205E) and above we remain of the view that the amendments are not based on clear, objective or justified evidence and that there is no evidence that demonstrates that scale and size of the Green Gap boundary as proposed is only land essential to prevent the coalescence of settlements. Particularly the fact that none of the Council's evidence tests the function of the Green Gap with additional development options (such as allocation of New Salts Farm); it merely identifies that there is a 'risk' that additional development will result in coalescence.

1.65 We have previously provided evidence within our reps which demonstrates that the boundary could follow the line of New Salts Farm Road and continue to maintain a green gap sufficient to prevent the coalescence of Lancing and Shoreham (see illustrative plan copied below).



1.66 The Main Modification proposed to policy 14 and amendments to the Policies Map is unsound as it is not justified (for reasons noted above and throughout our reps to date) and there is no evidence to demonstrate that the boundary proposed includes only land <u>essential</u> to prevent coalescence.

1.67 In order to make the policy sound we are of the view that the Policies Map should be adjusted to remove the whole of New Salts Farm from the Green Gap Boundary (i.e. the boundary would run north/south along New Salts Farm Road then west along the railway to adjoin the indicative boundary to New Monks Farm site allocation) as this land is not land essential to prevent coalescence of the settlements of Lancing and Shoreham.

Conclusion

1.68 We are of the view that the Main Modifications proposed are unsound, for the reasons set out in more detail above.

1.69 We would maintain that the Main Modifications do not make the plan sound. The Adur Local Plan continues to be unsound as it is not positively prepared, given it does not meet its OAN and has not engaged every effort to do so; is not justified as it has not considered all reasonable alternatives to meet its OAN and is based on a flawed and inconsistent process of sustainability appraisal; and is not consistent with the NPPF specifically paras 14, 17, 47 and 152, as it has not struck the right balance in terms of sustainable development given the significant shortfall in meeting its OAN. Further it is not consistent with paras 156 and 157 of the NPPF which require a Local Plan to include strategic policies to deliver the homes needed in the area and plan positively for the development required in its area.

1.70 The Main Modifications and Sustainability Appraisal Addendum have provided no new evidence which give us reason to change our view.

1.71 We submit that there are additional sites within the district which are available, suitable and achievable with no outstanding constraints and should be allocated in the Local Plan to contribute towards meeting housing need in the district. In respect of New Salts Farm we have provided substantial evidence to date that flood risk constraints are capable of mitigation and that the development of the site would not give rise to significant landscape impact and would not result in coalescence. Further to the evidence provided to date, further detailed evidence in respect of both would be prepared and submitted to support any future planning application and would enable the LPA to consider proposals against policy requirements to ensure that any development is delivered appropriately and in accordance with the policies in the Local Plan. We submit that in order for the plan to be found sound an additional policy should be included which allocated the land at New Salts Farm for residential development. We have copied in the box below the proposed wording below which was set out in response to the Council's Homework.

Proposed Policy Wording

Land at New Salts Farm (within the area shown on Map [ ]) will be allocated for residential development comprising:

300 – 450 homes

30% of all homes should be affordable, providing a mix of types and tenures in accordance with identified needs.

Suitable access onto Brighton Road in agreement with WSCC

Provision or funding of mitigation for off-site traffic impacts on the Strategic Road Network and local roads where required.

Provision or funding of sustainable transport infrastructure including public transport and cycle, pedestrian and equestrian links to Lancing, Shoreham-by-Sea and the South Downs National Park where required.

Site-specific travel behaviour initiatives which encourage sustainable modes of transport. (This should include a package of travel behaviour initiatives such as residential travel plans).

Developers will need to work with Adur District Council, West Sussex County Council and the Environment Agency to ensure that tidal and fluvial flooding as well as surface water and groundwater flooding are adequately mitigated without worsening flood risk elsewhere. A Flood Risk Assessment (FRA), which will include a site wide drainage strategy, will be required at the planning application stage. The FRA must take account of and seek to facilitate relevant recommendations of the Lancing Surface Water Management Plan.

A site wide drainage management plan should be produced and implemented to the satisfaction of the local planning authority and WSCC to ensure the long term maintenance of the drainage strategy for the site.

A site wide estate management plan should be produced and implemented to the satisfaction of the local planning authority to ensure the long term maintenance and management of estate roads, pavements, and open space.

No development shall take place within those parts of the site currently designated as Flood Zone 3b (functional floodplain) until the relevant section of the Shoreham Adur Tidal Walls has been completed.

As part of a Landscape Strategy / Green Infrastructure Strategy for the site, the following are to be delivered:

- Ecological enhancements to address safeguarding and enhancement of biodiversity assets
- Retention and enhancement of the existing network of ditches on site for drainage and ecological benefits.
- Openspace and recreation areas (to include children's play areas) located within the development in accordance with Council standards
- Strategically sited areas of landscaping and green space to provide a distinctive green edge to the east and north forming a buffer between the development and New Salts Farm Road and the development and the railway line.
- Bioretenion areas to provide landscaping, open space and recreation areas as well as accommodating surface water in extreme flood events. These should be strategically sited to correspond with the above green edge.
- A site wide landscape and ecological management plan to be produced and implemented to the satisfaction of thelocal planning authority to ensure the long-term

maintenance of retained and newly created habitats on-site.

Development of this site, the location and layout of built development, green infrastructure, landscaping and bioretention areas is to be based on the following principles and site-specific requirements:

- Development must respect the landscape of the surrounding countryside and South Downs National Park.
- Affordable housing is to be distributed throughout the whole development site
- The development is to be connected to sewerage and water distribution networks at the nearest points of adequate capacity, as agreed with Southern Water
- Built development should be directed towards the west of the site and landscaping / bio retention areas tot eh east and north to ensure an appropriate landscape buffer to New Salts Farm Road and the

Infrastructure requirements are to be secured through CIL/s106/planning conditions as appropriate

Appendix 1 – WSCC Groundwater Levels

Explanation of evidence to suggest the need to review the New Salts Farm Groundwater Monitoring Data in the context of wider Time-Series Data

1. In its letter entitled: Pre-Application Query: Tully De'Ath Flood Risk Assessment for The Hyde Group; Further Evidence Review (March 2017) and dated 26 March 2017, the LLFA included the following statement:

   The LLFA does not hold data for New Salts Farm for 2012 / 2013; however, there is evidence to suggest that groundwater levels (GWLs) at New Salts Farm would have been significantly higher than has been recorded for the site during 2015-16.

2. Subsequently, following a request from Tully De'ath, the LLFA agreed to provide an explanation of the evidence referred to in this statement;

3. Time series data for groundwater levels has been plotted for two EA boreholes in close proximity to New Salts farm, namely Sussex Pad, 1.6km away and Crosshaw Recreation Ground in South Lancing, 2.8km distant (see Figure 1).

   Figure 1 Locations of EA Boreholes at Crosshaw Recreation Ground and Sussex Pad.

4. It is stressed that these data are not intended to be proxy data for New Salts Farm; rather they show a relationship between groundwater levels and rainfall that, it is suggested, allow some basic conclusions to be made with regard to groundwater levels (in the context of previously provided photographic evidence) at New Salts Farm.

5. Figure 2 shows monthly rainfall data for Applesham Farm overlaid with EA averaged groundwater monitoring data for Sussex Pad; Figure 3 shows averaged monthly groundwater level data for Crosshaw.

6. Three observations are made in relation to these data:

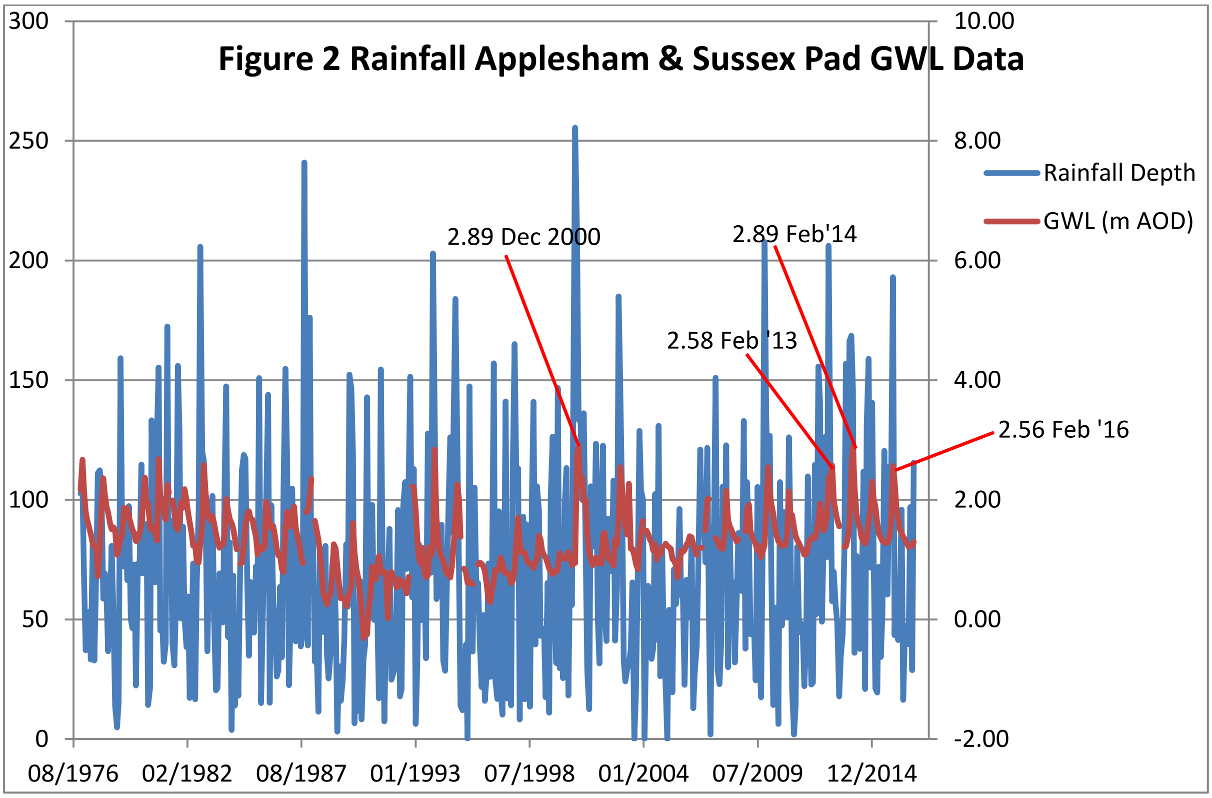
- Peaks in groundwater levels broadly coincide with or follow peaks in monthly rainfall or sustained high rainfall over a period of weeks; Thus record high monthly rainfall in October 2000 of 266mm (sandwiched between high rainfall during September and November) resulted shortly after in a peak in groundwater level of 2.89m AOD at Sussex Pad in Dec 2000. The same groundwater level was recorded in Feb 2014 following heavy rainfall during October (157mm) December (166mm) January (163mm) and February 146mm). Figure 2 also shows that low groundwater levels correspond to reduced monthly rainfall. Similarly the peak groundwater level of 5.47m AOD was recorded at Crosshaw in February 2000 following the very wet autumn. More recently, the wet autumn / winter of 2013/14 resulted in a peak in average groundwater levels in February 2014 (Figure 3).

- From their relative high level in January 2016, groundwater levels at both sites were in decline during the remainder of the year with the exception of June at Crosshaw which saw a slight increase in levels (in response to higher than average rainfall for the month).

- The correspondence between the peaks in groundwater levels at Crosshaw and Sussex Pad suggest that groundwater levels over the wider area are likely to show similar relative peaks, troughs, rises and falls for the same time period, albeit the absolute groundwater levels may be very different and local idiosyncrasies will apply

7. From the above observations, it is reasonable to suppose that groundwater levels recorded as part of the groundwater monitoring study in support of the New Salts Farm development that were taken between November 2015 and February 2016 were not at their seasonal maximum. It is suggested that levels would have been significantly higher during the wet autumn / winter of 2013/14. This is further corroborated by photographic evidence (plate 1) that shows extensive surface water flooding in March 2014. This notwithstanding, the peak groundwater level for WLS5 was recorded on 8 Feb '16 at 0.352m above ground level (1.62m AOD). The monitoring report also states that the groundwater level at WLS 5 consistently rose above ground level at high tide indicating the groundwater level is periodically artesian. Given that WLS5 is located 1.27m AOD, this is indicative of the drainage challenges for the eastern portion of the site. On 10 Feb '16, records from WLS 2 show groundwater level being recorded at 0.15m below ground level (estimated to be 1.6m AOD from LiDAR also suggest seasonal groundwater levels can exceed 1.6m).

8. The current drainage proposals for New Salts Farm are based upon monitored ground water levels during 2015/16 that the above evidence suggests are below their recent seasonal maxima. The recorded groundwater level of 1.62m AOD at WLS5, approximately 250m from the centre of the proposed detention basin, means that there is ~0.27m depth loss in the storage for the basin (based upon a basin level of 1.35m AOD) and the hydraulic calculations for the extreme storm event would need to take this into consideration. It is suggested that further allowance needs to be made for yet higher seasonal groundwater levels to avoid risk of design-failure.



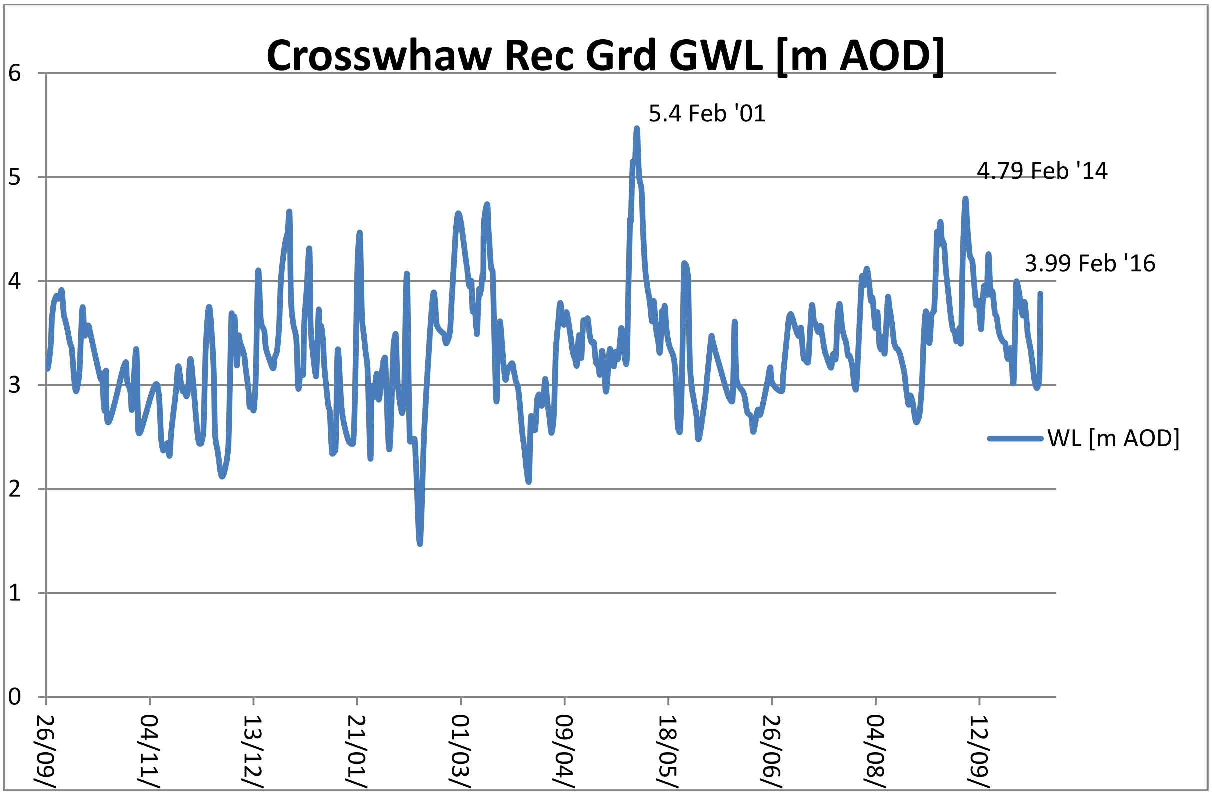


Plate 1 Aerial photo taken Mar 2015 showing western portion of New Salts Farm Site.

9. The LLFA is grappling with understanding better the relationship between groundwater levels and surface water flooding, particularly in the coastal plain. To this end, a groundwater study has been commissioned study to address this knowledge gap. The long term aim is to, where necessary, make applications to the RFCC for Local Levy funding for capital works to characterise and where appropriate, alleviate groundwater flooding issues.

10. The study will focus on selected groundwater flood risk monitoring pilot test locations characterisation of the local geological and hydrogeological setting and assessment of the availability, suitability and integrity of existing hydrometric data to characterise the hydrogeological regime(s). At least one of these locations will be on the coastal flood plain and the relationship between tidal influence and groundwater levels will be further investigated and consideration given to the long-term implications of sea-level rise.

Ray Drabble
Drainage Engineer
West Sussex County Council Lead Local Flood Authority

Appendix 2 – Response to WSCC Groundwater Levels

Ray Drabble
West Sussex County Council
Western Area Office
Drayton Lane
Chichester
West Sussex
PO20 2AJ

AJP/rw/11649
25th July 2017

Dear Ray

Re: New Salts Farm Road Land – Shoreham
In response to Ray Drabble letter issued on 26th May 2017

The WSCC letter provides ground water monitoring results for two boreholes located approximately 2.8 km and 1.6 km away from the development site. WSCC make the point that this data is not intended to be proxy data for New Salts Farm rather to show the relationship of ground water levels and rainfall events.

Our response to the items raised within the WSCC letter are based upon onsite borehole data which commenced in November 2015.

It is acknowledged that the two borehole locations provided indicate that there is a direct relationship between ground water levels and rainfall events, which is not uncommon. The ground water levels for the two locations provided show the long-term variations spanning across 40 years, which typically fluctuate by 1m- 2m over a year. What is not clear is whether these boreholes are influenced by the tides in a similar fashion to the NSF site, where water levels vary between 1-2m with the tides.

WSCC make specific observations within the EA boreholes regarding the ground water levels during 2016, which were at relatively high level in January and then continued to fall throughout the year. However, based upon the data set we have obtained on site the high tide ground water levels have remained relatively constant throughout 2016.

WSCC suggest that the ground water levels at the start of the New Salts Farm monitoring period were not at the seasonal maximum. The ground water monitoring was started in November 2015 where the rainfall in the Sussex area in 2015 was recorded as being above average. In addition, the rainfall in January 2016 was also above normal levels. Consequently, we would expect the ground water levels to be relatively high at the start of the monitoring period. In reviewing the data for the boreholes which have been installed for 19 months, high tide water levels have remained relatively constant, with only short term fluctuations, usually associated with spring tides.

Within the Lancing SWMP (the chapter that discusses flooding on the West Beach Estate), it is noted that due to the close proximity of the sea, variations within the ground water levels will reduce and there will be a lesser variation in the maximum and minimum levels. Within the SWMP it also states that '*ground water levels within the superficial despots are likely to remain high for most of the year, although there will be some recession through the autumn months'.* This statement backs up the findings of our own monitoring which indicates that the high and low ground water levels do remain relatively constant.

Continued 1/…

Ray Drabble
West Sussex County Council

AJP/rw/11649
25th July 2017

Continued …/2

As we have previously advised WSCC, and as accepted by Adur in their document ALP029 'ADC Comments on Homework Responses 28th April 2017,' the photographic evidence of surface water flooding is misleading as it does not cover the New Salts Farm development site. The photo appears to cover the Old Salts Farm, which is specifically noted within the SWMP as an area which is '*associated with widespread waterlogged ground*'. New Salts Farm is not mentioned as suffering from this form of flooding, which has been confirmed by the farmer who has been managing the farm since 2000.

The data collected from the monitoring wells demonstrates that as you move northwards across the site the tidal influence reduces. Adjacent to the southern boundary (WLS6) the change in level fluctuation is in the order of 2.5m whereas the northern sections (WLS108 & WLS109) of the site the level fluctuate between 1.0m and 1.3m. As well as the distance from the coast, other factors such as lower permeable geology will also affect the tidal influence, as can be seen in WLS108 (location of bioretention basin) where ground water levels have not risen above 1.0m AOD.

The ground water monitoring results clearly demonstrates that the depth of ground water levels do vary across the site. The location of the bioretention basin will be located where the high ground water levels are typically at 0.7m AOD, and have not risen above 1.0m AOD. The proposed base level of the bioretention basin is 1.1m AOD and (following confirmation that soakage is possible) will not be lined to allow for infiltration.

In the event that ground water levels rise above the base level of the detention basin an overflow (set at a level of 1.2m) will direct ground water flows into the adjacent watercourse discharging at greenfield run-off rates. Refer top drawing 11649-CIV-120B.

Surface water calculations have demonstrated the allowing for full infiltration (normal condition) water levels within the basin would reach a level of 1.204m AOD during a 1 in 100+CC event. Assuming no infiltration (worst case) the method of surface water disposal would be via the overflow, which would restrict the discharge to greenfield run-off rates. In this scenario, the water level in the basin would rise to 1.359m AOD. The top of the bank of the basin is proposed at 1.6m AOD with the adjacent roads set at a level of 1.8m AOD, consequently there is additional freeboard of storage built into design of the basin to take account of exceedance events, unusually high ground water levels and the effects of sea level rise associated with climate change for the life time of the buildings.

To provide an independent check on our flood and drainage strategy the BRE were consulted to review our proposals. It was the BRE's opinion that the flood risks on the site can be managed using the methods outline in our proposals.

In conclusion, the local ground conditions on New Salts farm appear to influence the ground water levels by smoothing out the fluctuations caused by rainfall events. Consequently, the results of the two EA boreholes do not represent the conditions found on the proposed development site.

To assist in collating all the information which has been developed over the last six months, our Flood Risk Assessment has been updated (Issue 5) to include the latest ground water monitoring results, updated drainage proposals and correspondence between all the consultees.

Yours Sincerely
for Tully De'Ath

[redacted]

Andrew Picton

Appendix 3 – New Salts Farm FRA

Tully De'Ath consultants

Feasibility Research

EIA, Flood Risk & Transport Assessments

Urban Planning and Design

Integrated Transport Solutions

Infrastructure Development

Structural Design

Eco and MMC Focused

Flood Risk Assessment

Issue 5

New Salts Farm - Shoreham

11649

For The Hyde Group

Engineering at its Best

Report For

Scheme No: 11649

The Hyde Group

New Salts Farm Road Land - Shoreham

Flood Risk Assessment Issue 5

Main Contributors

25th July 2017

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↘ Appendix G – SFRA Historical Flood Maps

↘ Appendix H – Lancing SWMP Extracts

↘ Appendix I – EA Flood Maps

↘ Appendix J – Indicative Drainage Drawing and Calculations

↘ Appendix K– SuDS Manual Extracts

↘ Appendix L – Adur Tidal Wall Scheme

↘ Appendix M – West Beach Estate Drainage

↘ Appendix N – Foundation and Ground Floor Construction

↘ Appendix O – Road Improvement Systems

↘ Appendix P – Green/Blue Roof Details

↘ Appendix Q – Lancing Brooks Modelling

↘ Appendix R – Correspondence from The EA, LLFA, Adur and BRE

1.0 Introduction

1.1 Tully De'Ath have been commissioned by The Hyde Group to prepare a Flood Risk Assessment (FRA) in support of the redevelopment of New Salts Farm for residential development in Shoreham.

To enable the whole of New Salts Farm to be considered for inclusion within the Local Development Plan this report will review the flood risks across the development area and will demonstrate how these risks can be mitigated.

It is anticipated that the site will be developed over a number of phases and as each future phase is constructed, a phase specific FRA will be provided to accompany each planning application.

1.2 The purpose of the report is to demonstrate to the Planners, the Environment Agency (EA) and West Sussex County Council as Lead Local Flood Authority that the proposed development is subject to an acceptable level of flood risk and should not increase the likelihood of flooding elsewhere.

The report has been prepared in accordance with the National Planning Policy Framework (NPPF) and the 2014 Planning Practice Guidance – Flood Risk & Costal Change, which has recently superseded the Technical Guidance document to the NPPF.

The surface water drainage principles will follow the guidance, Water.People.Places. - prepared by the Lead Local Flood Authorities (LLFA) of the South of England.

The FRA will make reference to the Adur and Worthing Strategic Flood Risk Assessment (SFRA) and the Lancing Surface Water Management Plan (SWMP).

Issue 5 of this report has been updated to include:

- Additional ground water monitoring data and infiltration test results
- Updated site development proposals with additional drainage details and calculations
- Updated hydraulic modelling reports of the Lancing Brooks drainage system
- Correspondence with the LLFA, Adur, the EA and the Buildings Research Establishment (BRE)

2.0 Site Location

2.1 New Salts Farm is located to the west of Shoreham and covers an area of 28.2Ha. The Farm is bounded by the railway line to the north, New Salts Farm Road to the east and West Beach Estate to the south. The south eastern corner of the site fronts onto the A259, Brighton Road. Shoreham Airport is located to the north and the River Adur to the East.

2.2 Refer to Appendix A for a location plan.

3.0 Existing Conditions

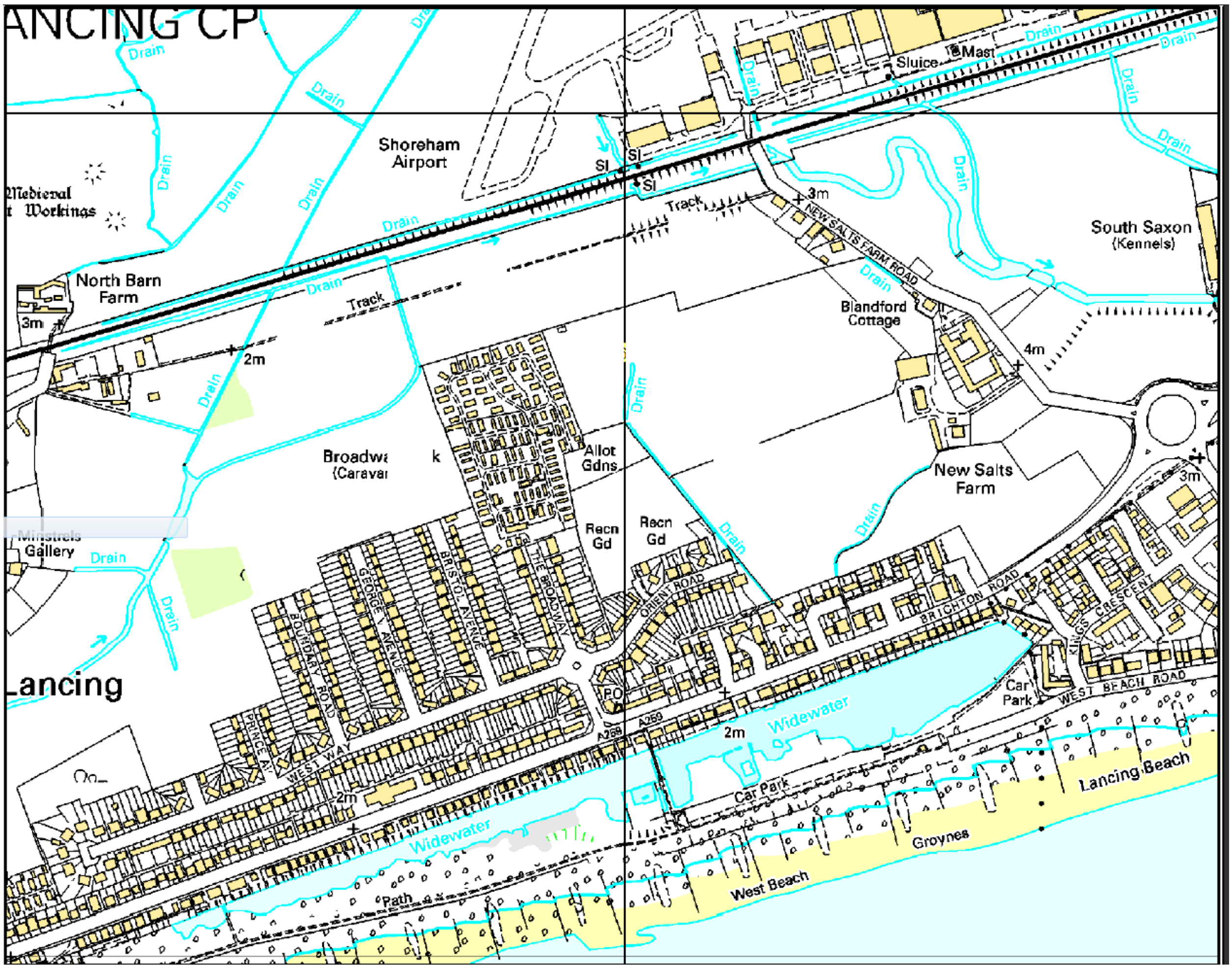
3.1 The Farm comprises agricultural fields which are currently used for grazing.

3.2 Topography

A topographical survey (Appendix B) shows the site levels across the Farm are relatively flat, typically ranging from 1.8m AOD along the southern boundary to 1.4m AOD along the northern boundary. The lowest area of the site is located between the Broadway Park site and the railway where levels are typically 1.4m to 1.1m AOD.

3.3 Water Features

There are a number of drainage ditches across the development area which form part of the Lancing Brooks drainage system. These ditches drain to the northern boundary, adjacent to the railway line, before ultimately discharging into the River Adur to the east. A detailed assessment of the Lancing Brooks has been undertaken by JBA and is included within Appendix Q. There are two drains highlighted on the survey plans on the eastern side of the site which appear not to drain to any specific outfall but do collect water during wet periods.



Lancing Brooks Drainage System

These ditches are linked to sluice gates (the Lancing Brooks Outfall) on the eastern side of New Salts Farm Road which stop tidal flows from the Adur flowing back into the ditches. Consequently, during high tides the ditches hold water until the levels in the Adur drop.

From the eastern boundary of the site the sluice gates of the River Adur are approximately 200m to the east, with the main river channel of the Adur a further 300m beyond.

Widewater Lagoon is a manmade feature which lies to the south of Brighton Road, approximately 100m from the south western boundary. It is a landlocked brackish Lagoon (approximately 1.2 km long and 50m wide) bordered on its south side by the sea defenses and shingle beach. Water levels rise following high tides and significant rainfall events.

The coast is located 250m to the south of the most southerly section of the site.

3.4 Sewerage System

Southern Water sewer records (Appendix C) indicate that there is an existing adopted 200 dia. foul sewer which runs across the eastern side of the site which appears to drain the buildings on New Salts Farm Road. A rising main crosses part of the site adjacent to the south eastern boundary linking the foul drainage from Wenceling Cottages to the adopted sewer in Orient Road.

The sewer records indicate that there are no adopted surface water sewers on the Farm

3.5 Geology

Geological maps indicate that the natural site geology consists of Alluvium/Marine Deposits over Newhaven Chalk.

Intrusive testing has established that beneath a thin layer of top soil a depth (0.35m-1.85m) of sandy clay overlies sand (0.9m-1.65m) which in turn overlies gravels which was proven to a thickness in excess of 3.5m.

A number of soakage tests were carried out (Appendix D) which indicated that for shallow infiltration devices (such as permeable paving) a mean design value of 0.4 litres/m²/min could be used with an appropriate factor of safety applied.

3.6 Ground Water

Ground water was struck during the fieldwork between a depth of 0.7m and 1.7m below ground level (bgl). Ground water monitoring wells and dataloggers were installed which established that the ground water levels on the eastern part of the farm are significantly influenced by the tide, although there appears to be a 1.5 – 2 hour time lag between high tide and high water level. Ground water monitoring results are included within Appendix D.

In the south eastern corner of the site the ground water levels were recorded from December 2015 to June 2017, (WLS4, 5 & 6) although WLS4 stopped working for one month and WLS6 stopped for three months due to corrosion.

Elsewhere on the site additional ground water monitoring wells (WLS107,108, 109, 110, & 111) were specifically installed in areas which were anecdotally noted to be wet. Ground water levels for these wells were recorded between September 2016 and June 2017.

Locations of Boreholes, Ground Water Data Loggers & Ditch Monitoring Points

Within the south eastern corner ground water levels were generally recorded at 0.4m - 1.0m bgl during high tide events, which dropped to 1.7m - 2.4m bgl at low tide. However, one of the dataloggers located adjacent to the southern boundary has consistently recorded above ground levels which suggests that groundwater is periodically artesian. Based upon these findings, this part of the site should periodically flood, however this has not been witnessed, neither are we are aware that this area routinely floods. This could be as a result of localised impermeable clay layer which prevents ground water rising to the surface. It should be noted that WLS5 is located in a low area adjacent to an existing ditch.

On two occasions over the monitoring period ground water was recorded reaching ground level in WLS6, which upon reviewing the tidal charts, was related to a particularly high spring tide event.

The recorded ground water levels in the additional boreholes all remained below ground level for the duration of the monitoring period with the exception of one event in WLS109 which corresponded to a high spring tide event. The water levels in WLS109 generally ranged between 0.3m and 0.9m bgl, however it should be noted that WLS109 is located in a localised low area. WLS108 levels remained at depth with the highest level recorded at 0.9m bgl. The levels in WLS107 rose to within 0.55m bgl during high tide events. The levels within WLS110 & 111 generally remained at depth (1.0m bgl) with little fluctuations although there have been four recorded peaks where levels rose to within 100mm of ground level which appears to be associated with particularly high spring tides.

The data collected demonstrates that as you move northwards across the site the tidal influence reduces. Adjacent to the southern boundary (WLS6) the change in level fluctuation is in the order of 2.5m whereas the northern sections (WLS108 & WLS109) of the site the levels fluctuate between 1.0m and 1.3m. As well as the distance from the sea, other factors such as lower permeable geology may also affect the tidal influence, as can be seen in WLS108 where ground water levels have not risen above 1.0m AOD

The LLFA have suggested that the ground water levels at the start of the monitoring period were not at the seasonal maximum. The ground water monitoring was started in November 2015 where the rainfall in the Sussex area in 2015 was recorded as being above average. In addition, the rainfall in January 2016 was also above normal levels. Consequently, it would be expected that the ground water levels at the start of the monitoring period to be relatively high. In reviewing the data for the borehole which have been installed for 19 months, high tide water levels have remained relatively constant, with only short term fluctuations, usually associated with spring tides.

Within the Lancing SWMP it is noted that due to the close proximity of the sea, variations within the ground water levels will reduce and there will be a lesser variation in the maximum and minimum levels. Within the SWMP it also states that *'ground water levels within the superficial deposits are likely to remain high for most of the year, although there will be some recession through the autumn months'*. This statement backs up the findings of the monitoring which indicates that the high and low ground water levels do remain relatively constant.

Based upon the above, high ground water levels on the site appear to be more aligned to high tides rather than rainfall.

Within the SWMP the relationship between the Lancing Brook Ditches and the ground water is also discussed. It states that due to the characteristics of the superficial deposits the high ground water levels may provide some base flows to the surface water ditches. However, it "*is likely to be only a relatively small contribution to the overall flow in the surface water channels."*

The EA's Ground Water Vulnerability map shows the site is not within a source protection zone but overlays a Minor Aquifer High Vulnerability.

3.7 Water Level Monitoring within the Ditches

Data loggers were installed to three of the main ditches on the site to establish water levels. The gauge within Ditch 2 disappeared. However, the two that remained were located on the road bridge over the stream on New Salts Farm Road (Ditch 1) and the ditch (Ditch 3) running west/east between the railway line and the West Beach Estate. Eleven months of data was collected between July 2016 and June 2017 the results of which are provided in Appendix D.

Over the period of monitoring the water levels do rise and fall, typically by 100mm although the rate of change is not particularly fast.

The water level rise in both ditches closely relate to each other, although the fluctuations within Ditch 1 are more pronounced compared to Ditch 3. This is probably due to Ditch 1 being closer to the Lancing Brooks control valves and the ditch itself being a more clearly defined channel.

Within Ditch 1 levels typically range between 0.45m AOD to 0.6m AOD with a recorded high level of 0.9m AOD and low level of 0.35m AOD.

Within Ditch 3 levels typically range between 0.40m AOD to 0.65m AOD with a recorded high level of 1.1m AOD and low level of 0.4m AOD.

Over the monitoring period all recorded water levels remained in-channel.

Whilst there is some correlation in high water levels within the ditches and high tide events the ditches (where the outfall gates are restricted for a longer period) they are by their very nature also influenced by surface water run-off associated with rainfall.

4.0 Development Proposals

4.1 The New Salts Farm development is providing circa 300 new homes, together with associated car parking open space and landscaping. Refer to Appendix E for details of an indicative site layout.

It is intended that all the units will have flat roofs incorporating green roofs with integral attenuation below (Blue Roof). Permeable paving will be provided to all roads, parking courts and hard paved areas.

Bioretention areas will be incorporated into the design which will match the existing low points on the site where surface water flooding would naturally occur in extreme events.

A new drainage channel will be constructed within the development site which will run parallel to the main channel within the Network Rail land. This channel will be designed to take additional/exceedance flows from the existing channel. Alternatively, to improve the limited access to the ditch within the Network Rail land it may be preferable to divert the main channel flows into the new channel. This option will be agreed with the EA and WSCC during the detailed design stage.

Vehicle access to the majority of the site will be via a new access road from the Brighton Road, although limited vehicle access may be provided off the existing roads within the West Beach Estate.

4.2 Sequential and Exception Tests

The Hyde Group are looking to promote New Salts Farm through the Local Plan process for a development allocation of circa 300 units. It is recognised that New Salts Farm is located within a flood risk area and if the site is to be identified on the Development Plan then a Sequential and Exception test will be required.

The purpose of the Sequential Test is to demonstrate that there are no sequentially preferable available sites at a lower flood risk within a defined search area which could deliver the proposed development.

On the basis that a Sequential Test has been passed, the site could be considered suitable for residential development where the Exception Test is also passed.

For the Exception Test to be passed it must be demonstrated that the development provides wider sustainability benefits to the community that outweigh flood risk, and a site specific Flood Risk Assessment must demonstrate that the development will be safe for its lifetime.

Within Appendix F Boyer Planning have undertaken a Sequential and Exception Test which demonstrates that both tests are passed and therefore the development site can be considered for residential development to deliver much needed new homes in the Local Plan area to meet housing need.

5.0 Flooding Assessment

5.1 Historic Flood Data

With reference to the SFRA Historic Flood Maps (Appendix G) there are areas of surface water flooding indicated to the western side of the development site which appear to follow the lines of the existing ditches/drains. The West Beach Estate and the area adjacent to the south western corner of the development site is noted as having tidal flooding incidents. The nearest recorded sewer flooding incident within the SFRA has been recorded in West Way, to the south of the site.

The Lancing SWMP (Appendix H) provides data of historic flooding over the winter of 2013/14 which was the wettest winter on record. During this period regular flooding of the highway occurred on the Broadway, West Way and Prince Avenue.

West Beach Estate suffers from regular surface water and ground water flooding. As part of the wider SWMP, a separate drainage report was produced by CH2M HILL to review the existing drainage issues within the West Beach Estate. The findings indicated that much of the flooding issues on the estate were associated with poor maintenance of the existing drainage system. However, there were ground water flooding incidents associated with high tide events. In addition, it has been established that a number of the estate roads drainage systems were not connected into the adjacent Lancing Brook ditches and as a consequence had no formal outfall.

A number of reports (undertaken in 1994, 2010 and as part of the SWMP) have also reviewed the condition and effectiveness of the Lancing Brook ditches, which form an integral part of the surface water drainage system within Lancing. The reports found that the ditches were poorly maintained which severely reduced their effectiveness (Appendix H).

During June 2012 there was significant flooding across the West Sussex area, however the rain gauge data (Appendix G) demonstrates that the Lancing area avoided the worst of the rainfall and as a consequence did not suffer from any significant flooding.

It is understood that during the winter of 2012 over pumping of the Lancing ditches was undertaken to try and reduce the water levels within the ditches during tide locked events. It is not known what return event caused the flooding and the event is not specifically mentioned within the SWMP.

Over recent years the ditches have been cleaned out, an obstruction has been removed near the outfall and the Lancing Brook Outfall was redesigned to increase its capacity. Consequently, the effectiveness of the ditches has significantly improved. Anecdotal evidence form local residents also suggests that the drainage ditches are working more effectively.

5.2 Flood Maps and Modelling

Within the SFRA, the New Salts Farm site has been assessed as one of the Core Strategy Sites, however it is referred to as 'Land North East of the Hasler Estate'. These details are included within Appendix G. Additional flood maps from the EA were also obtained and are appended within Appendix I.

To summarise the maps:

- Fluvial Flood Risk – Zone 3a for the whole of the New Salts Farm site with a residual risk associated with a breach of the River Adur west bank defences.
- Tidal Flood Risk – Generally Zone 3a although there are areas (39% of the site - to the north and west) which lies within a Zone 3b. There is a residual risk of breaching of the defenses along the River Adur and wave overtopping along the costal frontage.
- Ground Water Flooding – susceptible to ground water emergence is more than 75%. The geological data indicates that there are 'windows' of ground water emergence on the site.
- Surface Water Flood Risk - Low, as the limited areas highlighted as being susceptible to flooding can be aligned to the existing drains/ditches on the site.

- Sewer Flooding – Low, with no reported incident of sewer flooding within the site although there is recorded flooding issues in West Way, to the south east of the site.

The EA have provided flood model data for the New Salts Farm site for a series of storm events which include a 1:75, 1:200, 1:200 plus climate change (CC) and a 1:1000 event for both defended and undefended scenarios (Appendix I). The 1:200 event with an allowance for climate change was the more onerous value with a maximum flood height of 5.391m AOD for the undefended event and 5.050m AOD defended scenario.

The condition of the existing defences has been assessed as "relatively good" (i.e. not poor), consequently the defended 1 in 200 +CC flood level will be used when assessing flood levels.

5.3 Lancing Brooks Modelling

Following a request from the EA, the Lancing Books drainage system adjacent to the site have been modelled by JBA Consulting to assess what impact the new climate change values would have on the development.

In February 2016 new climate change guidance was published by the EA to support the assessment of flood risk in line with the NPPF which sets out how the planning system should help minimise vulnerability and provide resilience to the impacts of climate change. The climate change allowances are predictions of anticipated change for peak river flow by river basin district and peak rainfall intensity.

They are based on climate change projections and different scenarios of carbon dioxide (CO2) emissions to the atmosphere. There are different allowances for different epochs or periods of time over the next century.

In accordance with Table 2 of the updated climate change guidance, a 20% and 40% uplift has been applied to the modelling of the Lancing Brook system to account for the "Central" and "Upper End" anticipated changes in extreme rainfall intensity in small catchments for the 2080s epoch (2070 to 2115).

It had been anticipated that the new modelling would update the 1D model previously undertaken by CH2M Hill. However, in reviewing the details it was found that the original model was steady state rather than hydrodynamic and the flow estimates provided lacked proper justification of the return period assigned to them. This required new hydrology estimates to be generated. Due to the very flat relief it is difficult to identify with confidence topographic watersheds between catchments to apply traditional flow estimations methods therefore the model was converted into InfoWorks ICM and direct rainfall was applied to the wide area to provide in channel flows.

The ICM model also allowed for the representation of the urban drainage infrastructure in the neighbouring roads to be included into the model where relevant and will allow for future outline surface water drainage details to be tested within the wider drainage area.

Much of the catchment to the north of the site was not included as part of the analysis as it would not contribute to the flooding on the site.

The updated model has allowed for the impact of an 18-hour tide lock situation at the outfall of the Adur and the restriction of the New Salts Farm Road bridge. The new drainage channel adjacent to the northern boundary of the site was also included as part of the analysis.

Following a review of the initial JBA report the LLFA raised a number of questions regarding the catchment data used within the modelling (refer to Appendix R for copies of correspondence). To this end additional modelling has been undertaken to sensitivity test the assumptions on catchment and drainage paths. The report has been updated to provide more evidence for the approach taken.

To test the assumption of contributing area for the direct rainfall approach a larger extent model was developed. The Design domain was increased in area from $6.7km^2$ to approximately $11.6km^2$. Both the design model domain and the sensitivity model domain were run for the 1:100 AEP event.

The sensitivity test was undertaken to identify flow paths into the contributing area that may not have been accounted for within the original design domain. Two of these flow paths have been identified to drain outside of the previous domain.

Two other flow paths have been identified to drain to inside the previous domain. Of these, the southern flow path flowing Monks Avenue, has potential to contribute a small volume of additional flow to the top of a brook in this location. This brook drains to the south and then passes east across the study site. Given the small area concerned and the noted presence of urban drainage infrastructure in this area connected to soakaways (Chapter 5 of SWMP) it is considered that the additional volume of flow this area may contribute to flooding at the site is negligible.

The second flow path runs parallel to the south of the A27 carriageway. This flow path runs east and crosses Marsh Barn Lane where it would be picked up by the network of brooks to the west of Shoreham Airport. These brooks continue to drain east and outfall to the estuary. They do not flow though the study site, therefore their omission from the model domain is considered immaterial.

The results of the analysis are included within Appendix Q, which demonstrate that both the 1 in 100+40% CC and the 1 in 1000-year return period remains in channel and do not flood the site.

6.0 Flood Management and Mitigation

6.1 To reduce the flood risk, a number of mitigation measures are proposed.

6.2 Unit Types

All units will provide accommodation at first floor level only where the ground floors will consist of parking and utility spaces which will be formed in flood resilient materials. The ground floor areas will be allowed to flood in extreme coastal flooding events.

The units are to be constructed using flood resilient materials and will be structurally designed to withstand the potentially significant flood depths. The ground floors will incorporate robust materials so that the units can be easily reinstated to a habitable standard. Refer to Appendix E for details of the units.

6.3 Floor Levels

The first floor level will be set at a level 300mm above the 1 in 200+CC tidal event. This equates to a minimum floor level of 5.42m, AOD, which is in the order of 3.0m to 3.5m above existing ground level.

The preference is to set the ground floor levels 300mm above the existing ground level, with variations to suit localised ground conditions or development constraints. This is to mitigate against the risk of ground and surface water flooding. The surrounding ground levels will be designed to divert flood waters away from the buildings.

The ground floors to the units will consist of parking and utility spaces which will be formed in flood resistant materials.

6.4 Foundations and Ground Floor Details

It is proposed to use the Abbey Pynford 'Housedeck' system (or similar) which is specifically designed for use in poor ground conditions. It is a mini piling system which uses continuous flight auger (CFA) piles and a reinforced concrete ground floor slab.

The system uses a continuous hollow stemmed auger which is bored into the ground to the full design depth of the pile. When at the required foundation depth concrete is pumped under pressure down the auger stem to discharge at the base of the auger. The positive pressure of the concrete and the steady rotation of the auger on its withdrawal cycle forces the entire stem to rise allowing concrete to form a complete and solid infilling to the pile bore.

For all modern piling rigs instrumentation is used to record the pressure, the rate of placement of the concrete and the rate of ascent of the auger stem.

The benefit of this piling method is there is never a time during the installation process when an unsupported open bore exists. In addition, as the concrete is placed under pressure and against a soil surface that is roughened or distributed by the passage of the auger as it is withdrawn, a localized interstitial mixing of soil particles and the cement paste occurs at the soil/pile interface. This means that in the final case there is no preferential pathway for water migration in the long term.

This system is specifically designed so that there is little to no excavation required as the insitu concrete slab can be cast directly on the ground or on collapsible shutters. This enables the slab to be cast above the ground leaving a void below. The details provided within Appendix N demonstrate the two standard options which may be applicable to this site, depending on the final ground levels developed.

Where possible, services would be taken into the building externally above the ground floor slab level. Where it is unavoidable and services pass through the slab, two options for a waterproofing solution are available;

- Provide waterproof membrane either above or below the concrete slab and sealing the service openings
- Provide a waterproof concrete slab with a waterstop within the slab

6.5 Surface Water Run-off Rates

Where localised ground conditions indicate that infiltration into the sub-soils is not appropriate, due to high ground water levels or poor infiltration characteristics, surface water will be directed into a new ditch/swale system where the outflows will be restricted to match greenfield run-off rates via the use of flow control devices.

6.6 Surface Water Attenuation

Attenuation will be provided to accommodate a 1 in 100+CC pluvial event via a variety of devices which will include roof top attenuation (blue roof), permeable paving, bioretention areas and swales which is discussed in more detail in Chapter 8.

6.7 Exceedance Events

The attenuation within the permeable paving will be designed to hold a 100+CC event within the sub-base material, assuming no infiltration. This will replicate a high ground water event coinciding with a heavy rainfall event. Should the capacity of the attenuation within the hard paved areas be exceeded, any overflow will be directed to the adjacent swales/ditches and bioretention basin. These features will provide additional attenuation as well as a means of conveyance and surface water disposal via the Lancing Brooks Outfall. As discussed in Chapter 3, the water levels in the ditches are only partially influenced by ground water.

In the event that all the ditch and bioretention basins are also full the ground floor levels (mentioned above) will be locally elevated and the ground floors will be designed to be flood resilient. Accommodation is provided at first floor level, and as a consequence will provide an enhanced level of flood protection from exceedance events.

6.8 Safe Access and Egress

Due to the topography of the surrounding area, it may not be possible to provide a dry means of escape from the buildings during an extreme flood. To overcome this the units will have direct access to the first floor, which will be the primary area for refuge in the event of a major costal flood event.

All units will be linked to the EA's flood warning system and a site specific Flood Evacuation Plan will be provided, which gives guidance and advice to the residents with regards to the flood risks. The plan will also give details of the flood warnings, how the plan is triggered and what actions are required.

The Flood Evacuation Plan will need to be agreed with the local Emergency Planning Team.

This approach is a recognised strategy by the EA and has been accepted in a number of other developments within the Adur district.

6.9 Floodplain Compensation

The main flood risk associated with the site is from tidal/coastal flooding, consequently floodplain compensation will not be required. However, the existing drainage ditches will be extended and new ditches and bioretention areas will be added, which will provide additional surface water flood storage.

6.10 Other Sources of Flooding

Reservoirs

There are no reservoirs in the vicinity of the site and consequently this type of flooding is not applicable.

Foul Sewers

There are no recorded foul sewer flooding issues on the site, however there is an existing adopted foul sewer which crosses the south-eastern corner of the site. When this part of the site is developed, it is likely that the foul sewer will be diverted under a section 185 agreement with Southern Water.

It was anticipated that the existing foul sewerage system would not be able to accommodate a development of circa 300 units without the need for sewer upgrade works. Consequently, Southern Water have undertaken a capacity check, the results of which are included in Appendix C.

Southern Water have confirmed that there is currently insufficient capacity within their existing network to accommodate the increase in flows generated by the full development of circa 300 new units. Their report outlines that a total of 455m of existing 150mm dia and 200mm dia sewers require upgrading to 225mm dia. to provide the necessary sewerage infrastructure.

The report concludes that upgrading works are not required to the Wencelling Lancing Pumping Station or the Hasler Lancing Pumping Station.

At the design stage these upgrading works can be secured by way of a Section 98 Agreement (Sewer requisition) with Southern Water.

6.11 The Environment Agency

The above flood mitigation strategy has been submitted to the EA and their response is noted below. Copies of their correspondence is included within Appendix R.

Providing that finished floor levels to all habitable accommodation can be provided above 5.42mAOD to mitigate the tidal flood risks at the site, we would likely have no objections to the New Salts Farm development on flood risk grounds.

Although there are potentially other environmental issues that we have not considered in depth at this stage (e.g. biodiversity, groundwater protection), we have not identified any critical uncertainties with regards to these that would preclude residential development at the site.

The first floor level requested by the EA has subsequently been set at the 5.42m AOD level.

7.0 Sustainable Drainage Options

7.1 Many existing drainage systems can cause problems of flooding, pollution or damage to the environment and are proving unsustainable. Sustainable drainage systems (SuDS) are an alternative approach to conventional drainage design and implementation; they replicate natural drainage and deal with run-off where it occurs.

7.2 Appropriately designed, constructed and maintained SuDS are more sustainable than conventional drainage systems and can help to:

- Reduce run-off rates
- Reduce the risk of flooding
- Encourage natural groundwater re-charge
- Reduce volume of surface water run-off
- Provide habitats for wildlife

However, there are many site-specific factors which will influence the choice of any SuDS devices used within a development. The primary factors are:

- How the land is to be used- whether it be domestic, commercial or industrial
- Soil contamination
- Existing soil conditions i.e. ground permeability, water table levels
- Site topography e.g. steeply sloping
- Space availability – urban or non-urban

7.3 Most advice on the use of sustainable drainage techniques recommends the utilisation of ground infiltration, which may take the form of permeable paving, swales, infiltration basins or soakaways. However, these systems are dependent on the subsoil suitability, unsaturated soil zone to an adequate depth and the absence of leachable contaminants in the subsoil.

7.4 With reference to Appendix D, intrusive ground investigations have established that shallow infiltration is appropriate for this site. Consequently, within the development there is the potential to use a mixture of SuDS devices which include:

- Water Butts
- Green Roof
- Geocellular Roof Attenuation System (Blue Roof)
- Permeable Paving

- Swales, Bioretention Areas and Infiltration/Conveyance Ditches

7.5 Water Butts

Although not a primary SuDS device, when incorporated into other surface water management systems, water butts can reduce the total volume of storm water run-off and may also provide some additional storm water attenuation.

7.6 Green Roof

Green roofs have the benefit of providing an element of storm water attenuation and reducing the volume of surface water run-off, as well as the removal of air pollutants and dust.

Green roofs will be used across all roofs.

7.7 Geocellular Roof Attenuation Systems (Blue Roof)

These are plastic modular systems with a high void ratio that can be used to create a storage structure. They have the advantage of being flexible, lightweight and the flow control devices are integral with the system.

This system is to be used beneath the green roof system. Details of the Green/Blue roof systems are included within Appendix P.

7.8 Permeable Paving

Permeable paving provides a pavement suitable for pedestrians and vehicles whilst allowing rain water to infiltrate through the surface and into the underlying layers. The water is temporarily stored before infiltrating into the sub-soils. As well as providing surface water attenuation, they are also efficient at removing urban run-off pollutants, making them ideal for use in road and parking areas.

All hard paved areas, parking courts and access roads will be constructed using permeable paving.

Due to the potential for high ground water levels a geo-grid and a geotextile will be incorporated within the foundation of the road and parking areas. The geo-grid is specifically designed to create a stable road foundation within poor ground whilst still allowing infiltration, and the geotextile will prevent the potential upward migration of fine particles. Details of the geo-grid system and correspondence with the Council are included within Appendix O.

7.9 Swales, Bioretention Areas and Infiltration/Conveyance Ditches

Swales, Bioretention areas and infiltration/conveyance ditches are broad, shallow, soft landscaped areas designed to convey, store and infiltrate surface water run-off.

Soakage testing has established that infiltration into the ground will be permitted. The swales/infiltration ditches have been incorporated in the drainage strategy as an additional benefit to the principle of either 'total infiltration system' or 'no infiltration system'. Generally these ditches will infiltrate surface water runoff into the ground, however they will also be used to direct surface and ground water away from the buildings and into the bioretention basins during high ground water or heavy rainfall events. As a consequence the longitudinal gradients of these ditches need only provide a nominal falls to the bioretention basins and the standard guidance for gradients in swales is not applicable.

Detention areas will be located to match the existing low points on the site where surface water flooding would naturally occur in extreme events.

8.0 Surface Water Drainage Proposals

8.1 Guidance within The SuDS Manual states that surface water runoff from new developments should be dealt with in the following order of preference:

1. Discharge to the ground
2. Discharge to a surface water body
3. Discharge to a surface water sewer
4. Discharge to a combined sewer

Intrusive testing has confirmed that shallow infiltration is possible and as a consequence the proposed method of surface water disposal on the site will be via shallow infiltration.

At the request of the EA and the LLFA we were asked to develop an indicative site layout and drainage strategy to demonstrate that the drainage proposals discussed in this report can be delivered. It was agreed with the LLFA that for the purposes of demonstrating that drainage was possible details for the eastern half of the site would be provided.

With reference to the indicative drainage layout and calculations within Appendix J the roads and hard paved areas will be formed using permeable paving which will allow rainfall to percolate directly into the sub-soils. The dwellings will incorporate green roofs with a blue roof storage system below. An integral flow control device will restrict the run-off from the roofs to the minimum practical value, before discharge onto the permeable paving below, which will in turn percolate into the ground.

During heaver rainfall events where the rainfall exceeds the percolation rates surface water will be directed into the adjacent swales which will direct flows to the bioretention basin which is sized to temporarily store water while infiltrating through the base.

With reference to the ground water monitoring results in Appendix D. the ground water levels in the vicinity of the bioretention basin (WLS108) did not rise above 1.0m AOD throughout the duration of the monitoring period. Based upon these results the base level of the basin has been set at 1.1m AOD.

Using the Micro-drainage suite of design programmes it has been calculated that for a 1 in 100 year return pluvial event with an additional 40% allowance for climate change, a maximum depth of 104mm of water (1.204m AOD) would be temporarily stored within the basin.

The LLFA requested that the drainage strategy should model the effects of a 1 in 100+40% CC pluvial return period during a high ground water event, which would temporarily prevent infiltration into the sub-soils. To replicate the existing greenfield scenario during such an event, surface water would be directed to discharge into the adjacent Landing Brooks drainage system.

In a similar fashion to the infiltration scenario the surface water run-off from the hard paved areas would be directed into the adjacent swales and the bioretention basin.

The model was re-run with no infiltration and with a controlled outflow restricting flows to match the greenfield run-off rates (52.4l/s for a 14.69ha site area). The depth of water calculated within the bioretention basin was calculated to be 259mm rising to a level of 1.359m AOD.

Non-return values will be incorporated at the outfall into the existing ditch system to avoid water from the Lancing Brooks backing up into the new detention basins during a tide lock event.

Refer to Appendix J for detailed drainage calculations.

A tide lock situation with no outfall to the Lancing Brooks, and with no infiltration was also modelled for a 100+40% event. In this scenario the water level in the basin rose to 1.38m AOD with 280mm depth of water.

In the event that ground water levels temporarily rise above the base level of the bioretention basin, the outfall to the Lancing Brook will be set at a level of 1.2m AOD so that no more than 100mm of ground water would be retained within the basin. Consequently, a 1 in 100+40% rainfall event can still be accommodated within the basin.

The bank level of the basin is set a 1.6m AOD to provide an additional freeboard to accommodate potential exceedance effects as well as potentially higher ground water levels resulting from a rise in sea levels.

Other SuDS devices will be incorporated within the drainage design which includes:

- All blocks will incorporate roof attenuation (blue roof) below a green roof system, which will restrict the outflows to the minimum practical value of 0.5 l/s. The attenuation system will be designed to accommodate a 1 in 100 return period which includes an allowance for climate change.

- The discharge from the roof attenuation system will connect into the sub-base of the permeable roads and car parking areas.

- The new access road, parking courts and hard paved areas will be permeable with base infiltration.

- The sub-base thickness within the roads and hard paved areas will be designed to accommodate a 1 in 100 return period with an allowance for climate change. The design of the sub-base thickness will include the design flows from the houses.

- New swales/infiltration ditches will be introduced at the side of the new roads which will link into the bioretention basins. As well as providing an open drainage channel for infiltration they will also provide additional exceedance event storage and conveyance to the bioretention basin.

- Bioretention areas will be introduced within the landscaping design to provide additional areas for infiltration as well as exceedance event storage. The basins will be sizes to accommodate both infiltration and non-infiltration events.

- A new drainage channel will be constructed within the development site which will run parallel to the main channel within the Network Rail land. This channel will be designed to take additional/exceedance flows from the existing channel drainage ditch as well as providing additional flood storage capacity.

  Alternatively, to improve the limited access to the ditch within the Network Rail land it may be preferable to divert the main channel flows into the new channel. This option should be agreed with the EA and WSCC during the detailed design stage.

- All the existing ditches within the development area will be widened to improve the hydraulic characteristics as well as increasing the storage capacity within them.

8.2 Building Research Establishment (BRE)

As an independent check of the proposed drainage strategy mentioned above, the details and calculations provided within this report have been issued to the BRE for them to review. Their full response is attached within Appendix R, however *'it is the BRE's opinion that the flood risks can be managed on the site using the approaches set out by Hyde Housing Association and their consultants'.*

They do make comment that additional work should be undertaken during the detailed design stage to consolidate and add to the current evidence base, regarding the storage requirements for the SuDS design, and a comparison of the greenfield and developed sites design hydrographs. It would be expected this work would appropriately be carried out at the planning application stage.

8.3 Surface Water Treatment

To protect the quality of the ground water, all surface water run-off from the roof and hard paved areas will receive an element of surface water treatment before discharging into the ground.

The tables below refer to Chapters 26 within The SuDS Manual (Appendix K) which demonstrate that the proposed Pollution Mitigation Measures exceeds the Pollution Hazard Index, which consequently satisfies the level of treatment recommended within The SuDS Manual.

Table1: Pollution hazard indices for different land use classifications

Land Use	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Residential Roofs	Very low	0.2	0.2	0.05
Individual property driveways, residential car parks, low traffic roads (e.g. cul de sacs, homezones and general access roads) and on-residential car parking with infrequent change (eg schools, offices) i.e. <300 traffic movements/day.	Low	0.5	0.4	0.4
Commercial yards and delivery areas, non-residential car parking with frequent change (e.g. hospitals, retail) all roads except low traffic roads and trunk roads/ motorways.	Medium	0.7	0.6	0.7

Table 2: Indicative SuDS mitigation indices for discharge to groundwater

Characteristics of the material overlying the proposed infiltration surface, through which the runoff percolates	TSS	Metals	Hydrocarbons
Constructed permeable pavement (where a suitable filtration layer is included that provides treatment and including a geotextile at the base separating the foundation from the subgrade) underlain by a soil with good contaminant attenuation potential of at least 300mm in depth.	0.7	0.6	0.7

As each phase is developed the level of treatment prior to discharging will need to satisfy the above criteria.

9.0 Phased Development

It is anticipated that the development will be built over a number of phases. The drainage principles and flood mitigation measures previously discussed will be used throughout all phases. However, the ground conditions for each phase will need to be reviewed as the intrusive ground investigations undertaken to date has established that the geology and ground water levels across the development area do vary. If intrusive testing identifies that infiltration may not be appropriate, then surface water will discharge into the adjacent Lancing Brook drainage ditches.

Across the development phases new drainage ditches/swales will be introduced to provide additional surface water attenuation when the Lancing Brooks outfall becomes tide locked. If infiltration is not appropriate within any area, these swales can be used as a means of surface water disposal as they will be linked into the Lancing Brook ditch system. The new swales and bioretention areas will be designed to accommodate the additional volume associated with a tide lock period during a 100+CC event.

The ground floor levels will be locally raised to reduce the risk of ground water flooding habitable accommodation will only be permitted at first floor level, which will be set at the 1 in 200+CC level with an additional 300mm freeboard allowance.

Those areas which are currently located within a Flood Zone 3b (39% of the site) will not be developed until the Adur Tidal Wall scheme has been constructed which would then place these areas within a Flood Zone 3a.

10.0 Maintenance

Maintenance of any drainage scheme is essential to ensure that it continues to perform as designed. The SWMP notes that in the past the Lancing Brooks drainage system has been poorly maintained which has created a number of flooding issues.

The Lancing Brook ditches within the New Salts Farm site are an integral part of the drainage strategy and as such will require regular maintenance to prevent silt build-up and plant over growth in order to maintain an effective cross-sectional area of the ditch system.

The new surface water drainage system will require regular inspection/clearing to prevent blockages due to accumulation of silt. It is recommended that the system is initially inspected and cleared by a suitable trained person every 6 months for at least the first 2 years of operation to establish a long-term inspection/clearing interval appropriate for this site. Inspection/clearing should also be carried out after every major storm and flood event. The SuDS maintenance schedule will need to be agreed with the LLFA as part of the detailed design.

The SuDS features proposed within this development will be in areas that are visible and can be accessed without the need to access private land.

Details of the type and frequency of maintenance required for each element of the drainage system (including the Lancing Brooks) will be noted within the site Health, Safety and Maintenance file.

The freehold will be retained by Hyde, who are a Housing Association, and any reasonable maintenance obligations in respect of flooding and drainage can be secured by a S106 agreement.

11.0 New Flood Defences

It should be noted that the above mitigation measures are based upon the current flood data and does not take into account the benefits of the Adur Tidal Wall Scheme which has recently obtained a planning approval. These defenses were programmed to start construction in Spring 2016 (actually started in October 2016) and completed in 2018. At the time of updating this report (July 2017) seven of the ten construction phases are underway, which includes Reach W6, closest to the NSF development.

Upon completion, the Adur Tidal Wall Scheme will provide up to a 1 in 300 level of flood protection from tidal events to the New Salts Farm development site. This will reclassify the areas which are currently Flood Zone 3b to Flood Zone 3a. Details of the flood scheme are included in Appendix L.

12.0 West Beach Estate Drainage

The surface water drainage issues on the West Beach Estate are well documented. The historic plans suggest that it was the intention to drain the West Beach Estate roads onto the New Salts Farm, although it appears that the final connections were never made. The CH2M Hill report states that residents on two of the estate roads have made an informal connection onto the New Salts Farm land, whilst a number of the other roads have not (Appendix M).

The development of the New Salts Farm will look to formalise the surface water drainage connections from the West Beach Estate into the development proposals. Consequently, this would help to reduce the existing flooding issues currently experienced on the estate. As an interim measure, new ditches have recently been constructed adjacent to the site boundary (Appendix M), however these ditches will be subject to realignment during the detailed design stage of the adjacent construction phase.

13.0 Conclusion

New Salts Farm is located within a Flood Zone 3 area, where the main flood risk for the development is associated with coastal flooding. To overcome this, no habitable accommodation will be provided at ground floor level and the first floor level will be set 300mm above the 1 in 200+CC tide event. All units will be constructed using flood resilient materials and will be structurally designed to withstand potentially high flood depths.

Long term ground water monitoring has been undertaken on the site. Ground water levels fluctuate with the tides although local ground conditions do impact on the level of fluctuation across the site. Due to the close proximity to the sea, long term ground water level fluctuations appear to be reduced and remain relatively stable throughout the year. Level fluctuations appear to be more influenced by high tide events rather than rainfall, and as a consequence, only remain high for relatively short durations.

The Lancing Brooks drainage system crosses the development site, collecting surface water from the surrounding areas. They ultimately discharge to the River Adur via the Lancing Brooks Outfall. Historically these ditches have been poorly maintained which has caused a number of flooding issues. Recently these ditches have undergone a number of improvements/repairs throughout the Lancing Brooks network to improve their capacity and effectiveness.

It is proposed to divert part of the ditch system which runs through Network Rail land into this development which will provide improved access for inspection and maintenance as well as providing additional flood storage.

As part of the development the existing ditch system within the site will be widened to further improve the hydraulic characteristics of the ditches as well as increasing the storage capacity within them.

A new hydraulic model of the Lancing Brook has been undertaken which has confirmed that the flows within the ditches remain in channel for the 1 in 100+40% CC and 1 in 1000 pluvial event during a tide lock scenario.

Soakage testing has established that shallow infiltration is possible as a means of surface water disposal. An indicative drainage design based upon infiltration has been developed which demonstrates that the site can be drained solely by infiltration. However, based upon a worst-case scenario where ground water levels are unusually high the drainage system has been modelled with no infiltration but with a discharge to the adjacent Lancing Brooks system. In this situation outflows will be restricted to match greenfield run-off rates and non-return valves installed to ensure water within the exiting ditches does not back up into the new drainage system.

Tide lock scenarios with no infiltration have also been modelled for a 1 in 100+40% event. Water levels remain 220mm below the bank level of the bioretention basin.

All units will have green roofs with integral surface water attenuation (blue roof) which is designed to accommodate a 1 in 100+CC event. The run-off from these areas will be restricted to the minimum flow rate, which will, in turn, connect into the permeable sub-base of the roads.

All roads and hard standings will be permeable with base infiltration. The formation levels will generally sit above the high ground water levels. For particularly high ground water events the road foundations are designed to be stable if the water level rises to be within the road make-up. The sub-base thickness of the permeable paving will be designed to hold a 100+CC event to replicate a tide lock situation.

To reduce the risk of pollution all surface water run-off will receive the necessary level of treatment to accord with the requirements of The SuDS Manual.

A detailed maintenance strategy will be developed to ensure the drainage system continues to work as designed for the lifetime of the development. The long-term maintenance will be undertaken by a management company controlled by the Hyde Group for the lifetime of the development.

There are areas on the New Salts Farm which lie within a Flood Zone 3b. However, when the Adur Tidal Wall is completed the whole of the Farm will have improved flood protection. Those areas which currently fall within a Flood Zone 3b will be re-categorized as 3a. The mitigation methods proposed do not rely on the completion of the Adur Tidal Walls but once completed they will clearly provide additional flood protection.

As part of the New Salts Farm development, new drainage diches will be implemented to formally collect the surface water run-off the part of the West Beach Estate. This will help to reduce the surface water flooding issues currently experienced across part of the estate.

The principles developed to reduce the flood risks both within and beyond the site will be used when developing across all phases of the development. However, those areas which currently lie in Flood Zone 3b will not be brought forward until the Adur Tidal Wall scheme has been implemented.

The mitigation measures proposed will provide significant flood protection for the lifetime of the development.

A Sequential and Exception Test has been undertaken which demonstrates that the wider benefits of the proposed development outweigh the flood risks.

Extending the drainage ditches within the development area and providing an enhanced level of maintenance will also help to improve the efficiencies of the Lancing Brook Ditches. This, combined with improving the drainage to the West Beach Estate will also help to reduce the risk of flooding beyond the site.

The BRE have undertaken an independent review the flood and drainage proposals, and it was their opinion that the flood risks can be managed on the site

As a result of raising the first floor level to the units slightly the EA have also removed their objections to the development proposals.

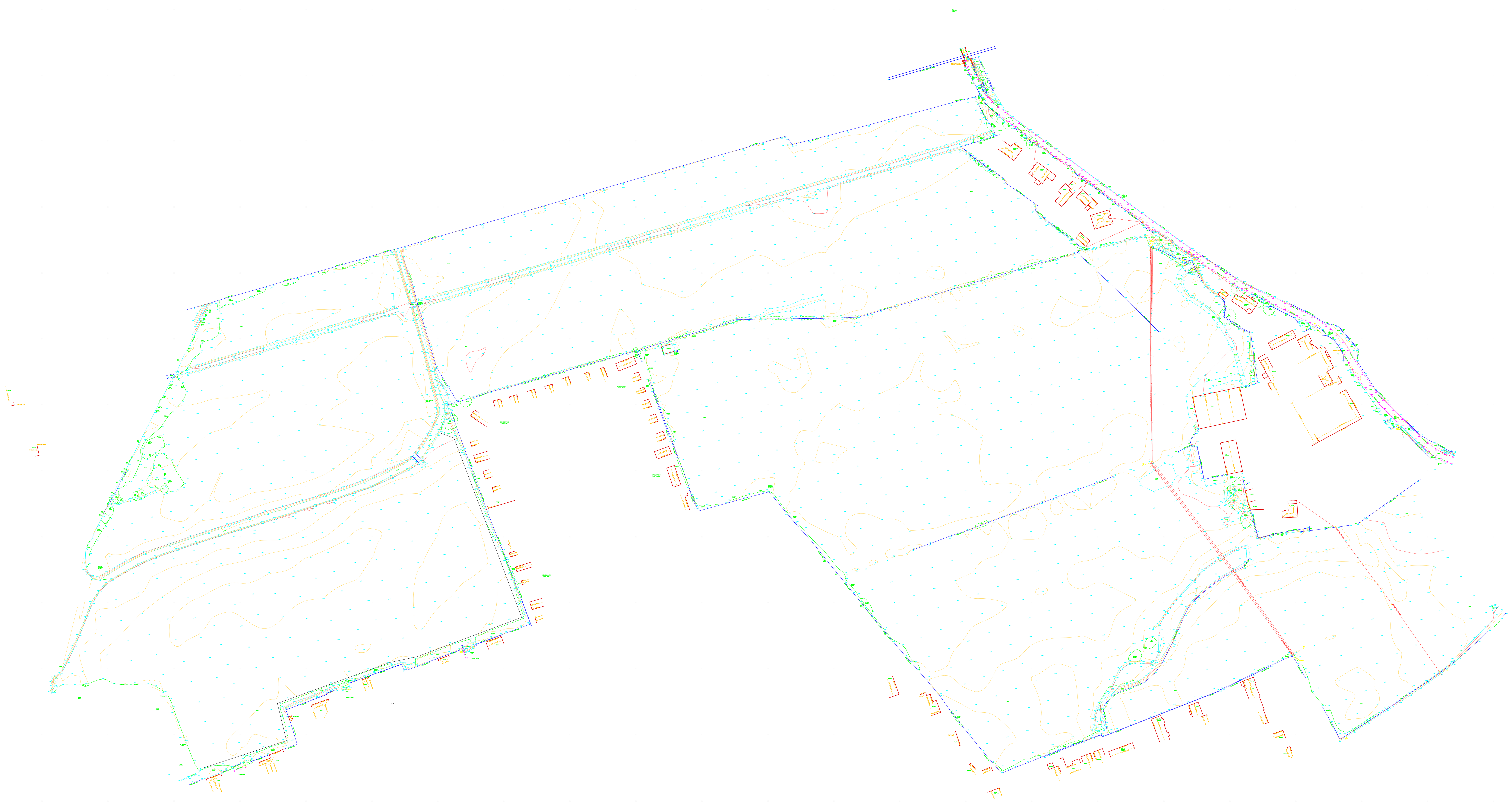
By introducing the above measures, it is considered that the proposed development would be suitable for inclusion within the Local Development Plan.

Appendix A – Site Location Plan

Site Location Plan

New Salts Farm Road,
Shoreham-by-Sea,
West Sussex,
BN43 5FE

Appendix B – Topographical Survey



Coordinate System:
Site grid is a Scale Factor 1, site centred around Stn.Field1, derivative of Ordnance Survey National Grid

project	New Salts Farm Lancing
client	Hyde New Build Ltd
drawing title	Land Survey

Dorset Land Surveying
Land Surveyors
Setting Out Engineers
Civil Engineering Design
Measured Building Surveys
Elevations and Street Scenes
Expert Witness Reports
GPS Surveys
Monitoring

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checked	MJH	grid	OSGB36*
datum			OSGM02
drawing number	6205	revision	

Appendix C – Southern Water Sewer Records and Capacity Check Report

105377

Node	Cover	Invert	Size	Material	Shape	
0101X	3.23	3.23	225	VC	CIRC	
0201X			UNK	UNK	CIRC	
0202X			UNK	UNK	UNK	
0200X			150	UNK	CIRC	
0302X			200	VC	CIRC	
0303X			UNK	UNK	CIRC	
0304X			UNK	UNK	UNK	
0305X			UNK	UNK	UNK	
0306Y	1.63		200	UNK	CIRC	
0307X			UNK	UNK	CIRC	
0308X			UNK	UNK	CIRC	
0300X	1.13		200	VC	CIRC	
0310X			150	UNK	CIRC	
0320X			150	UNK	CIRC	
0340X			UNK	UNK	CIRC	
0342X			200	UNK	CIRC	
0380X			150	UNK	CIRC	
0401X			UNK	UNK	CIRC	*
0401X	4.79	2.56	150	VC	CIRC	*
0402X			UNK	UNK	CIRC	
0403X			UNK	VC	CIRC	
0404X			UNK	VC	CIRC	
0405X			UNK	VC	CIRC	
0406X			UNK	UNK	CIRC	
0400X			UNK	UNK	CIRC	
0501X	3.41	1.96	150	VC	CIRC	
0502X	3.05	1.52	150	VC	CIRC	
0503X	2.31	0.89	150	VC	CIRC	
0601X	2.06	0.21	150	VC	CIRC	
0602X	2.13	−0.25	150	VC	CIRC	
0603X	1.92	−0.37	150	VC	CIRC	
0604X	1.89		150	VC	CIRC	
0605X			175	VC	CIRC	
060PY			175	VC	CIRC	
0904X	0.4		200	VC	CIRC	
1001X	1.9		200	VC	CIRC	
110DX			OTHER	OTHER	CIRC	
1150X	4.77	2.32	675	CP	CIRC	
1151X	4.82	2.49	900	CP	CIRC	
1201X	4.95	2.96	150	VC	CIRC	
1202X	4.97	2.85	150	VC	CIRC	
1250X	4.86	3.17	600	CP	CIRC	
1251X	4.73	2.34	600	CP	CIRC	
1301X			200	VC	CIRC	*
1301X	2.5	1	300	VC	CIRC	*
1302X	1.97		UNK	UNK	CIRC	
1303X	2.07		150	VC	CIRC	
1304X			UNK	UNK	CIRC	
1305X	1.82		200	VC	CIRC	
1306Y	2		200	VC	CIRC	
1300X			150	UNK	CIRC	
130DX			100	VC	CIRC	
1310X			UNK	UNK	CIRC	
1320Y			200	VC	CIRC	
1330X			150	VC	CIRC	
1340X			150	VC	CIRC	
1360X			200	VC	CIRC	
1401Y	4.53	1.8	150	VC	CIRC	*
1401X			UNK	UNK	CIRC	*
1401X	4.53	1.8	150	VC	CIRC	*
1402X	4.66	2.02	150	VC	CIRC	
1501X	2.38	1.1	150	VC	CIRC	
1500X			150	VC	CIRC	
1510X			UNK	VC	CIRC	
1601X	2.03	−0.48	200	VC	CIRC	
1602X	2.79		150	VC	CIRC	
1605X	2.32		150	VC	CIRC	
1901X	1.99		150	VC	CIRC	
1903X	2.33	1.54	150	VC	CIRC	
2001X	2.03	−0.26	200	VC	CIRC	
2002X	2.07	−0.18	150	VC	CIRC	
2003X			350	UNK	CIRC	
2000X			350	SI	CIRC	
2302X	2.08		150	VC	CIRC	
2303X	2.1	0.28	200	VC	CIRC	
2310X			200	VC	CIRC	
2401X	1.86		200	UNK	CIRC	
2901X	2.22	1.31	150	VC	CIRC	
3001X	2.27	1.41	150	VC	CIRC	
3002X	2.01	0.07	200	VC	CIRC	
3003X	2.13	0.29	200	VC	CIRC	
3004X			150	VC	CIRC	
3005X			150	VC	CIRC	
3000X			150	VC	CIRC	
3010X			150	VC	CIRC	
3020X			200	VC	CIRC	
3030X			150	VC	CIRC	
3040X			150	VC	CIRC	
3102X			100	VC	CIRC	
3103X			100	VC	CIRC	
3104X			100	VC	CIRC	
3105X			100	VC	CIRC	
3202X	5.73		150	ST	CIRC	
3201X	5.75		150	VC	CIRC	*
3203X	5.75	4.8	150	ST	CIRC	*
3204X	5.87		150	ST	CIRC	
3301X	2.12	1.32	150	VC	CIRC	
3403X	2.05		200	VC	CIRC	
340X			200	UNK	CIRC	
3406X	2.07	−0.15	200	VC	CIRC	
3407X			200	VC	CIRC	
3408X			UNK	UNK	CIRC	
3409X			UNK	UNK	CIRC	
3410X			UNK	UNK	CIRC	
3411X			UNK	UNK	CIRC	
3601X		−1.35	200	VC	CIRC	
3701X	3.28	1.68	200	VC	CIRC	
3702X			200	VC	CIRC	
4001X	2.25	1.25	150	VC	CIRC	
4003X	1.99	0.5	200	VC	CIRC	
4004X			UNK	UNK	CIRC	
4005X			UNK	UNK	CIRC	
4006X			UNK	UNK	CIRC	
4007X			UNK	UNK	CIRC	
4008X			UNK	UNK	CIRC	
4009X			UNK	UNK	CIRC	
400DX			200	VC	CIRC	
4010X			200	UNK	CIRC	
4011X			UNK	UNK	CIRC	
401DX			UNK	UNK	CIRC	
4101X	1.9	0.62	200	VC	CIRC	
4102X	2.21	1.25	200	CI	CIRC	
4103X			UNK	UNK	UNK	
4106X			100	VC	CIRC	
410AX			350	UNK	CIRC	
410DX			200	VC	CIRC	
411AX			250	UNK	CIRC	
4201X	6.11		150	ST	CIRC	*
4201X			UNK	UNK	CIRC	*
4401X	2.12	0.04	200	VC	CIRC	
4402X	2.29	0.09	200	VC	CIRC	
4403X	2.91	0.71	200	VC	CIRC	
4404X	1.72	0.36	150	VC	CIRC	
4405X	1.91		150	VC	CIRC	
4410X	0.35		200	VC	CIRC	
4420X	0.37		150	VC	CIRC	
4430X			UNK	UNK	CIRC	
4501X	1.08	−0.04	200	VC	CIRC	
5001X	2.33	1.49	150	VC	CIRC	
5101X			UNK	VC	UNK	
5102X			200	VC	CIRC	
5103X	1.4	0.51	150	VC	CIRC	
5104X			UNK	UNK	CIRC	
510DX			200	VC	CIRC	
5201X	1.75		200	CI	CIRC	
5202X	1.82	1.19	200	CI	CIRC	
5203X			150	VC	CIRC	
5204X			UNK	VC	CIRC	
5301X			225	VC	CIRC	
5302X			225	VC	CIRC	*
5302X			150	VC	CIRC	*
5303X			UNK	UNK	CIRC	
5304X			UNK	UNK	CIRC	
5401X	2.3	0.46	200	VC	CIRC	
5403X	2.14	1.03	150	VC	CIRC	
5408X			100	VC	CIRC	
5409X			150	VC	CIRC	
5410X			100	VC	CIRC	
541DX			OTHER	OTHER	CIRC	
5501X	2.03		150	UNK	CIRC	
5502X	2.19	1.3	150	UNK	CIRC	
5503X	2.21		150	UNK	CIRC	
5504X	2.4		150	UNK	CIRC	
5505X	2.37	0.66	150	UNK	CIRC	
5507X			150	UNK	CIRC	
6101X	2.02	1.34	150	VC	CIRC	
6102X	1.99	1.2	150	VC	CIRC	
6103Y	1.9	0.78	200	VC	CIRC	
6104X	1.4	0.82	150	VC	UNK	
6105X	1.4	0.76	150	VC	UNK	
6106X	1.4	0.57	150	VC	UNK	
6107X			UNK	UNK	CIRC	
610DX			150	VC	CIRC	
6201X	1.75	0.56	200	CI	CIRC	
6202X			UNK	UNK	UNK	
6203X			UNK	VC	CIRC	
6301X	2.36	1.56	150	VC	CIRC	*
6301X	1.97		200	CI	CIRC	*
6302X			225	VC	CIRC	*
6302X			UNK	UNK	UNK	*
6303X			UNK	UNK	UNK	
6304X			225	VC	CIRC	*
6304X			UNK	UNK	UNK	*
630DX			200	CI	CIRC	
6401X	2.16		150	VC	CIRC	*
6401X	2.01	1.34	200	CI	CIRC	*
6402X	2.16		150	VC	CIRC	*
6402X			UNK	UNK	CIRC	*
6403X			UNK	UNK	CIRC	
6405X	2	0.7	150	VC	CIRC	
6407X	2.34		150	VC	CIRC	
6408X			UNK	UNK	UNK	
640DX			UNK	UNK	CIRC	
6501X	2.58	0.36	150	UNK	CIRC	
6504X	2.51	0.08	150	UNK	CIRC	
7101X	1.98	1.27	150	VC	CIRC	
7102X	2.23	1.35	150	VC	CIRC	
7201X	1.84	0.24	200	VC	CIRC	
7202Y	1.81	0.51	200	VC	CIRC	
7203X	1.78		200	CI	CIRC	
7204X			UNK	VC	CIRC	
7205X			200	CI	CIRC	
7206X			UNK	VC	CIRC	
720DX			UNK	UNK	CIRC	
722DX			UNK	VC	CIRC	
723DX			200	CI	CIRC	
7301X			225	VC	CIRC	*
7301X			UNK	UNK	CIRC	*
7302X			225	VC	CIRC	
7401X			UNK	UNK	CIRC	*
7401X	3.11	0.74	150	VC	CIRC	*
7402X			UNK	UNK	CIRC	*
7402X	3.29	1.29	150	VC	CIRC	*
7403X	3.31	1.54	100	VC	CIRC	*
7403X			UNK	UNK	CIRC	*
7404X			UNK	UNK	CIRC	
7405X			UNK	UNK	CIRC	
7501X	2.11	0.3	150	UNK	CIRC	
7503X	2.02		150	UNK	CIRC	
7504X			150	UNK	CIRC	
7505X	1.88		150	UNK	CIRC	
7506X	2.01	0.28	125	CI	CIRC	
7507X	2.05	0.67	150	UNK	CIRC	
7508X	1.94		150	UNK	CIRC	
7509X	1.65		150	UNK	CIRC	
750PX			125	CI	CIRC	
7510X			UNK	UNK	CIRC	
7512X			150	UNK	CIRC	
7513X			UNK	UNK	CIRC	
7514X			UNK	UNK	CIRC	
7515X			UNK	UNK	CIRC	
7516X			UNK	UNK	CIRC	
751DX			UNK	VC	CIRC	
8201X	2.19	1.29	150	VC	CIRC	
8202X	2.33	1.29	150	VC	CIRC	
8203X	1.85	0.49	200	CI	CIRC	
8204Y	1.87		200	VC	CIRC	
8205Y			150	VC	CIRC	
8206X			350	UNK	CIRC	
8207X			150	VC	CIRC	
820DX			150	VC	CIRC	
820PX			250	UNK	CIRC	*
820PY			200	VC	CIRC	*
821DX			150	VC	CIRC	
822DX			UNK	UNK	CIRC	
823DX			UNK	UNK	UNK	
824DX			200	CI	CIRC	
8301Y			UNK	UNK	CIRC	
8302X			200	CI	CIRC	
8303X			UNK	UNK	CIRC	*
8303X	4.6	3.57	150	VC	CIRC	*
8304X			UNK	UNK	UNK	
8305X			UNK	UNK	UNK	
8306X			UNK	UNK	UNK	
830DX			UNK	UNK	CIRC	
831DX			UNK	UNK	UNK	
832DX			UNK	UNK	CIRC	
8401X	1.92	−0.52	150	VC	CIRC	*
8401X			UNK	UNK	CIRC	*
8402X			UNK	UNK	UNK	
8403X			UNK	UNK	UNK	
8404X			150	UNK	CIRC	
8405X			150	VC	CIRC	
8406X			150	VC	CIRC	
840DX			150	VC	CIRC	
8501X	2.23		150	UNK	CIRC	
8502X	2.3		UNK	UNK	UNK	
850ZX			125	CI	CIRC	
8615X	1.97	−0.91	150	UNK	CIRC	
8616X	2.1	0.73	150	UNK	CIRC	
9201X	2.28	1.29	150	VC	CIRC	
9202X	1.84	1.84	200	CI	CIRC	
9203X			UNK	VC	CIRC	
9204X			UNK	VC	CIRC	
9301X	1.81	0.56	200	VC	CIRC	
9302X			UNK	UNK	CIRC	
9303X			UNK	UNK	CIRC	
9304X			UNK	UNK	CIRC	
9305Y			150	VC	CIRC	*
9305X			UNK	UNK	UNK	*
9306X			UNK	UNK	UNK	
9307X			UNK	UNK	UNK	
9308X			UNK	UNK	UNK	
9309X			UNK	UNK	CIRC	
930DX			150	UNK	CIRC	
9310X			UNK	UNK	CIRC	
9312X			UNK	UNK	UNK	
931DX			150	UNK	CIRC	
932DX			150	UNK	CIRC	
933DX			UNK	UNK	CIRC	
934DY			UNK	UNK	CIRC	
93ADY			UNK	UNK	CIRC	*
93ADX			150	VC	CIRC	*
9401X	4.63	3.05	150	VC	CIRC	*
9401X			UNK	UNK	UNK	*
9402X			100	VC	CIRC	
9403X			100	VC	CIRC	
9501X	3.09	1.13	150	UNK	CIRC	
9503X	2.67	0.74	150	UNK	CIRC	
9504X			UNK	UNK	UNK	
9602X	2.07		150	UNK	CIRC	
9603X	2.27	0.33	150	UNK	CIRC	
9604X	2.1		150	UNK	CIRC	
9605X	2.06	0.34	150	UNK	CIRC	
960AX			125	CI	CIRC	
9610X			UNK	UNK	UNK	
9611X			150	UNK	CIRC	

Node Cover Invert Size Material Shape

LINE STYLES / COLOURS

- Brown: Foul; Foul Syphon Sewer; Foul Vacuum Main; Foul Rising Main
- Red: Combined; Combined Syphon Sewer; Combined Rising Main; Lateral Drain
- Orange: Building Over Agreement Area
- Dark Blue: Treated Effluent
- Purple: Sludge; Sewer Catchment; Section 104 Area
- Light Blue: Surface Water; Surface Water Rising Main
- Yellow: Private
- Green: Access Shaft; Decommissioned

MATERIALS

AK Alkathene; BAC Bonded Asbestos Cement; BRC Brick (Common); BRE Brick (Engineering); CC Concrete Box Culvert; CI Cast Iron; CO Concrete (In-Situ); CP Concrete (Pre-Cast); CSB Concrete Segments (bolted); CSU Concrete Segments (unbolted); DI Ductile Iron; GRC Glass Reinforced Concrete; GRP Glass Reinforced Plastic; MAC Masonry in regular Courses; MAR Masonry in random Courses; PE Polyethylene; PF Pitch Fibre; PP Polypropylene; PVC Polyvinyl Chloride; RPM Reinforced Plastic Matrix; SI Spun Iron; ST Steel; VC Vitrified Clay; XXX Other; ZZZ Unknown

WARNING: BAC pipes are constructed of Bonded Asbestos Cement

LEGEND - SEWERS

Manhole (SW); Manhole (F&C); Lamp hole (SW); Lamp hole (F&C); Pumping Station (SW); Pumping Station (F&C); Side entry manhole (SW); Side entry Manhole (F&C); Blind shaft (SW); Blind shaft (F&C); Ejector station (SW); Ejector station (F&C); Watertight door (SW); Watertight door (F&C); Flushing ch. Mn-e (SW); Flushing ch. Mn-e (F&C); Flushing ch. No-e (SW); Flushing ch. No-e (F&C); Demarcation Chamber

Washout (SW); Washout (F&C); Rodding Eye (SW); Rodding Eye (F&C); Gauging point (SW); Gauging point (F&C); Intercept chamber (SW); Intercept chamber (F&C); Storm Tank (SW); Storm Tank (F&C); Vortex chamber (SW); Vortex chamber (F&C); Label ellipse; Dummy/S24 manhole; Outfall; Penstock chamber; Damboards; Storm Overflow; Backdrop manhole

Other (s); Other; Change in sewer (s); Change in sewer; Reflux valve; Flap valve; Cascade; Anode; Valve; Closed Valve; Air Valve; Hatch box (SW); Hatch box (F&C); Direction arrow; Emptying valve; Catchpit; Soakaway; Inlet; Balancing Pond

Wastewater treatment works; Marine treatment works; Outfall headworks; Vent; Vent column; Tidal storage tank; Blank end; Head of Public Sewer; Micro Pumping Station

SHAPES (S)

A Arched; B Barrel; C Circular; E Egg; H Horseshoe; R Rectangular; S Square; T Trapezoidal; U U Shape; X Other

NODE REFERENCING SYSTEM

1st digit: hundred metre easting identifier
2nd digit: hundred metre northing identifier
3rd digit: sewer type identifier 0-4 = Foul/Combined 5-9 = Surface Water
4th digit: next sequential node

WARNING: Unknown (UNK) materials may include Bonded Asbestos Cement

O.S. Ref.	TQ2004SW	Drawn by:	kumaria
Scale:	1:2500	Date:	17/02/2016

Title: 210503_1 Millennium Close, Sho

N

Southern Water

The information contained on this plan is provided as a guide to the approximate position of existing public sewers. The accuracy thereof cannot be guaranteed.

THIS INFORMATION MUST BE TREATED WITH CAUTION AND THE ACTUAL POSITION OF THE PUBLIC SEWERS MUST BE ESTABLISHED ON SITE, IN ALL CASES.

Southern Water must be informed before lifting manhole covers or excavating trial holes.

103439

519061

SEWER RECORDS

521198

Southern Water

Tully De'Ath Consultants
Sheridan House
Hartfield Road
Forest Row
East Sussex
RH18 5EA

Developer Services
Southern Water
Sparrowgrove House
Sparrowgrove
Otterbourne
Hampshire
SO21 2SW

Tel: **0330 303 0119**
Email: ***developerservices@southernwater.co.uk***

Your Ref:

Our Ref:
CC-SUSS-00371

Date:
7th July 2016

F.A.O: Andrew Picton

Site: New Salts Farm, Shoreham-by-Sea, West Sussex

Dear Sirs,

Further to your recent application for a level 2 enquiry regarding the above development site.

FOUL WATER

Please find enclosed the report which details the extent of works required in order for your proposed development site to be accommodated within the local sewerage infrastructure. It should be noted that the report is only a hydraulic solution to provide capacity for the proposed development site. There are other options available to you/ your client in discharging your proposed flows from the site such as making a connection application/requisition to a sewer at a point where capacity is currently available. Section 98 of the Water Industry Act 1991 provides a legal mechanism through which the appropriate infrastructure can be requested (by the developer) and provided to drain a specific location.

As you are aware the owner or occupier of the premises or private sewer or drain must give the sewerage undertaker notice of the proposed works. Upon receipt of the notice the undertaker may, within 21 days, refuse to allow him to make the connection if it considers that the mode of construction or the condition of the connecting sewer or drain either does not satisfy the standards reasonably required by the undertaker, or is such that the making of the communication would be prejudicial to its sewerage system.

The right of refusal is limited to these criteria alone and the undertaker cannot refuse to allow the connection on the grounds that the sewerage system is inadequate to take the extra liquid that the connection will discharge into it. This is a matter that should be dealt with at the planning stage.

Yours faithfully

Developer Services

Please note: - The information provided above does not grant approval for any designs /drawings submitted for the capacity analysis. The results are an indicative hydraulic assessment and should not be used as a basis for design. The results quoted above are only valid for 12 months from the date of issue of this letter.

INFRASTRUCTURE ASSESSMENT FOR FOUL DRAINAGE AT

NEW SALTS FARM,

NEW SALTS FARM ROAD,

SHOREHAM-BY-SEA,

WEST SUSSEX

BN43 5FE

7th July 2016

REQUESTED:
TULLY DE'ATH CONSULTANTS

Infrastructure Assessment for Foul Drainage at New Salts Farm

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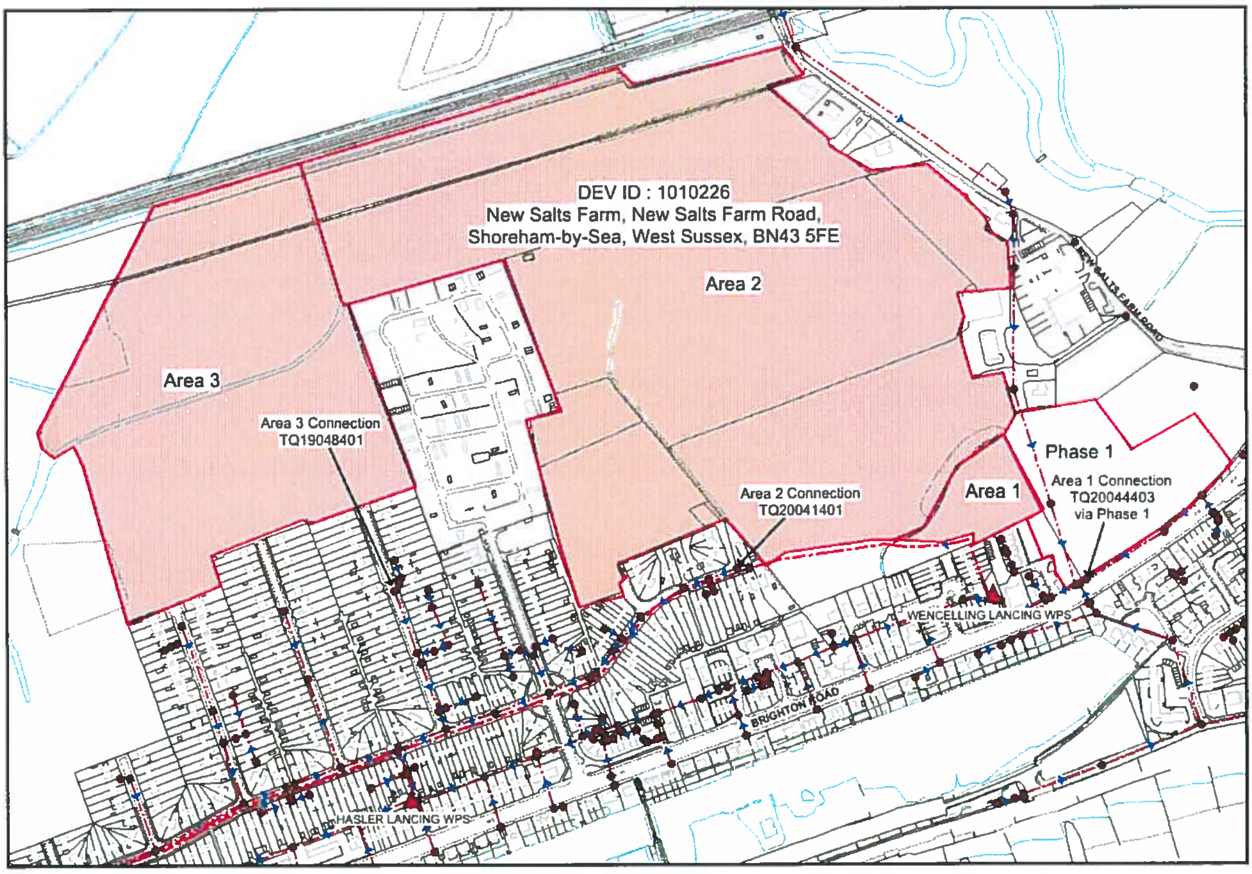
I. Development Details:

No preferred discharge manholes were specified in the application. The Development has been subdivided into three areas to allow discharge to local manholes as shown in Figure 1 below.

- Area 1 to discharge to Manhole TQ00044403 on Brighton road via the Phase 1 Development (see CC-SUSS-00372).
- Area 2 to discharge to Manhole TQ20041401 on Orient Road.
- Area 3 to discharge to Manhole TQ19048401 on Bristol Avenue.

Figure 1 - Proposed Development



II. Results and Conclusions:

Foul Water:

There is currently inadequate capacity within the local foul sewerage network to accommodate the foul flow from the proposed development site at MHs TQ20044403, TQ20041401 and TQ19048401. The proposed development would increase flows to the local network and as a result existing properties and land may be subject to a greater risk of flooding. Additional off-site sewers or improvements to existing sewers will be required to provide sufficient capacity to service the proposed development as indicated in Figure 2 and Table 2 below, in addition to those indicated in the Phase 1 Development response (see CC-SUSS-00372).

Figure 2 - Proposed Improvement – Foul system

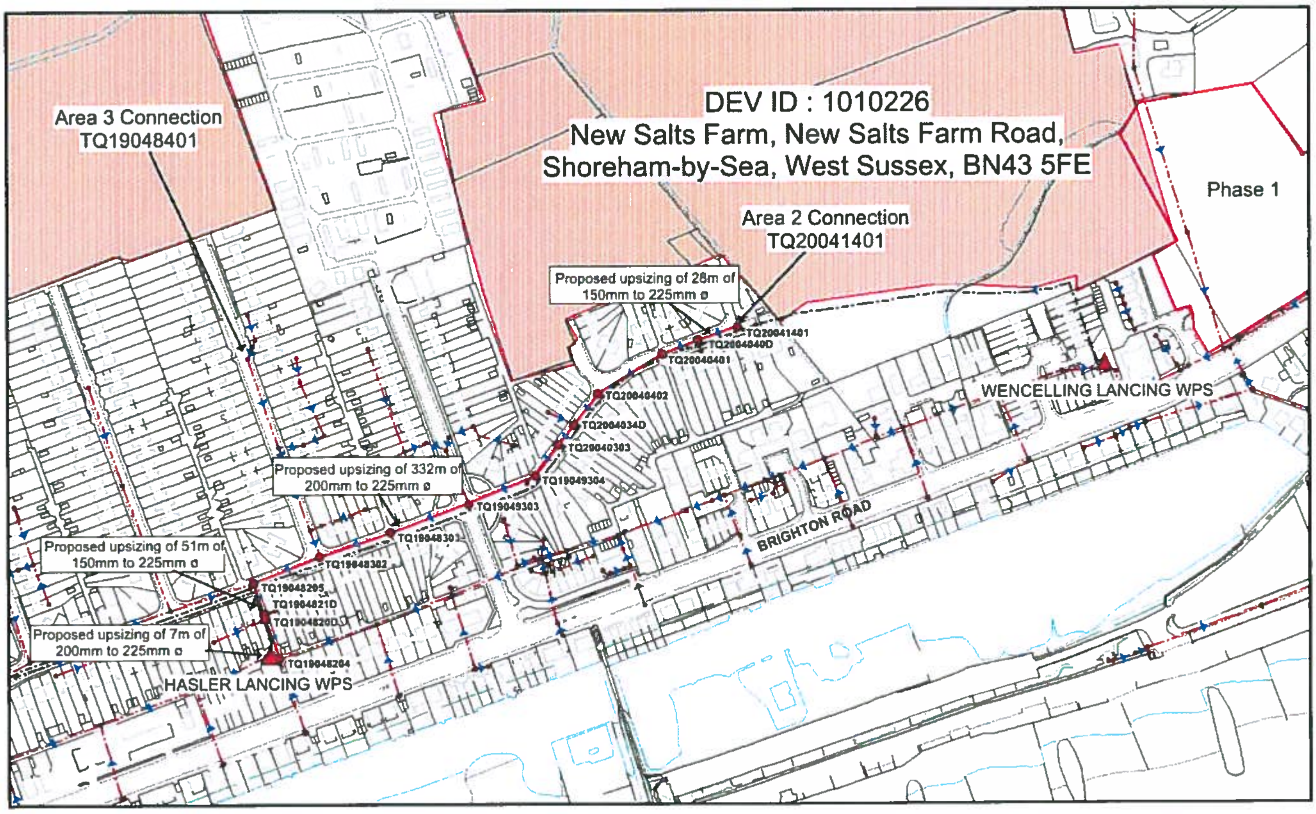


Table 2 Proposed Improvements Schedule

Manhole	Manhole	Sewer Diameter (mm)		Av. Depth (m)	Length (m)
		Existing	Proposed		
TQ20044403	TQ2004441D	200	225*	2.6	8
TQ2004441D	TQ20044402	200	225*	2.3	4
TQ20044402	TQ20044401	200	225*	2.2	25
TQ20041401	TQ2004040D	150	225	1.2	28
TQ2004040D	TQ20040401	200	225	1.2	25
TQ20040401	TQ20040402	200	225	1.2	48
TQ20040402	TQ2004034D	200	225	1.2	26
TQ2004034D	TQ20040303	200	225	1.2	15
TQ20040303	TQ19049304	200	225	1.2	25
TQ19049304	TQ19049303	200	225	1.2	46
TQ19049303	TQ19048303	200	225	1.3	53
TQ19048303	TQ19048302	200	225	1.5	48
TQ19048302	TQ19048205	200	225	1.6	46
TQ19048205	TQ1904821D	150	225	1.7	20

TQ1904821D	TQ1904820D	150	225	1.7	4
TQ1904820D	TQ19048204	150	225	2.0	27
TQ19048204	TQ1904820P	200	225	2.3	7
				Total	455

*Upsizing required for Phase 1 Development (see CC-SUSS-00372)

Surface Water System:

As a surface water capacity check has not been requested it is assumed that all Surface Water will be disposed of by alternative means i.e. Soakaway or any local drainage watercourses, subject to all interested parties approval.

Before any connections are made, an application form needs to be completed and approved by Southern Water Services.

Please note: - The information provided above does not grant approval for any designs /drawings submitted for the capacity analysis. The results are an indicative hydraulic assessment and should <u>not</u> be used as a basis for design. The results quoted above are only valid for 12 months from the date of issue of this letter.

PUBLIC SEWER RECORD

Appendix D – Ground Monitoring Results and Soakage Test Results

Our Ref: AH/BC/J12765

Southern Testing
Environmental & Geotechnical

Keeble House, Stuart Way
East Grinstead, West Sussex RH19 4QA

t 01342 333100 f 01342 410321
e info@southerntesting.co.uk w southerntesting.co.uk

Directors M W Stevenson BSc MBA CEng CEnv MICE CGeol FGS MconsE (Chief Executive)
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S F Pratt BSc MSc CGeol FGS DIC
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P J Sugden BSc MSc FGS
J M Hickmott BSc CEng CEnv MICE FGS (Non Executive)
A J Timms CEng MICE (Non Executive)
Co. Secretary J N Joseph
Consultant Dr Derek Petley BSc PhD DIC MHIT FGS
D Illingworth BSc FGS

13 March 2017

Tully De'Ath Consultants
Sheridan House
Hartfield Road
Forest Row
East Sussex - RH18 5EA

For the attention of Mr Andrew Picton

Dear Sirs,

Re: BRE365 Soakage Tests at: New Salts Farm, Shoreham
National Grid Reference: TQ 204 045
Geology: Alluvium/Marine Deposits over Newhaven Chalk

Introduction

The site has been subject to several phases of ground investigation and ongoing groundwater monitoring, and additional work has been requested to determine the infiltration rate of the shallow soils on site with respect to the use of permeable paving as part of the proposed development.

Presented here are the logs of the trial pits, a location plan, and the results of all the BRE 365 soakage tests.

Scope

The site investigation was conducted and this report has been prepared for the sole internal use and reliance of Hyde Housing and their appointed Engineers. This report shall not be relied upon or transferred to any other parties without the express written authorization of Southern Testing Laboratories Ltd. If an unauthorised third party comes into possession of this report they rely on it at their peril and the authors owe them no duty of care and skill.

Fieldwork

A total of three shallow trial pits were excavated at locations as specified by our client (see figure 1). The soils encountered comprised a shallow covering of topsoil over firm orange brown mottled grey sandy Clay with frequent shell fragments.

BRE365 soakage tests were carried out at each trial pit location over a period of two days.

The BRE 365 paper on soakaway design allows for the design of trench soakaways as well as traditional square and circular soakaways.

The test to measure the soil infiltration rate is carried out in pits which are excavated to the full depth of the proposed soakaway. The trial pits are filled and allowed to drain to empty or near empty, three times, on the same day or on consecutive days.

The pit is considered full when the water level is the same as the proposed inlet invert. The time for the water level to fall from ¾ full to ¼ full is obtained and the soil infiltration rate is obtained from the following formula:

$$f = \frac{V_{p75-25}}{a_{p50} \times t_{p75-25}}$$

Where: f = soil infiltration rate (in this case expressed in l/m^2/minute)

V_{p75-25} = the effective storage volume of water in the trial pit between 75% and 25% effective depth;

a_{p50} = the internal surface area of the trial pit up to 50% effective depth and including the base area;

t_{p75-25} = the time for the water level to fall from 75% to 25% effective depth.

The soakage rate in this report in expressed as l/m^2/minute, which is a convenient rate to use. The BRE use a unit of m/sec, which is the value in l/m^2/minute divided by 60,000.

BRE 365 stipulates that the pit should be refilled three times in all, on the same day, or on successive days. Where the sides are unstable the pit should be filled with granular material to provide stability.

Results

A total of three soakage tests were carried out across the site, at the locations shown on the attached site plan (Figure 1). The full results of the soakage tests are appended, however, the soakage rate from each trial hole is summarised in the table below.

Results of BRE 365 Soakage tests				
Pit	Depth (m)	f l/m^2/min	BRE Units (m/sec)	Comments
TP1	0.6	0.712	7.12E-06	Incomplete test 25% not reached
TP1 (Test 2)	0.6	0.340	5.66E-06	Incomplete test 25% not reached
TP2	0.7	0.337	5.61E-06	Incomplete test 25% not reached
TP2 (Test 2)	0.7	0.154	2.57E-06	Incomplete test 25% not reached
TP3	0.6	1.91	3.19E-05	
TP3 (Test 2)	0.6	1.03	1.72E-05	
TP3 (Test 3)	0.6	0.773	1.29E-05	

The BRE Digest 365 indicates that 3 fillings must be carried out. Where 3 fillings have not been carried out, a reduction factor should be applied.

Discussion

The BRE 365 test carried out at the location of TP1 indicated generally poor infiltration. Over the test period, only 2 fills were possible, due to poor soakage rate. Inferred infiltration rates of 0.427 litres/m^2/min and 0.340 litres/m^2/min have been calculated.

Poor infiltration was also recorded at the location of TP2, whereby similarly the tests were not completed. Due to very slow soakage, an inferred infiltration rate of 0.337 litres/m^2/min and 0.154 litres/m^2/min were calculated.

The soakage tests undertaken at the location of TP3 did comply with the BRE 365 test method, with a complete test of three fillings achieved. Infiltration rates of between 1.91 litres/m^2/min and 0.773 litres/m^2/min were calculated.

Since two of the three BRE 365 soakage tests carried out did not fully comply to the BRE365 test standard due to the poor infiltration rates, the value used for infiltration should be carefully considered. The BRE365 test method is used for the design of conventional shallow soakaways through the use of concrete rings buried in the ground or gravel filled trenches. For this site it is understood that the surface water discharge plan is to use permeable paving only, whereby effective drainage is over a large area, and the BRE 365 methodology does not provide advice on such systems.

Therefore, based on the test results carried out, a mean design value could be considered for design, which based on the results achieved on site is approximately 0.4 litres/m^2/min. The designer should apply and appropriate factor of safety.

We hope that the attached information is of assistance.

Yours faithfully,

Andrew Holley MSc FGS
Senior Geological Engineer
For and on behalf of
Southern Testing Laboratories Limited

Southern Testing ST Consult	**Project Name:**	New Salt Farm	**Project ID:**	J12765	BRE 365 Test Pit Location Plan
	Location:	Shoreham-By-Sea	**Engineer:**	AH	
	Client:	Hyde Housing	**Scale:**	1:2500	

Legend Key

BRE 365 test pits - BRE 365 test pits

Southern Testing ST Consult

www.southerntesting.co.uk tel:01342 333100 www.stconsult.co.uk tel:01604 500020

Start - End Date:	Project ID:	Machine Type:	TP1
07/07/2017	J12765	JCB 3 CX	Sheet 1 of 1

Project Name:	New Salt Farm	**Remarks:**	**Co-ordinates:** E 520436 - N 104528	**Level (m AOD):**	**Logger:** DK
Location:	Shoreham-By-Sea	Dry			
Client:	Hyde Housing				

Samples and Insitu Testing: Depth (m)	Type	Results	Level (m AOD)	Thickness (m)	Legend	Depth (m bgl)	Stratum Description
				(0.20)		0.20	Grass over brown organic slightly sandy clayey TOPSOIL with frequent fine roots and flint gravel.
				(0.40)		0.60	Firm orange brown mottled grey silty fine sandy CLAY with occasional fine rootlets and shell fragments.
							Pit terminated at 0.60m.

Depth scale: 1, 2, 3, 4

Pit Dimension (m)		Pit Stability:	Water Strikes:
Width:	0.60	Stable	
Length:	1.20		
Depth:	0.60		

Southern Testing ST Consult

www.southerntesting.co.uk tel:01342 333100 www.stconsult.co.uk tel:01604 500020

Start - End Date:	Project ID:	Machine Type:	TP2
07/07/2017	**J12765**	JCB 3 CX	**Sheet 1 of 1**

Project Name:	New Salt Farm	**Remarks:**	**Co-ordinates:** E 520192 - N 104535	**Level (m AOD):**	**Logger:** DK
Location:	Shoreham-By-Sea	Dry			
Client:	Hyde Housing				

Samples and Insitu Testing: Depth (m)	Type	Results	Level (m AOD)	Thickness (m)	Legend	Depth (m bgl)	Stratum Description
				(0.30)		0.30	Grass over dark brown organic silty clayey TOPSOIL with frequent fine roots and fine to coarse flint gravel.
				(0.40)		0.60	Firm, orange brown mottled pale grey silty fine sandy CLAY with occasional fine roots and shell fragments.
				(0.40)		0.70	Firm, orange brown mottled grey silty slightly sandy CLAY with occasional shells and roots.
							Pit terminated at 0.70m.

Pit Dimension (m)		Pit Stability:	Water Strikes:
Width:	0.60	Stable	
Length:	1.30		
Depth:	0.70		

Southern Testing ST Consult www.southerntesting.co.uk tel:01342 333100 www.stconsult.co.uk tel:01604 500020	Start - End Date:	Project ID:	Machine Type:	TP3
	07/07/2017	J12765	JCB 3 CX	Sheet 1 of 1

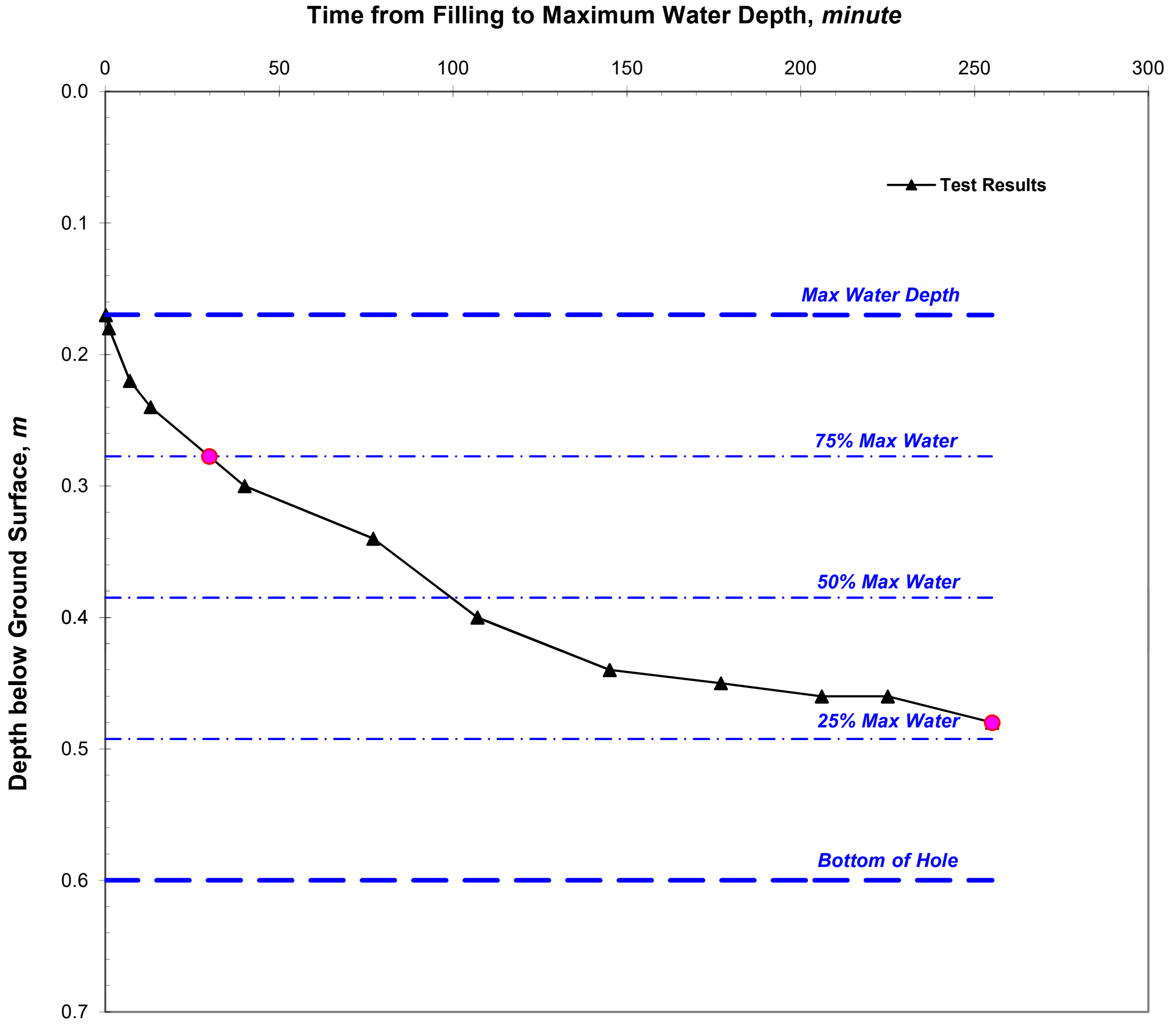
Project Name:	New Salt Farm	**Remarks:**	**Co-ordinates:** E 520126 - N 104657	**Level (m AOD):**	**Logger:** DK
Location:	Shoreham-By-Sea	Dry			
Client:	Hyde Housing				

Samples and Insitu Testing: Depth (m)	Type	Results	Level (m AOD)	Thickness (m)	Legend	Depth (m bgl)	Stratum Description
				(0.15)		0.15	Grass over dark brown organic clayey TOPSOIL with frequent roots and flint gravel.
				(0.45)		0.60	Firm to stiff, orange brown slightly sandy silty CLAY with occasional fine roots.
							Pit terminated at 0.60m.

Pit Dimension (m)		Pit Stability:	Water Strikes:
Width:	0.60	Stable	
Length:	1.20		
Depth:	0.60		

BRE Digest 365 Soakage Test

Test Hole No: TP1
Test No: Test No 1 (Initial)



Pit Length, m	1.200	Depth to Water at Start of Test, *m*	0.170
Pit Width, m	0.600	Max Water Dropdown during Test, *m*	0.310
Depth to Pit Base, m	0.600	Total Soakage Test Time, *min*	255.0
Depth to Top of Permeable Soils, *m*		Mean Internal Discharge Area, m^2	1.517
Depth to Groundwater Surface, *m*		Discharge Rate, *litre/min*	0.648
Depth to Top of Granular Fill, *m*		Soakage Rate, $litre/m^2/min$	**0.427**
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, *m/sec*	**7.12E-06**

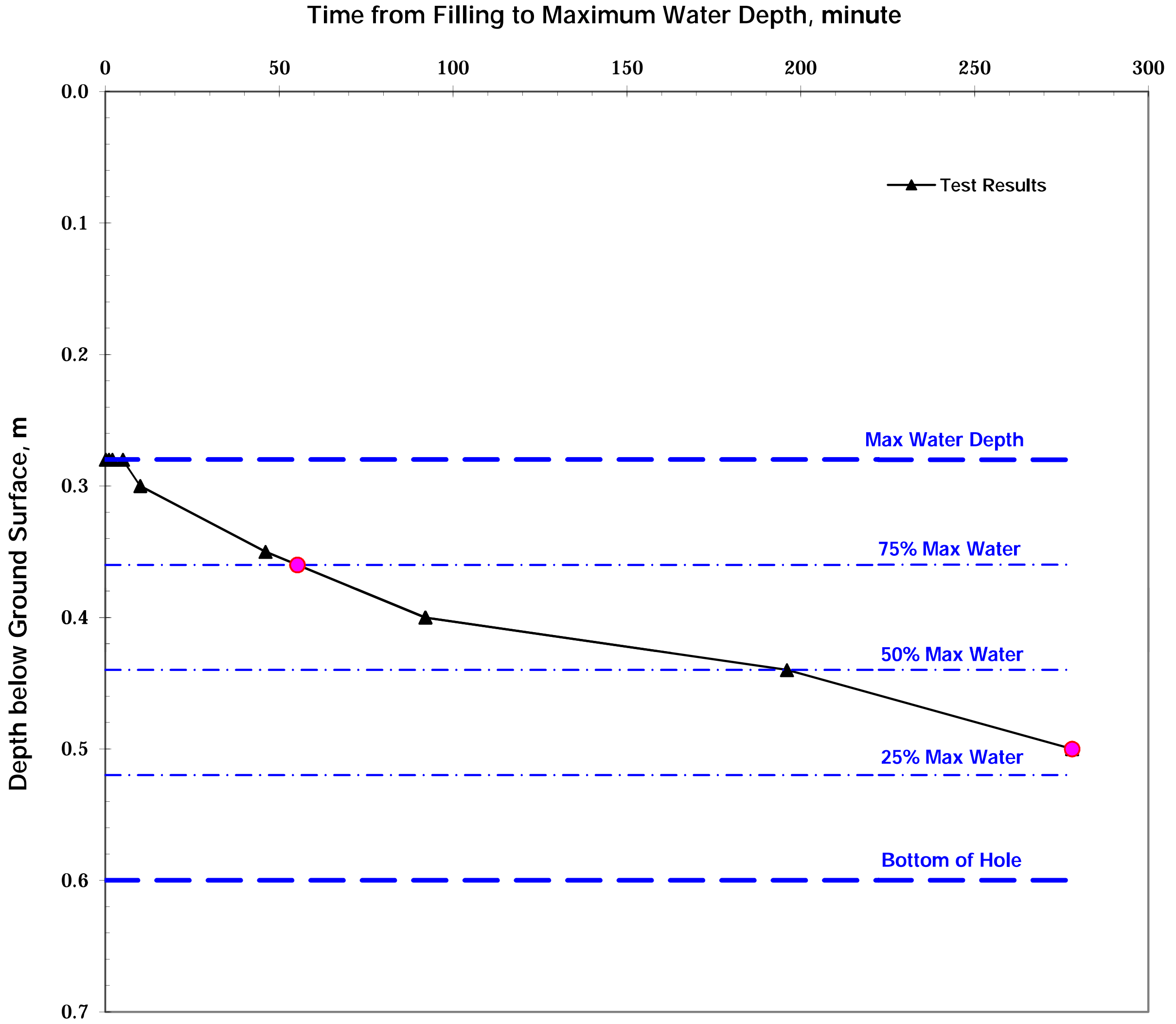
Comments:

Water level fell to 50% -- 25% max water depth, calculations were based on actual fall of water level achieved.
Result not compliant with BRE365 requirement since water did not fall to 25% max water depth.

Client: Hyde Housing Association Ltd	**Job No:** J12765		**Test Date:** 07/Mar/2017		
Site: New Salts Farm, Shoreham	**Tested By:** RA		**Engineer:** AH		**Fig. S1**

BRE Digest 365 Soakage Test

Test Hole No: TP1
Test No: Test No 2 (Repeated)



Pit Length, m	1.200	Depth to Water at Start of Test, m	0.280
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.220
Depth to Pit Base, m	0.600	Total Soakage Test Time, min	278.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	1.332
Depth to Groundwater Surface, m		Discharge Rate, litre/min	0.452
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	0.340
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	5.66E-06

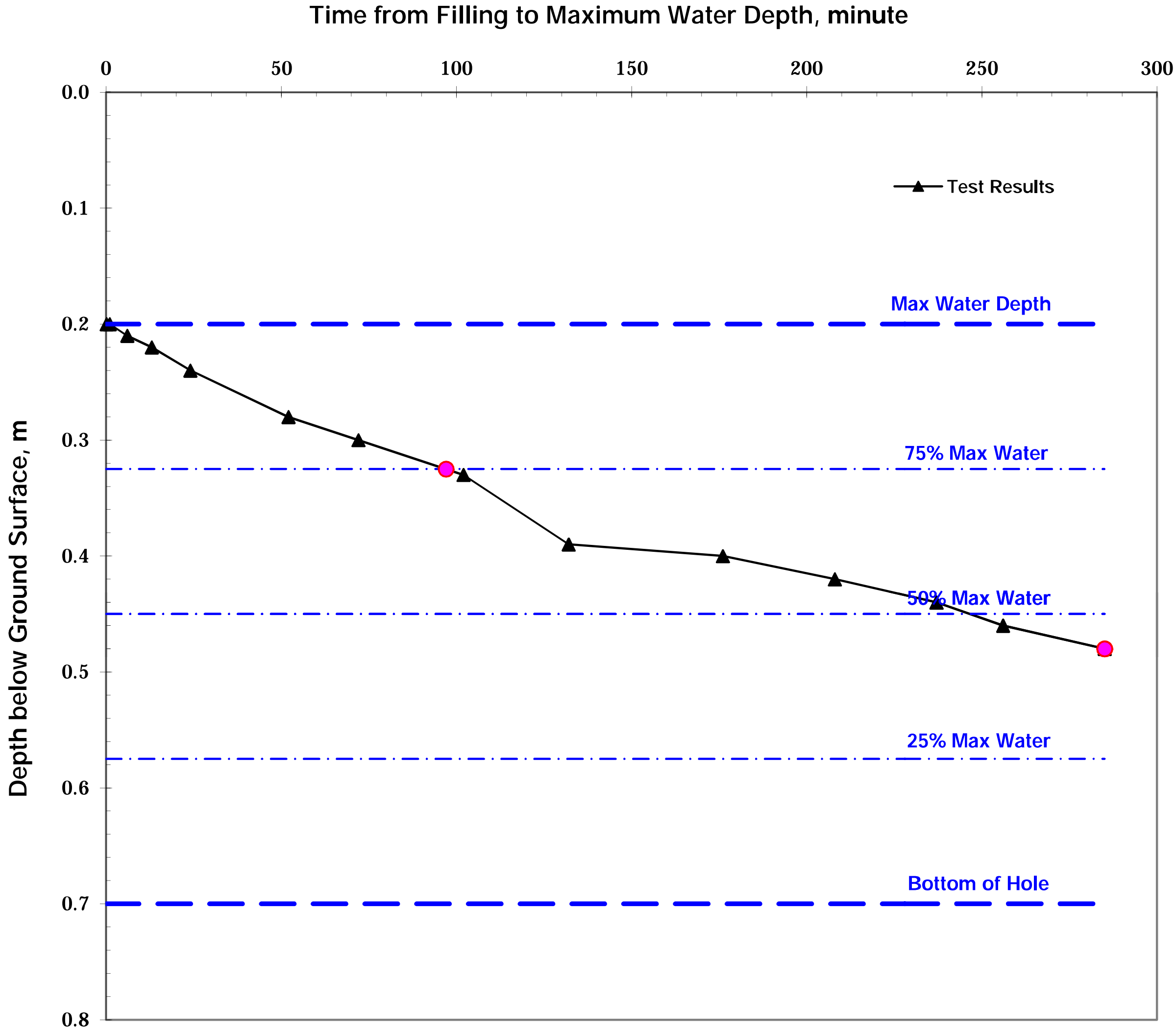
Comments:

Water level fell to 50% -- 25% max water depth, calculations were based on actual fall of water level achieved.
Result not compliant with BRE365 requirement since water did not fall to 25% max water depth.

Client: Hyde Housing Association Ltd	Job No: J12765	Test Date: 08/Mar/2017	
Site: New Salts Farm, Shoreham	Tested By: RA	Engineer: AH	Fig. S2

BRE Digest 365 Soakage Test

Test Hole No: TP2
Test No: Test No 1 (Initial)



Pit Length, m	1.300	Depth to Water at Start of Test, m	0.200
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.280
Depth to Pit Base, m	0.700	Total Soakage Test Time, min	285.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	1.911
Depth to Groundwater Surface, m		Discharge Rate, litre/min	0.643
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	0.337
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	5.61E-06

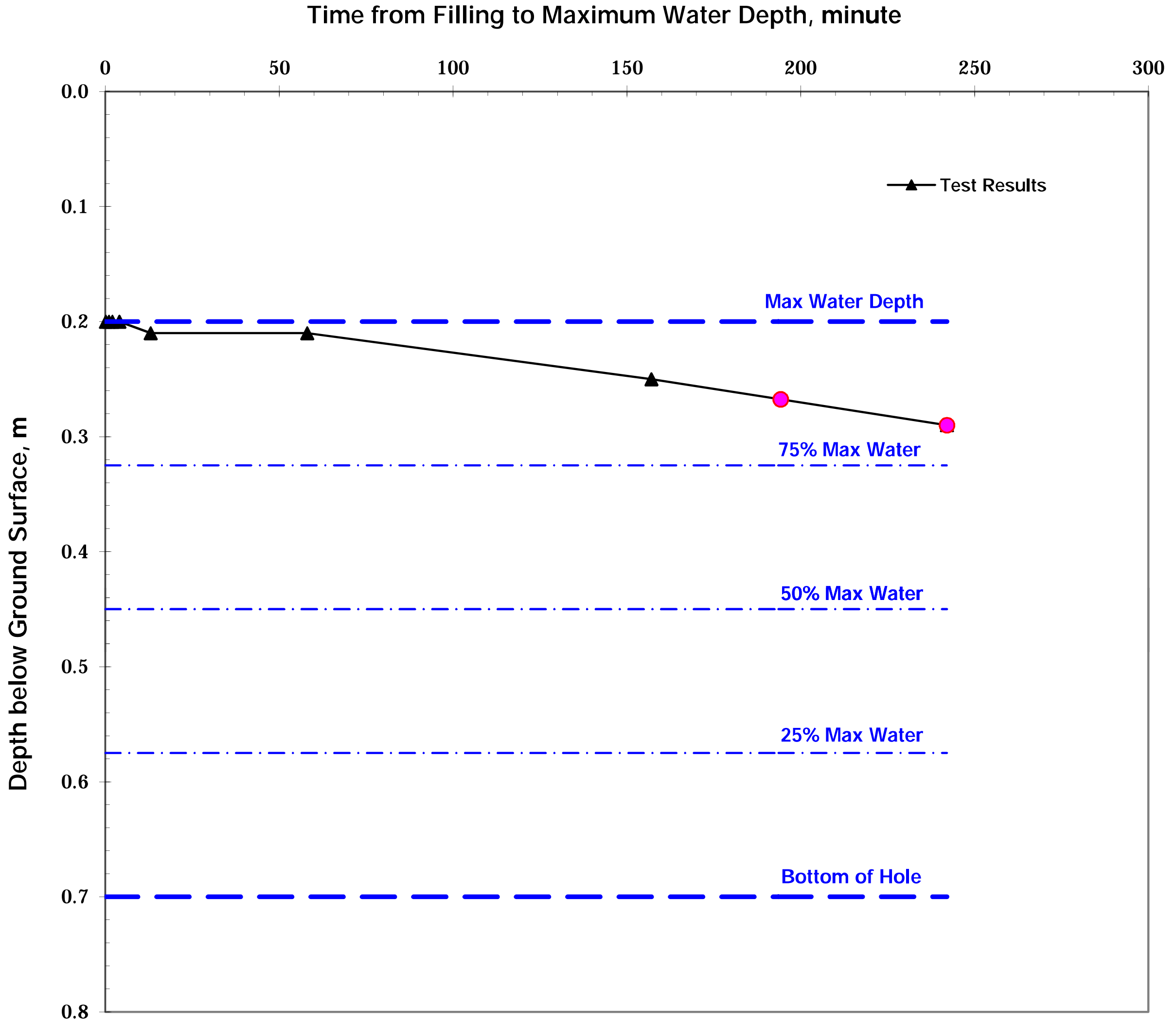
Comments:

Water level fell to 50% -- 25% max water depth, calculations were based on actual fall of water level achieved.
Result not compliant with BRE365 requirement since water did not fall to 25% max water depth.

Client: Hyde Housing Association Ltd	Job No: J12765	Test Date: 07/Mar/2017
Site: New Salts Farm, Shoreham	Tested By: RA	Engineer: AH / Fig. S3

BRE Digest 365 Soakage Test

Test Hole No: TP2
Test No: Test No 2 (Repeated)



Pit Length, m	1.300	Depth to Water at Start of Test, m	0.200
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.090
Depth to Pit Base, m	0.700	Total Soakage Test Time, min	242.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	2.381
Depth to Groundwater Surface, m		Discharge Rate, litre/min	0.367
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	0.154
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	2.57E-06

Comments:

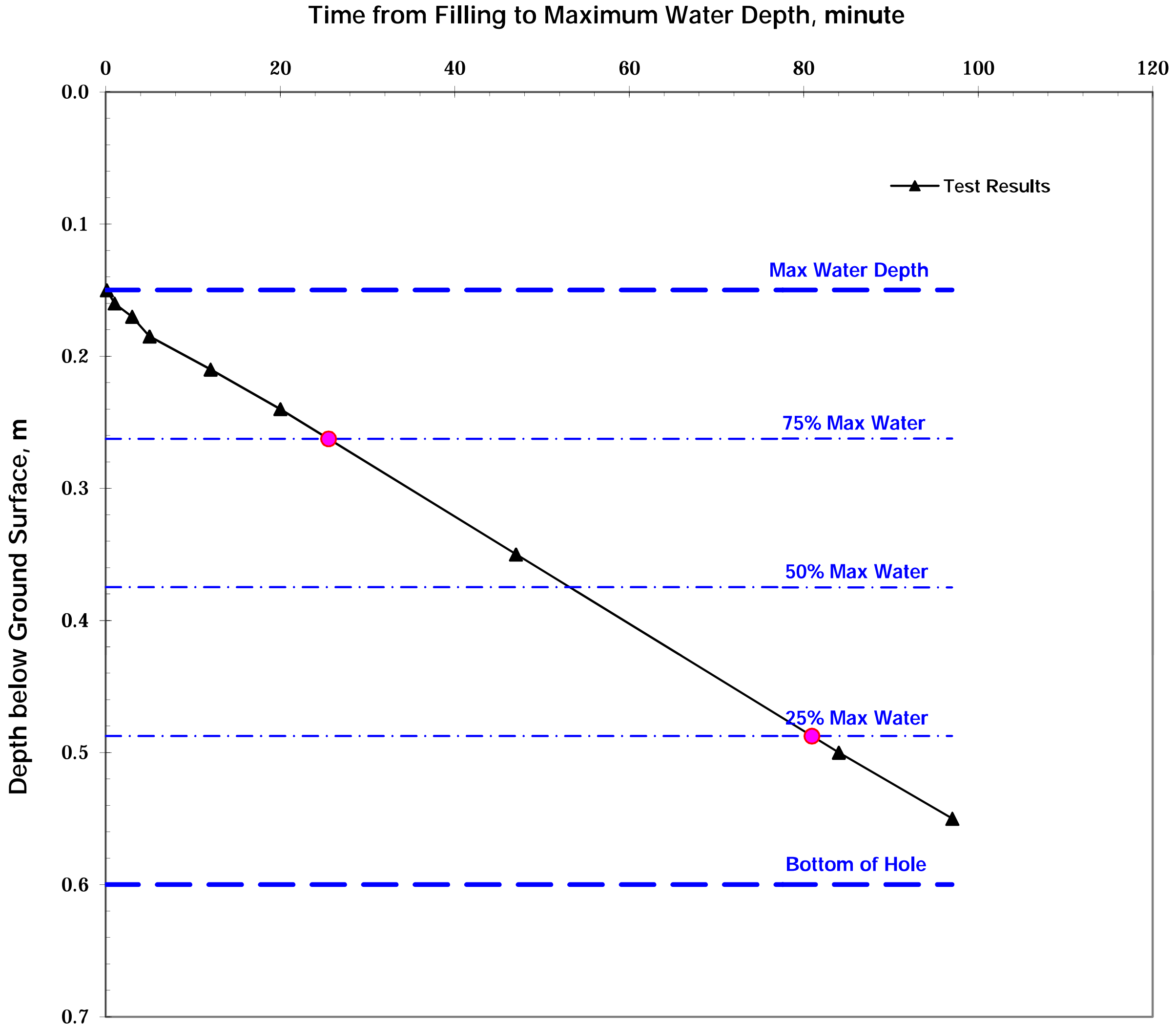
Water level did not fall to 75% max water depth, calculations were based on actual fall of water level achieved.
Result not compliant with BRE365 requirement since water did not fall to 25% max water depth.

Client: Hyde Housing Association Ltd	Job No: J12765	Test Date: 08/Mar/2017	
Site: New Salts Farm, Shoreham	Tested By: RA	Engineer: AH	Fig. S4

BRE Digest 365 Soakage Test

Test Hole No: TP3

Test No: Test No 1 (Initial)



Pit Length, m	1.200	Depth to Water at Start of Test, m	0.150
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.400
Depth to Pit Base, m	0.600	Total Soakage Test Time, min	97.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	1.530
Depth to Groundwater Surface, m		Discharge Rate, litre/min	2.925
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	1.91
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	3.19E-05

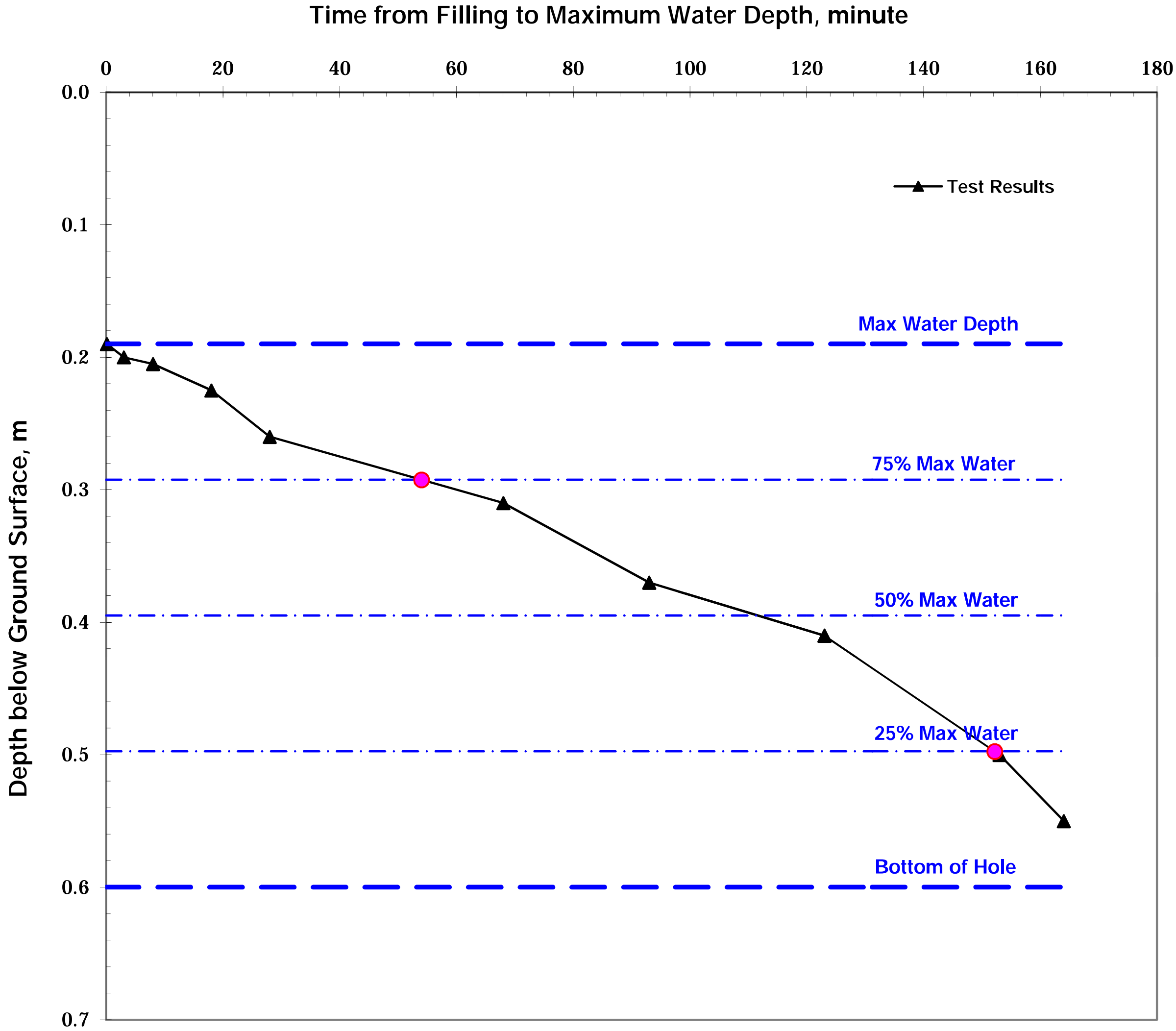
Comments:

Pit was nearly emptied at finish of test.

Client: Hyde Housing Association Ltd	Job No: J12765		Test Date: 07/Mar/2017		
Site: New Salts Farm, Shoreham	Tested By: RA		Engineer: AH		Fig. S5

BRE Digest 365 Soakage Test

Test Hole No: TP3
Test No: Test No 2 (Repeated)



Pit Length, m	1.200	Depth to Water at Start of Test, m	0.190
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.360
Depth to Pit Base, m	0.600	Total Soakage Test Time, min	164.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	1.458
Depth to Groundwater Surface, m		Discharge Rate, litre/min	1.504
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	1.03
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	1.72E-05

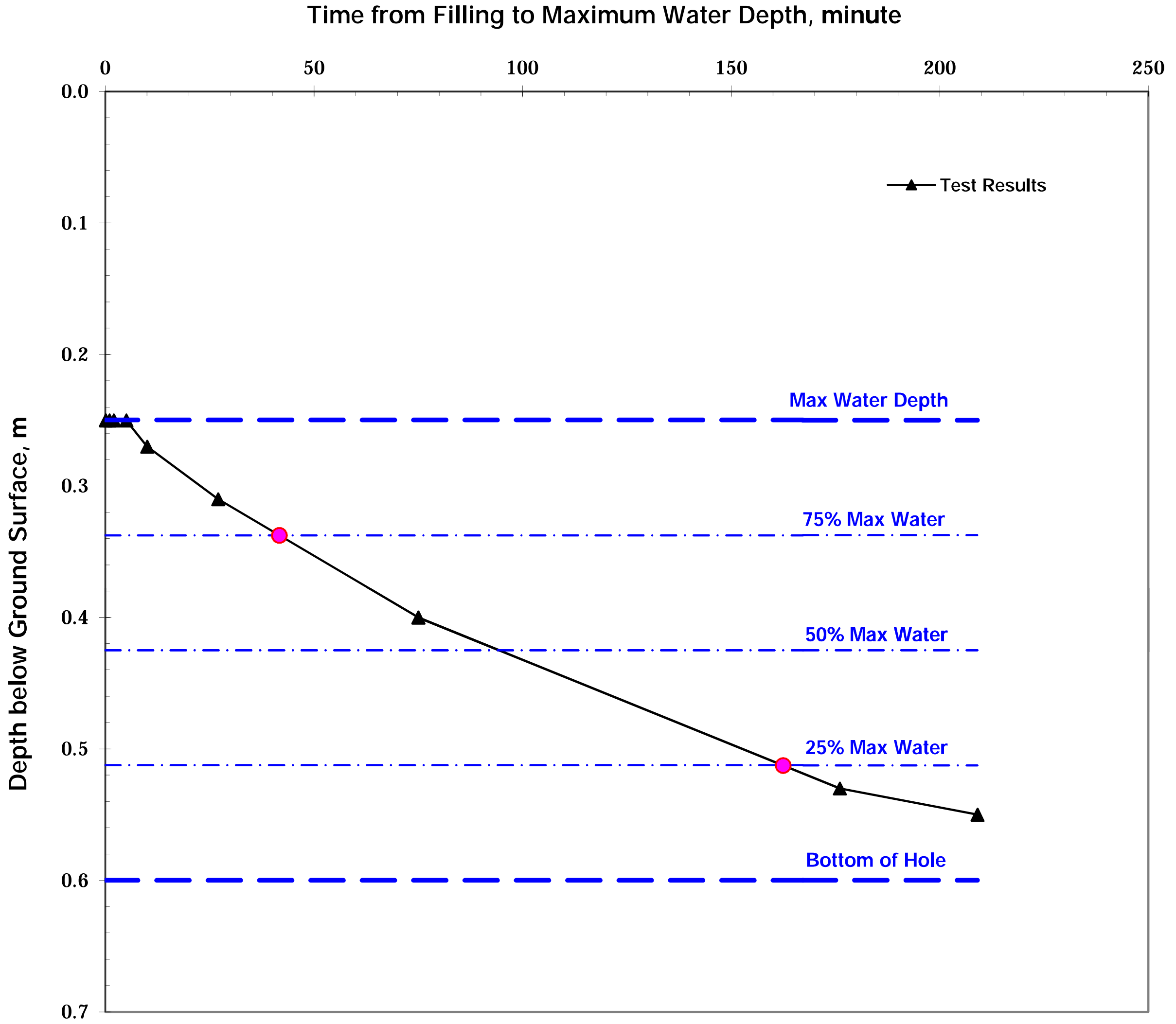
Comments:

Pit was nearly emptied at finish of test.

Client: Hyde Housing Association Ltd	Job No: J12765	Test Date: 07/Mar/2017
Site: New Salts Farm, Shoreham	Tested By: RA	Engineer: AH / Fig. S6

BRE Digest 365 Soakage Test

Test Hole No: TP3
Test No: Test No 3 (Repeated)



Pit Length, m	1.200	Depth to Water at Start of Test, m	0.250
Pit Width, m	0.600	Max Water Dropdown during Test, m	0.300
Depth to Pit Base, m	0.600	Total Soakage Test Time, min	209.0
Depth to Top of Permeable Soils, m		Mean Internal Discharge Area, m^2	1.350
Depth to Groundwater Surface, m		Discharge Rate, litre/min	1.044
Depth to Top of Granular Fill, m		Soakage Rate, litre/m^2/min	0.773
Voids Assumed for Granular Fill, %	100%	BRE Soil Infiltration Rate, m/sec	1.29E-05

Comments:

Pit was nearly emptied at finish of test.

Client: Hyde Housing Association Ltd	Job No: J12765	Test Date: 08/Mar/2017	
Site: New Salts Farm, Shoreham	Tested By: RA	Engineer: AH	Fig. S7

Southern Testing
Environmental & Geotechnical

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Dr J Kelly BSc PhD DIC
Dr L D Mockett BSc PhD PGDip FGS
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A J Timms CEng MICE (Non Executive)
Co. Secretary A L Gurney FCIS ACIB
Consultant Dr Derek Petley PhD DIC BSc MHIT FGS
D Illingworth BSc FGS

Our Ref: AH/BC/J12495

24 February 2016

Hyde Housing
c/o Tully De'Ath Consultants
Sheridan House
Hartfield Road
Forest Row
East Sussex - RH18 5EA

For the attention of Mr Andrew Picton

Dear Sirs,

Re: Groundwater Monitoring Investigation at: New Salts Farm, Shoreham
National Grid Reference: TQ 204 045
Geology: Alluvium/Marine Deposits over Newhaven Chalk

Introduction

The site comprises a grass covered rural plot, and it is proposed to develop the site with housing.

A ground investigation has been undertaken for the proposed development (our ref: J12495) to which the reader is referred. The investigation included the installation of groundwater monitoring wells and dataloggers (instruments used to automatically measure groundwater level), in 3 No. boreholes (WLS4 to 6).

It was also requested that groundwater monitoring wells be installed in the grass covered area to the north and northwest of the proposed development plot (WLS1 to 3), to determine the groundwater regime for this area. During the drilling of the first borehole in this area (WLS 1), the ground conditions were found to comprise running sands, and unfortunately, due to these ground conditions, and having to allow for the borehole to be scanned for UXO, and redrilled, the drilling equipment became 'locked' together, and the installation of groundwater monitoring wells at these positions were not completed.

We returned to site on 10th February to install 3 No. drive-in (hand driven) monitoring wells. These monitoring wells have a narrower diameter than the dataloggers, so the depth to groundwater was manually measured over the course of a day, to determine if the groundwater is influence by the tide.

Presented here are the results thus far from the dataloggers installed in WLS 4 to 6, and the results from a day's monitoring of WLS1 to 3.

Findings

A graph (fig 2) has been produced showing the results of the day's monitoring carried out on WLS1 to 3. It can be clearly seen that groundwater level in both WLS2 and 3 are influenced by the tide. The groundwater level in WLS 1 shows little fluctuation, and is therefore shown to have negligible tidal influence.

The time of high tide at Shoreham-by-Sea Harbour has been shown on the graph. There is a clear time lag between high tide and the peak groundwater level in WLS2 and 3. An approximate time lag of +131min and +163min has been calculated for WLS3 and WLS2 respectively.

It is unclear at this stage why there is no apparent tidal influence at WS1, but this may be due to lower permeability geology.

Graphs (fig 3, 4 and 5) have also been included showing the groundwater level between mid December 2015 and mid February 2016 for WLS 4, 5 and 6. The groundwater level in these trial holes can also be seen to have a tidal influence. From these graphs, the groundwater levels at high tide for WLS 4 and 6 generally fluctuates between around 0.4m and 1.0m below ground level. The groundwater level at high tide in WLS 5 consistently rose above ground level, indicating that groundwater is periodically artesian.

Groundwater during low tide is generally between 1.7m and 2.4m below ground level in WLS 4 and 6, and 1.2m and 1.5m in WLS5.

A graph (fig 6) has also been included that shows the groundwater levels for WLS 4, 5 and 6 over the course of a single day, and the time of high tide at Shoreham-By-Sea Harbour. The time lag between the high tide and the high groundwater in the boreholes is approximately +111min, +106min and +101min for WLS 4, 5 and 6 respectively.

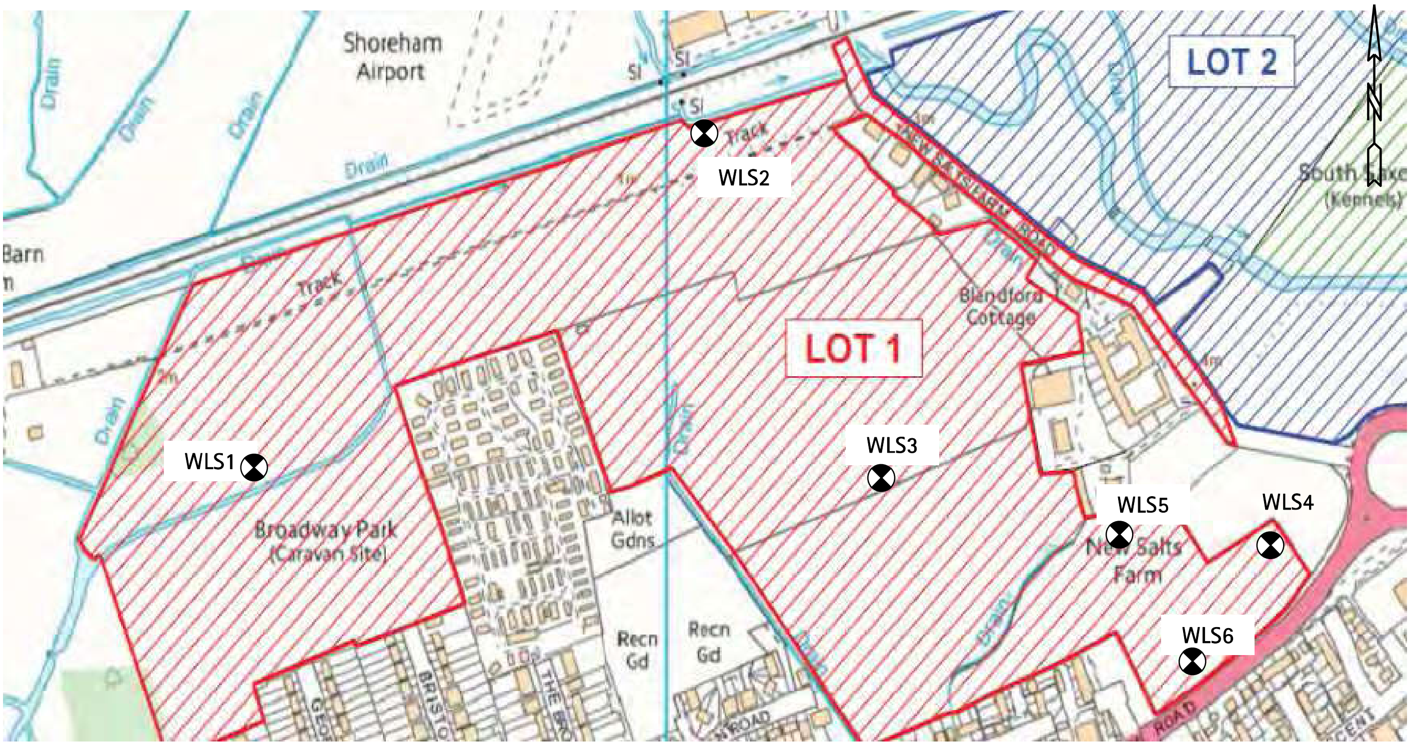
Based on these findings, the ground around WLS5 should be periodically flooded. We have not witnessed, neither are we aware that this area routinely floods, which could be the result of the impermeable clay layer that covers the site preventing the groundwater from rising to the surface. There is therefore, a risk that any development that penetrates this clay 'cap' could form a pathway for groundwater, and thus could instigate flooding.

The influence of storm surges or high spring tides has not been investigated. It is possible that higher groundwater levels than that measured in this investigation, may become evident under certain climatic or tidal conditions.

We hope that the information is useful. Should you require any further information, please do not hesitate to contact us.

Yours faithfully,

Andrew Holley MSc FGS
Senior Geological Engineer
For and on behalf of
Southern Testing Laboratories Limited



NB: Positions of Boreholes and/or Trial Pits are only indicative unless dimensioned

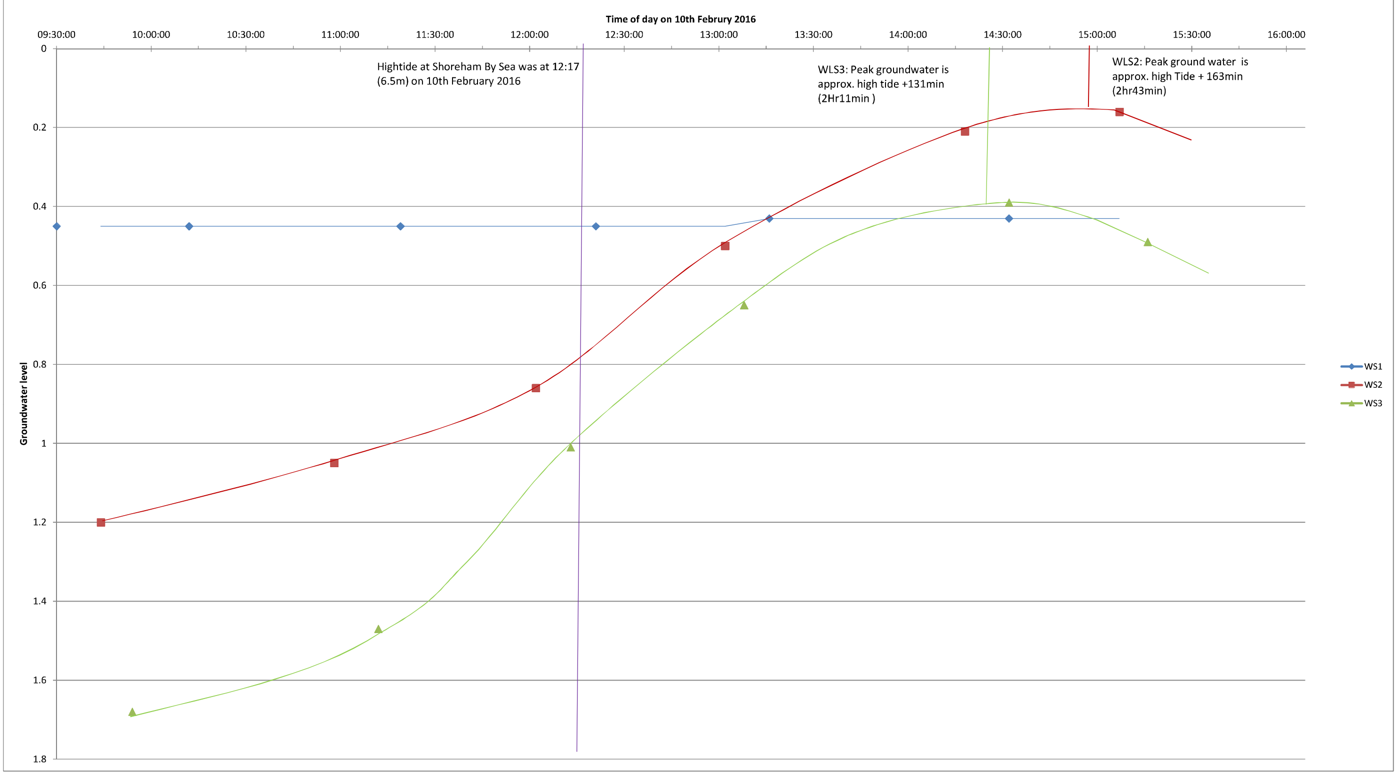
Site: New Salts Farm, Shoreham-By-Sea	STL: J12495	Fig No: 1
Date: 12 February 2016	Trial Hole Location Plan	

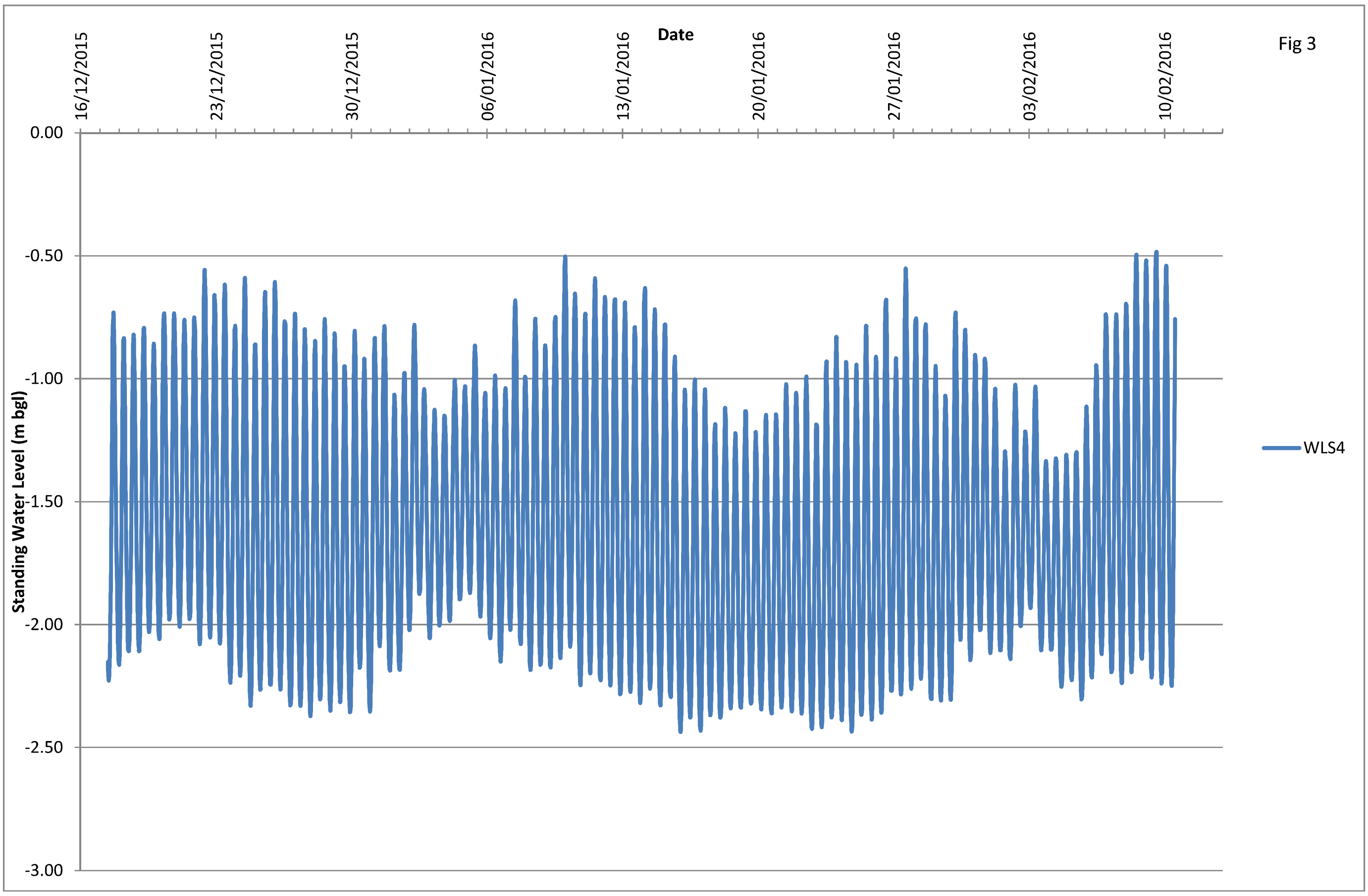
Southern Testing

Southern Testing: Keeble House, Stuart Way, East Grinstead, West Sussex RH19 4QA
ST Consult: Twigden Barns, Brixworth Road, Creaton, Northampton NN6 8NN

ST Consult

Fig. 2: J12495, New Salts Farm
Groundwater level (Measured Below Ground Level)




Date
Fig 3
16/12/2015
23/12/2015
30/12/2015
06/01/2016
13/01/2016
20/01/2016
27/01/2016
03/02/2016
10/02/2016
0.00
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
Standing Water Level (m bgl)
WLS4

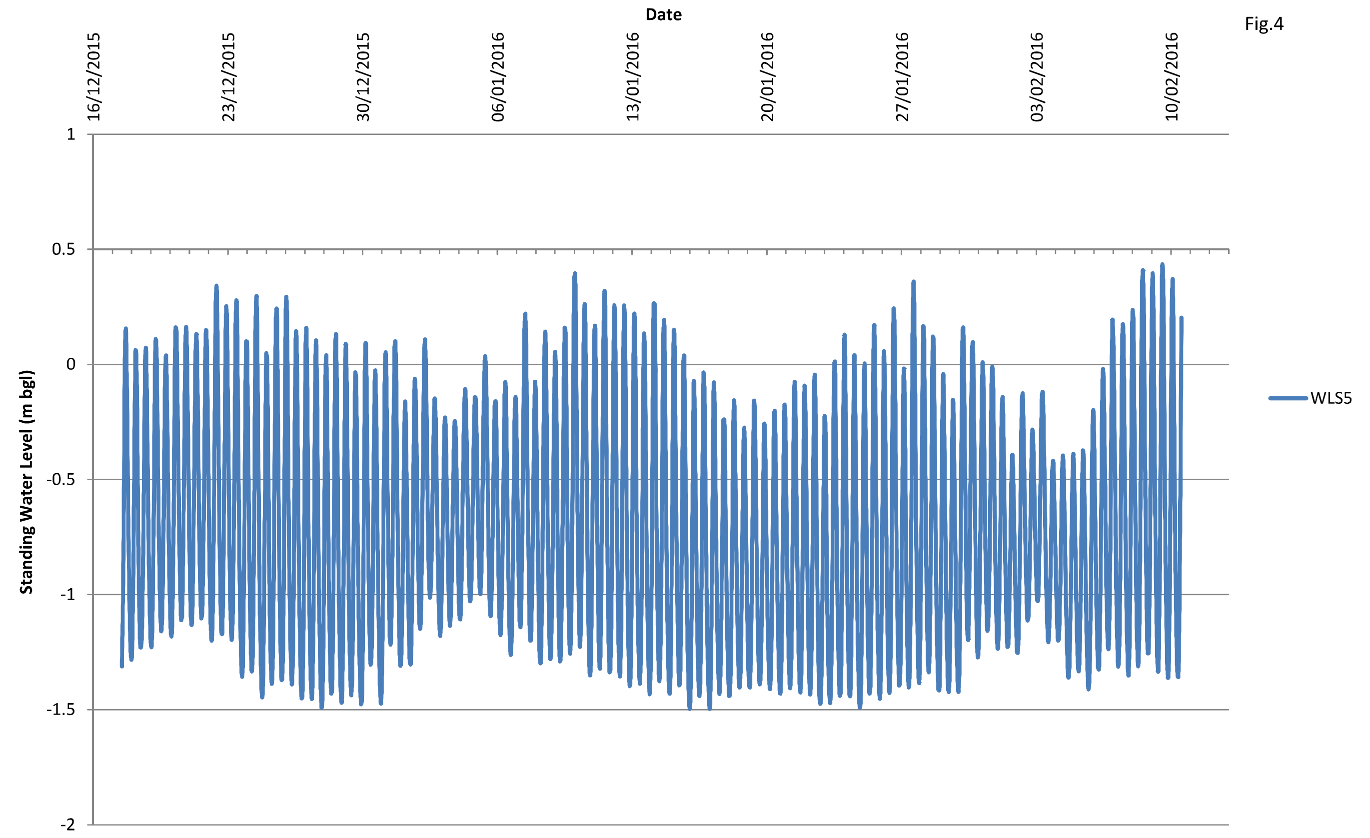
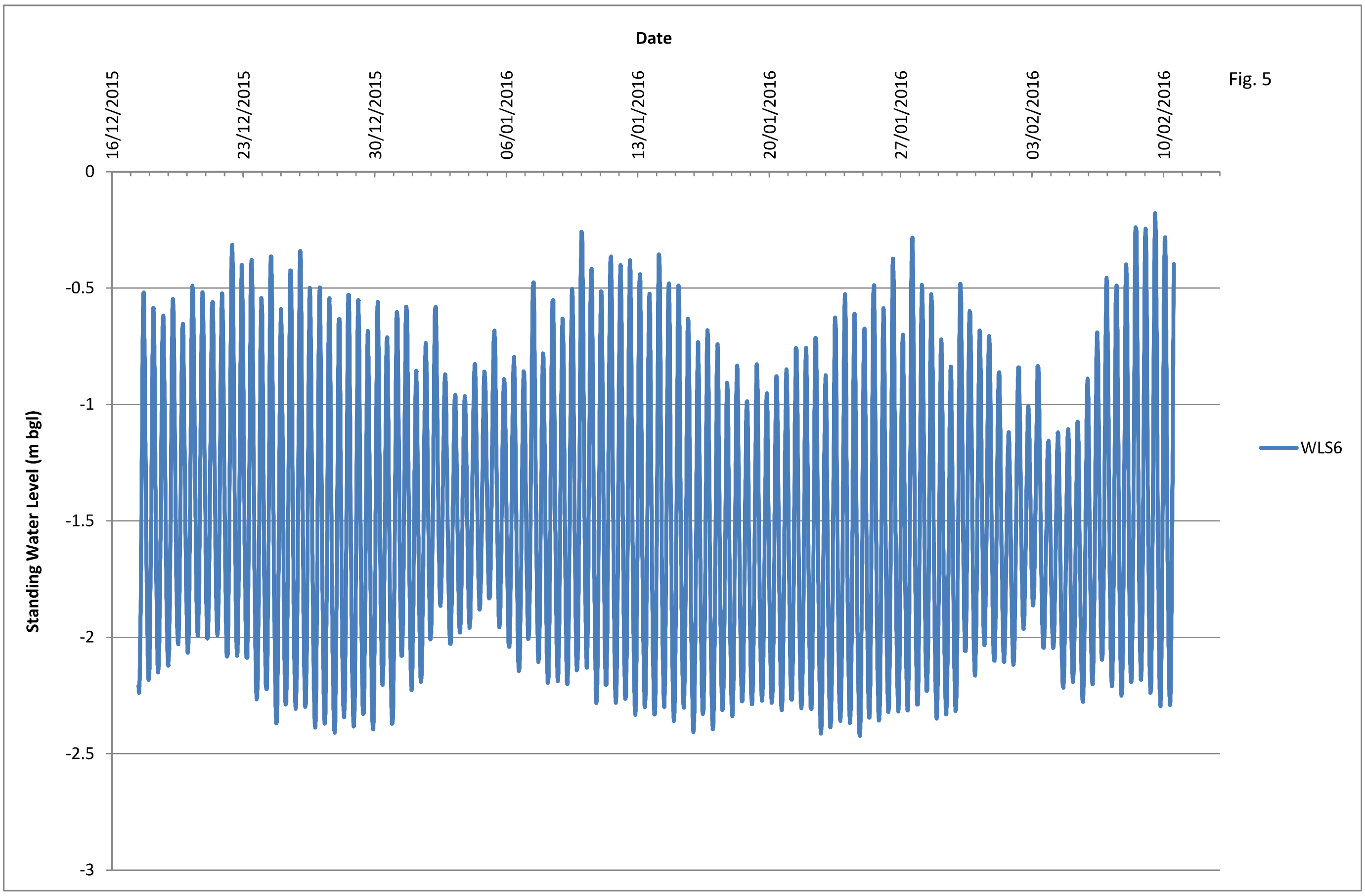

Date
16/12/2015
23/12/2015
30/12/2015
06/01/2016
13/01/2016
20/01/2016
27/01/2016
03/02/2016
10/02/2016
Standing Water Level (m bgl)
1
0.5
0
-0.5
-1
-1.5
-2
WLS5

Fig.4



Date
16/12/2015
23/12/2015
30/12/2015
06/01/2016
13/01/2016
20/01/2016
27/01/2016
03/02/2016
10/02/2016
Standing Water Level (m bgl)
0
-0.5
-1
-1.5
-2
-2.5
-3
WLS6

Fig. 5

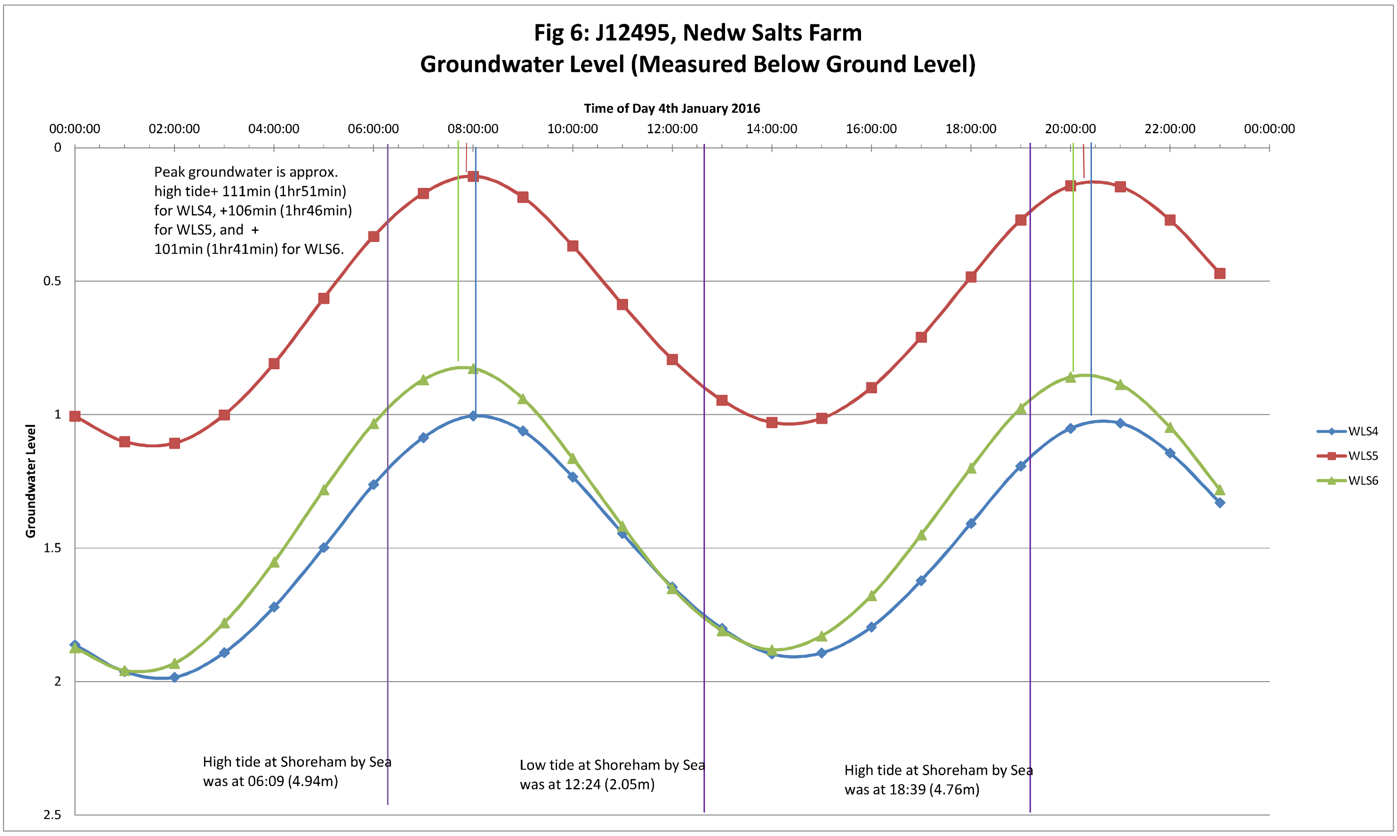
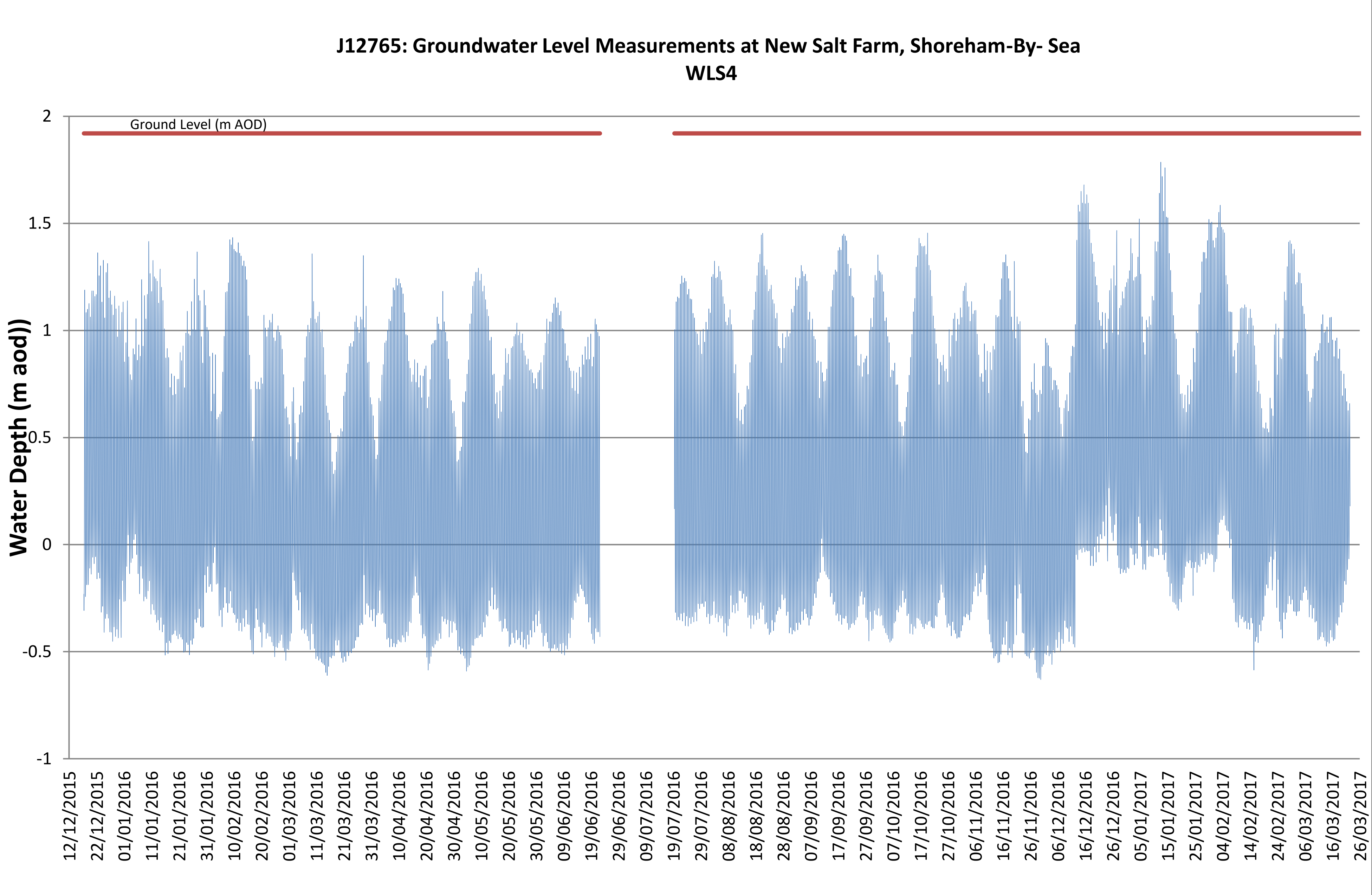


Fig 6: J12495, Nedw Salts Farm
Groundwater Level (Measured Below Ground Level)
Time of Day 4th January 2016
00:00:00
02:00:00
04:00:00
06:00:00
08:00:00
10:00:00
12:00:00
14:00:00
16:00:00
18:00:00
20:00:00
22:00:00
00:00:00
0
0.5
1
1.5
2
2.5
Groundwater Level
Peak groundwater is approx. high tide+ 111min (1hr51min) for WLS4, +106min (1hr46min) for WLS5, and + 101min (1hr41min) for WLS6.
High tide at Shoreham by Sea was at 06:09 (4.94m)
Low tide at Shoreham by Sea was at 12:24 (2.05m)
High tide at Shoreham by Sea was at 18:39 (4.76m)
WLS4
WLS5
WLS6

Southern Testing ST Consult	**Project Name:**	New Salt Farm	**Project ID:**	J12765	Site Plan
	Location:	Shoreham-By-Sea	**Engineer:**	AH	
	Client:	Hyde Housing	**Scale:**	1:5000	

Legend Key

Boreholes - Boreholes


J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS4
Ground Level (m AOD)
Water Depth (m aod))
2
1.5
1
0.5
0
-0.5
-1
12/12/2015
22/12/2015
01/01/2016
11/01/2016
21/01/2016
31/01/2016
10/02/2016
20/02/2016
01/03/2016
11/03/2016
21/03/2016
31/03/2016
10/04/2016
20/04/2016
30/04/2016
10/05/2016
20/05/2016
30/05/2016
09/06/2016
19/06/2016
29/06/2016
09/07/2016
19/07/2016
29/07/2016
08/08/2016
18/08/2016
28/08/2016
07/09/2016
17/09/2016
27/09/2016
07/10/2016
17/10/2016
27/10/2016
06/11/2016
16/11/2016
26/11/2016
06/12/2016
16/12/2016
26/12/2016
05/01/2017
15/01/2017
25/01/2017
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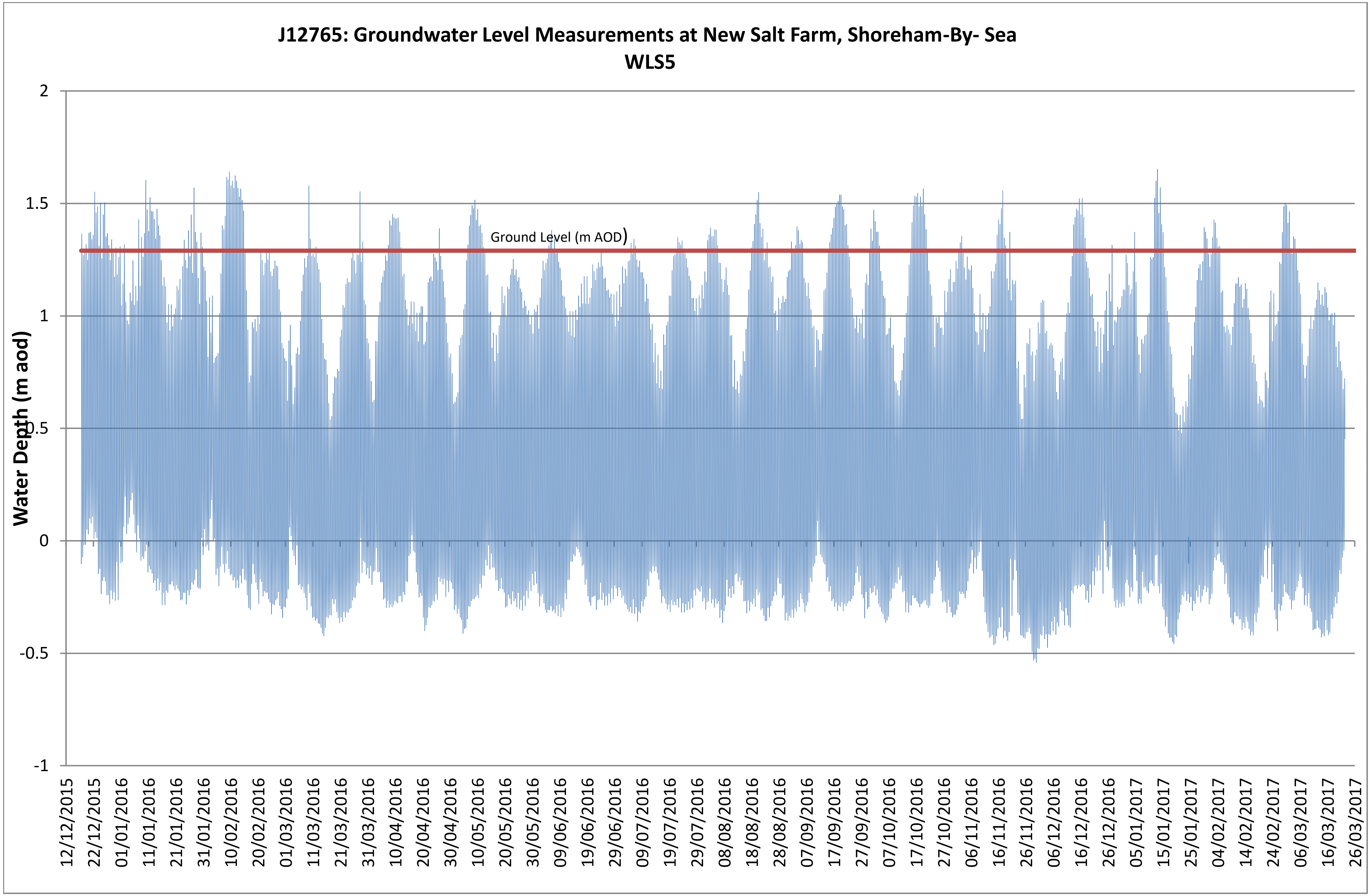
24/02/2017
06/03/2017
16/03/2017
26/03/2017


J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS5
Ground Level (m AOD)
Water Depth (m aod)
2
1.5
1
0.5
0
-0.5
-1
12/12/2015
22/12/2015
01/01/2016
11/01/2016
21/01/2016
31/01/2016
10/02/2016
20/02/2016
01/03/2016
11/03/2016
21/03/2016
31/03/2016
10/04/2016
20/04/2016
30/04/2016
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29/06/2016
09/07/2016
19/07/2016
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18/08/2016
28/08/2016
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06/11/2016
16/11/2016
26/11/2016
06/12/2016
16/12/2016
26/12/2016
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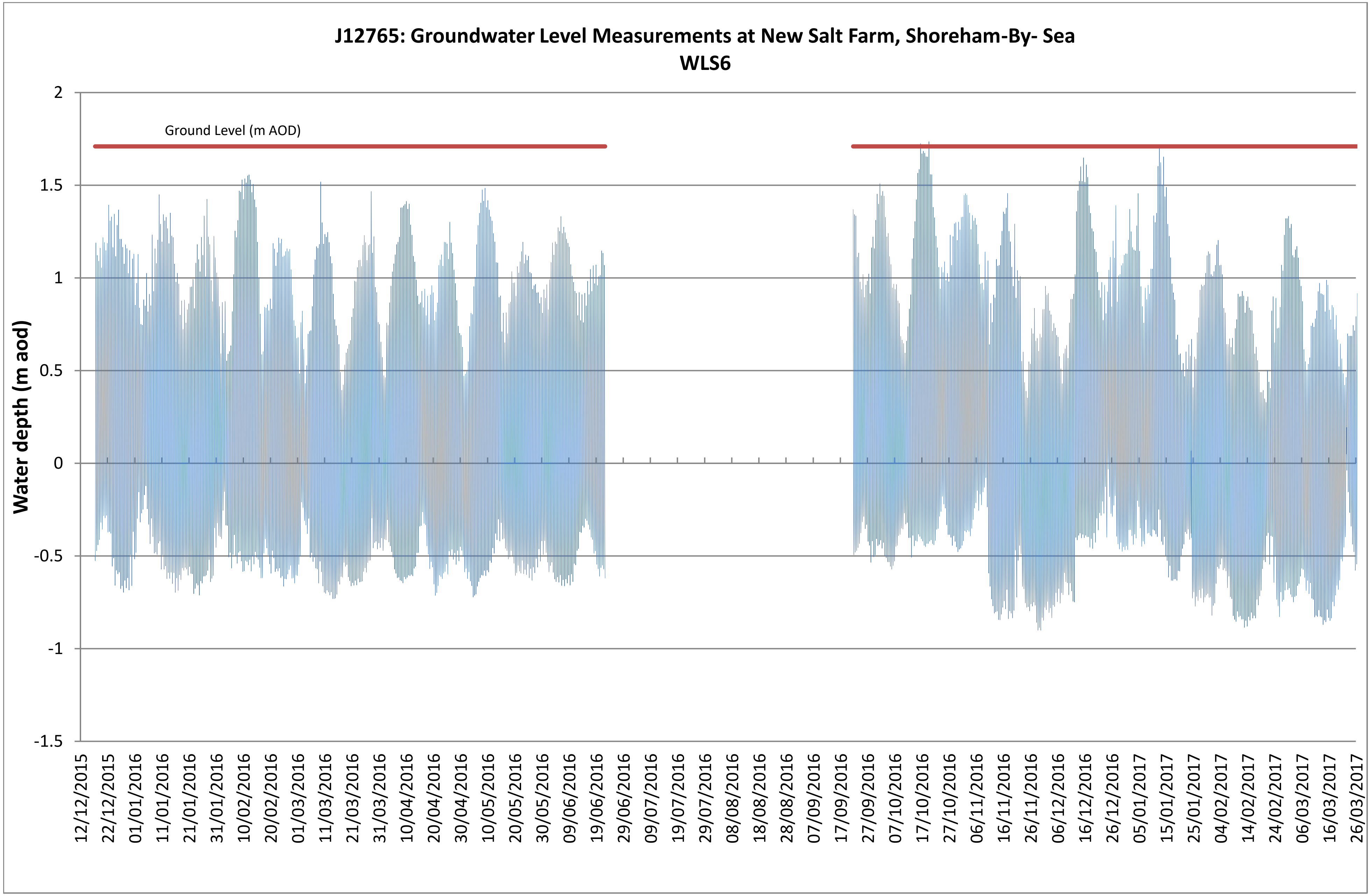
16/03/2017
26/03/2017


J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS6
Ground Level (m AOD)
Water depth (m aod)
2
1.5
1
0.5
0
-0.5
-1
-1.5
12/12/2015
22/12/2015
01/01/2016
11/01/2016
21/01/2016
31/01/2016
10/02/2016
20/02/2016
01/03/2016
11/03/2016
21/03/2016
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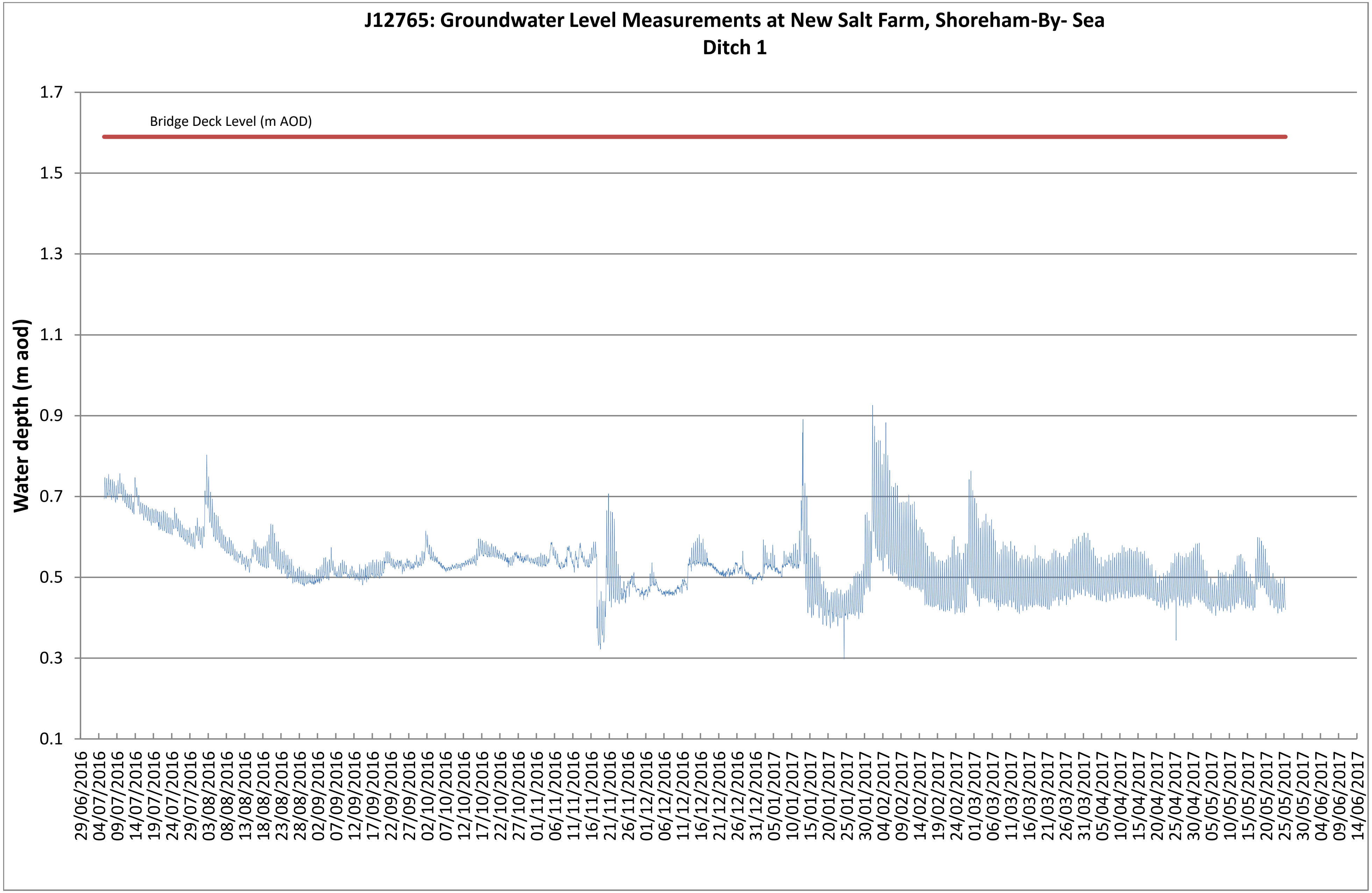
26/03/2017



J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
Ditch 1
Bridge Deck Level (m AOD)
Water depth (m aod)
1.7
1.5
1.3
1.1
0.9
0.7
0.5
0.3
0.1
29/06/2016
04/07/2016
09/07/2016
14/07/2016
19/07/2016
24/07/2016
29/07/2016
03/08/2016
08/08/2016
13/08/2016
18/08/2016
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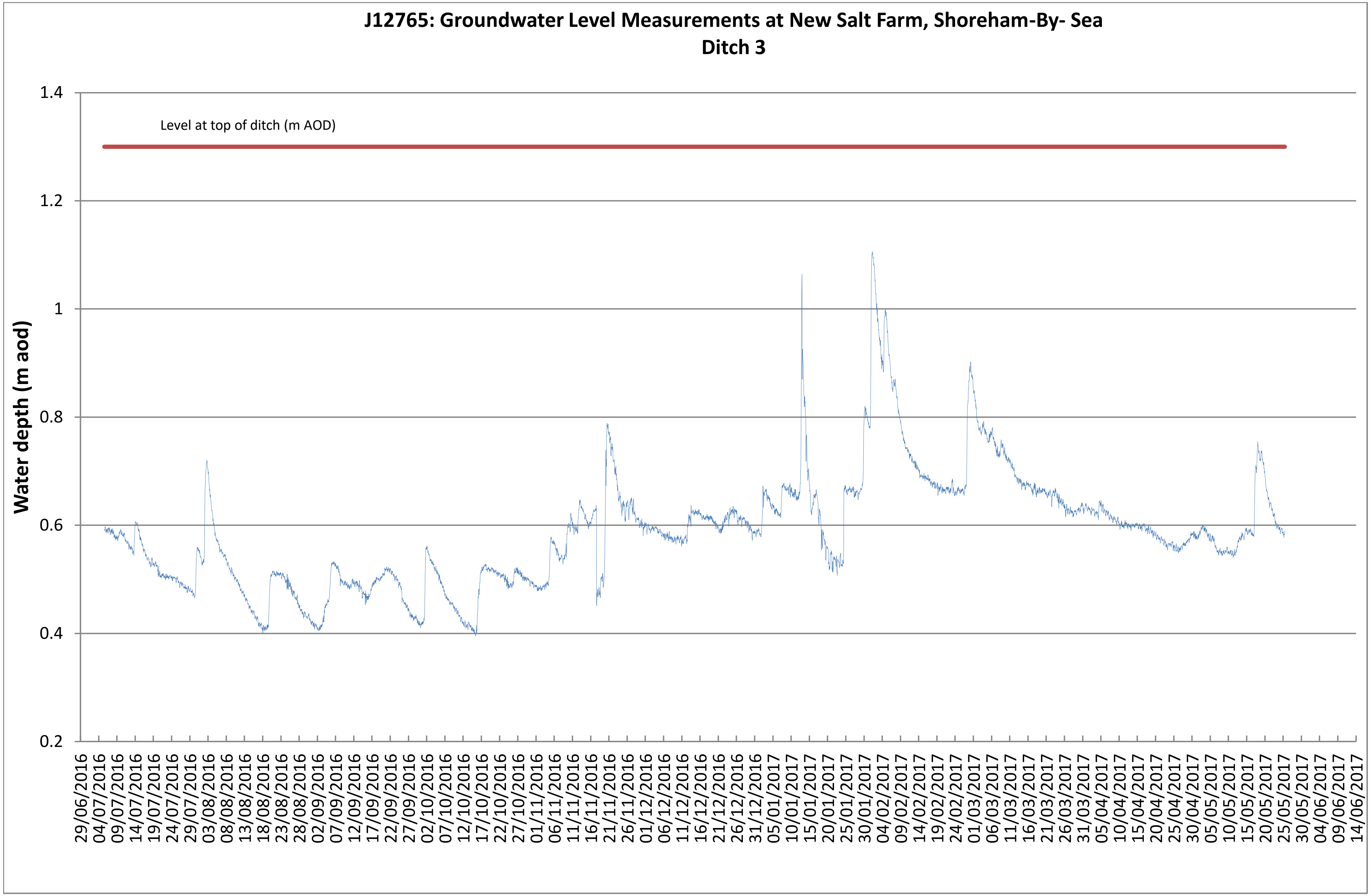

J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
Ditch 3
Level at top of ditch (m AOD)
Water depth (m aod)
1.4
1.2
1
0.8
0.6
0.4
0.2
29/06/2016
04/07/2016
09/07/2016
14/07/2016
19/07/2016
24/07/2016
29/07/2016
03/08/2016
08/08/2016
13/08/2016
18/08/2016
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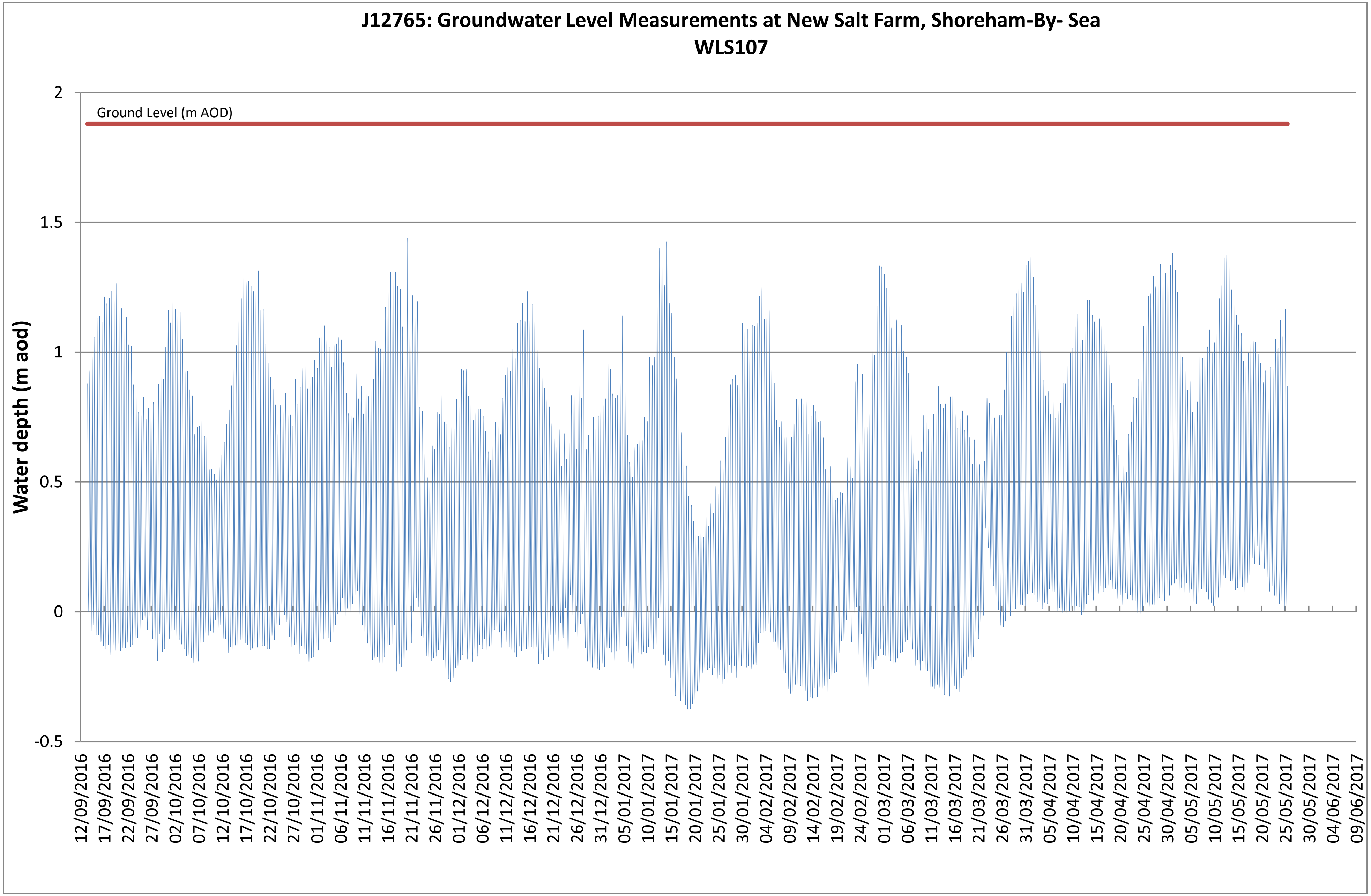
J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS107
Ground Level (m AOD)
Water depth (m aod)
2
1.5
1
0.5
0
-0.5
12/09/2016
17/09/2016
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27/09/2016
02/10/2016
07/10/2016
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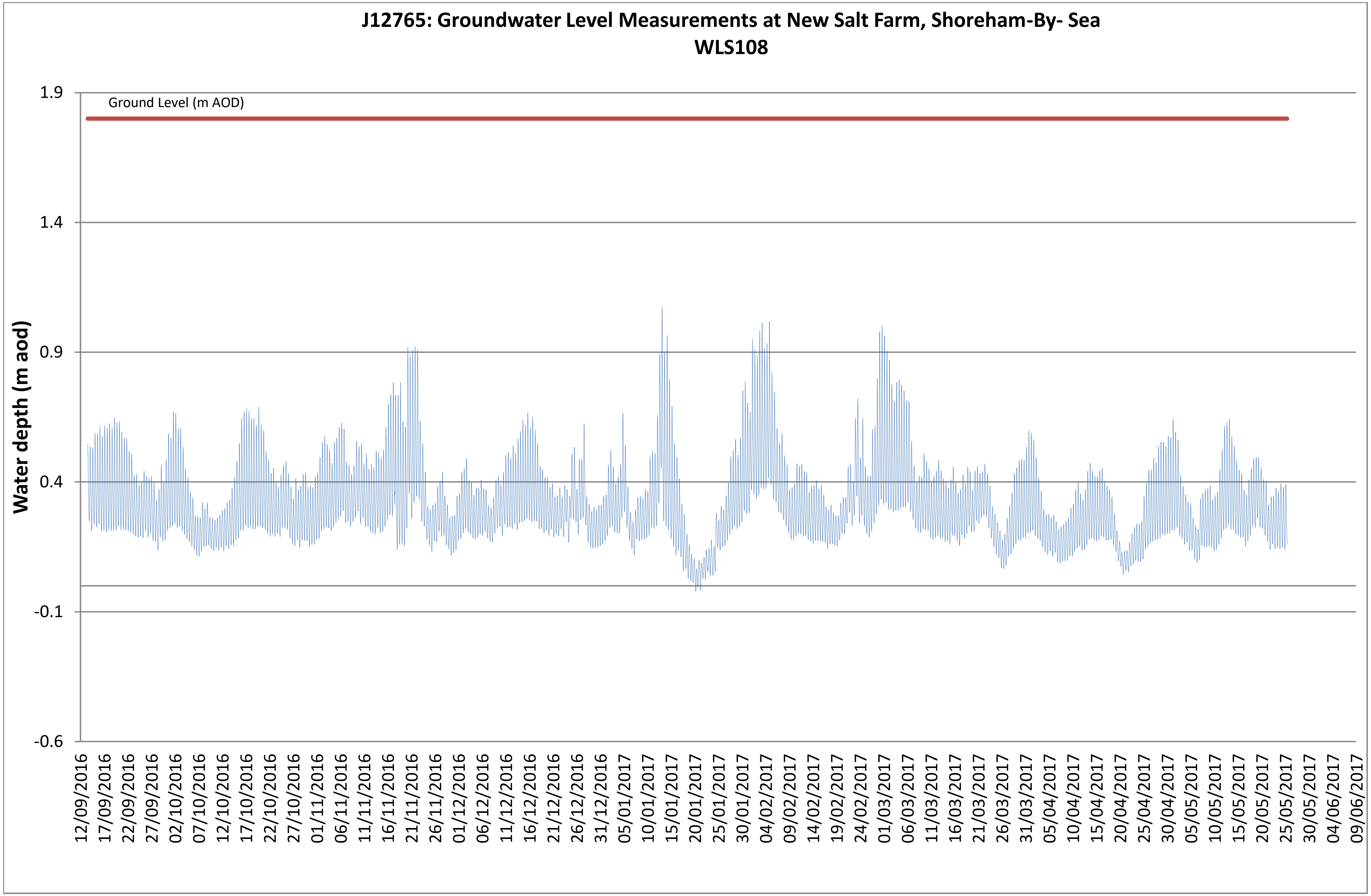

J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS108
Ground Level (m AOD)
Water depth (m aod)
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1.4
0.9
0.4
-0.1
-0.6
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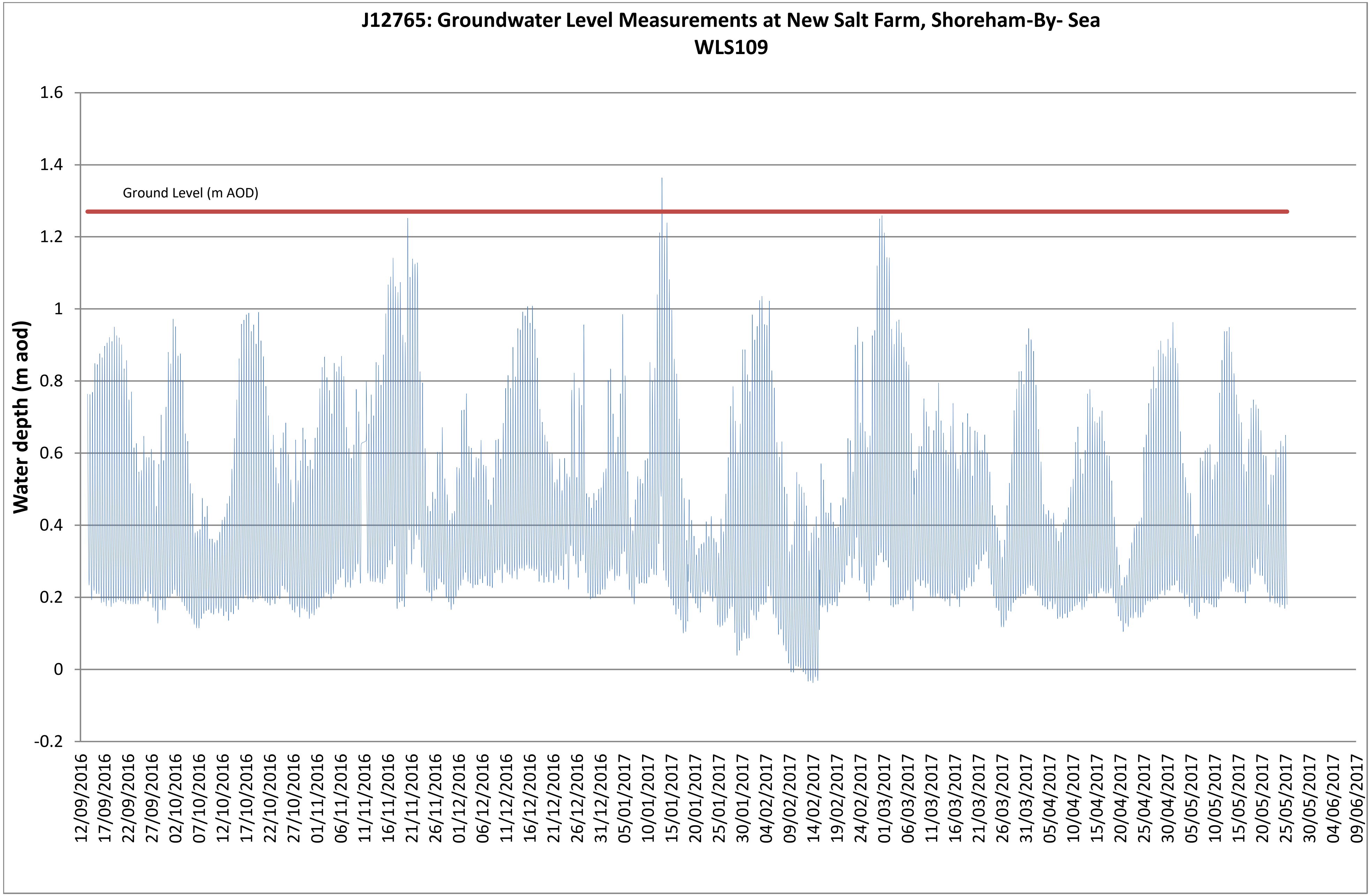
J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS109
Ground Level (m AOD)
Water depth (m aod)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
12/09/2016
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27/09/2016
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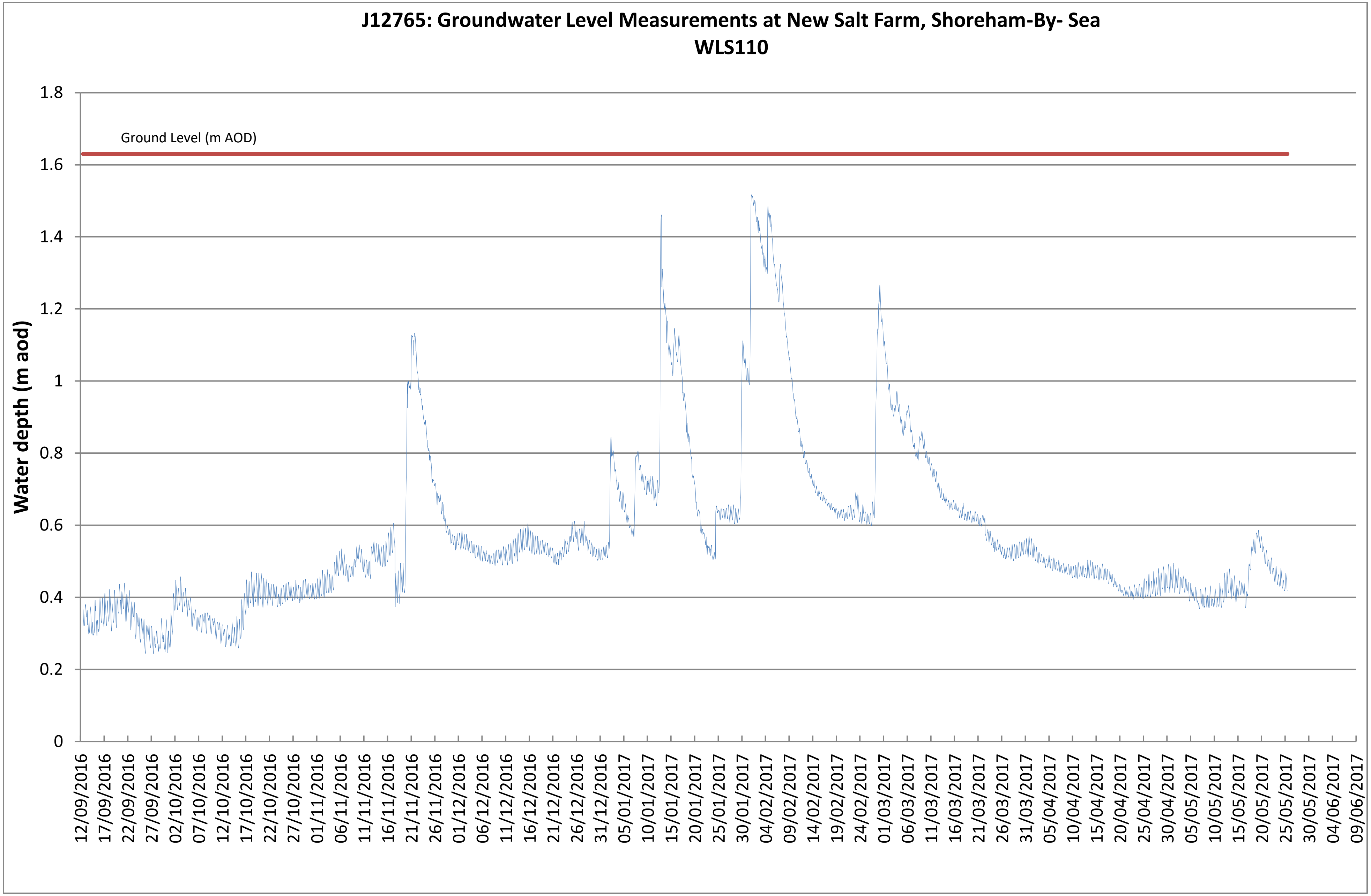

J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS110
Ground Level (m AOD)
Water depth (m aod)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
12/09/2016
17/09/2016
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27/09/2016
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07/10/2016
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17/10/2016
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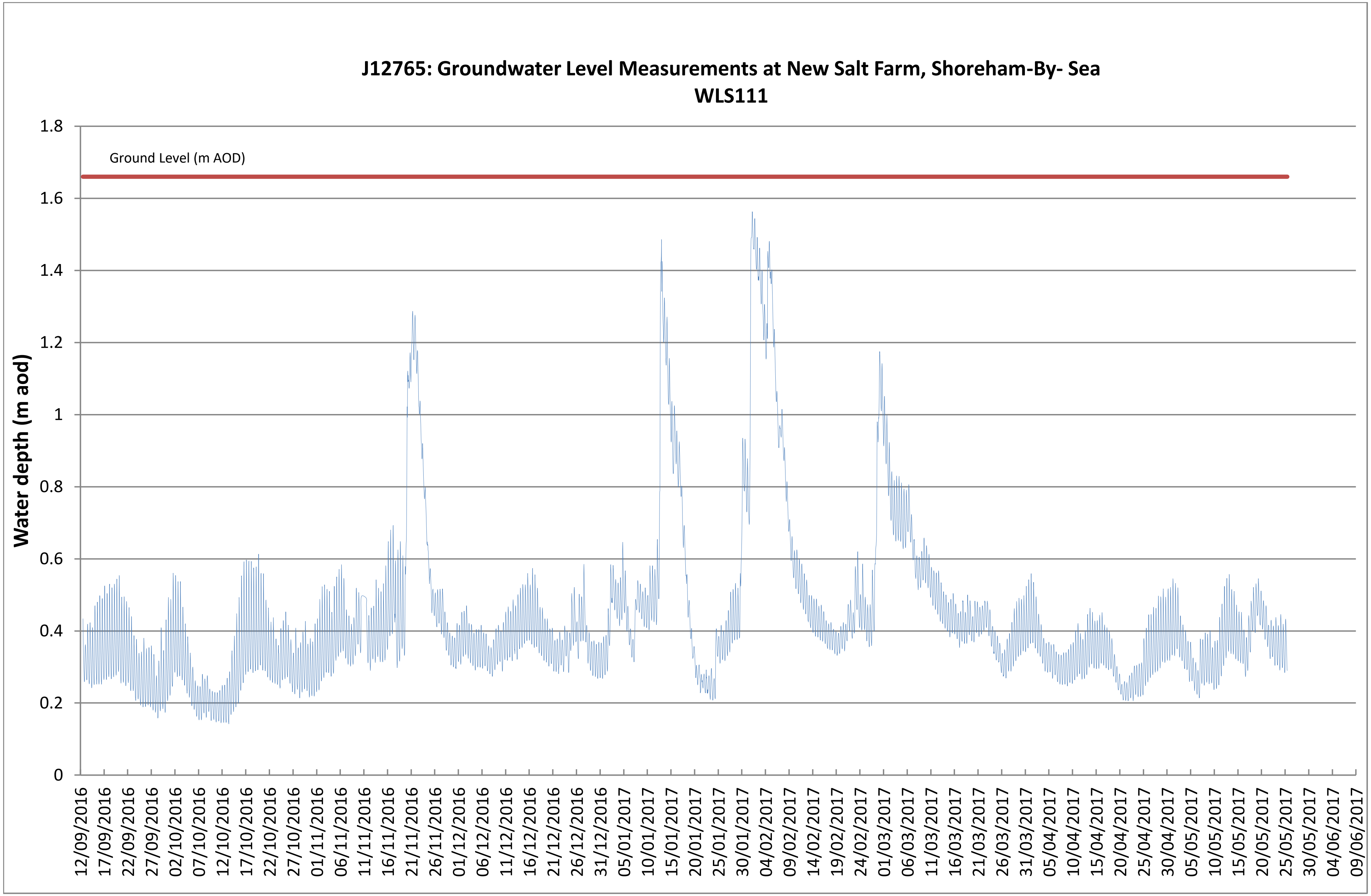
J12765: Groundwater Level Measurements at New Salt Farm, Shoreham-By- Sea
WLS111
Ground Level (m AOD)
Water depth (m aod)
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
12/09/2016
17/09/2016
22/09/2016
27/09/2016
02/10/2016
07/10/2016
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17/10/2016
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27/10/2016
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30/05/2017
04/06/2017
09/06/2017

Appendix E – Development Proposals

B	26.08.16	Phase 1 updated. Air corridor lines removed.	IS
A	26.08.16	Air corridor added. Site Layout amended to avoid the air corridor.	IS

REVISIONS

NOTES:

FOR COMMENT

0 10 15 20 30 40 50m 60 70 80 90 100m

N

CLIENT

The Hyde Group
Making a lasting difference

TITLE

NEW SALTS FARM
PROPOSED MASTERPLAN

HGP ARCHITECTS

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Date: © FEBRUARY 2016
Drawn by: CK
Checked by: GG
Drawing No: 15.003_010
Revision: E

Appendix F –Sequential and Exception Test

Sequential & Exception Test

New Salts Farm

Boyer

Prepared on behalf of Hyde New Homes | December 2016

Report Control

Project: New Salts Farm	
Client: The Hyde Group	
Reference: 14.455	
File Origin:	http://lucas/sites/Boyerplanning/twickenham/14.455/4 Boyer Planning/4.02 Reports/EiP/FRA and SandE/161205 SandE Test Whole Site Update FINAL.docx
Primary Author: Dinny Shaw	
Checked By: Andrew Williams	

Issue	*Date*	*Status*	*Checked By*
1	16/12/2016	DRAFT	Andrew Williams
2	16/12/2016	FINAL	Dinny Shaw

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This report has been prepared by Boyer on behalf of The Hyde Group in support of the redevelopment of their site at New Salts Farm for residential development. It is considered that the site could deliver around 455 new homes along with associated car parking and landscaping. The site is located within Flood zone 3a and part within 3b.

This report comprises a Sequential and Exception Test for the site to demonstrate that both tests have been passed and the site is suitable for residential development.

Adur District Council is not meeting its objectively assessed housing need in the Emerging Adur Local Plan (Amendments to the Proposed Submission Adur Local Plan 2016). It is therefore necessary for additional housing to be delivered within the plan period to meet housing need.

The Sequential Test has considered alternative sites within Adur District, having regard to the Strategic Housing Land Availability Assessment 2014 (CD04/11) and Update 2016 (CD07/20). The report has found that there were no other sites within Adur District of a similar capacity which could provide the development proposed at New Salts Farm and which would fall into an area with a lower probability of flooding. Therefore the Sequential Test has been passed.

On the 1st June 2016 planning permission was approved for Shoreham Adur Tidal Walls scheme, a scheme for improved flood defences in the River Adur. When implemented these will have a positive impact at the New Salts Farm site by partly addressing concerns regarding tidal and fluvial flooding. It would also redesignate those parts of the site within Flood Zone 3b as Flood Zone 3a.

The Exception Test has taken the Shoreham Adur Tidal Walls scheme into account. It comprises a review of the development site against the sustainability objectives of the Emerging Adur Local Plan Sustainability Appraisal (October 2016) (CD07/02). The results show that the development site scores positively in regard to the sustainability objectives and therefore would provide wider sustainability benefits to the community that outweigh flood risk, in particular the provision of new homes. The second part sets out what measures could be included in the development to manage and mitigate flood risk to demonstrate that it could remain safe for its lifetime, supported by a site specific Flood Risk Assessment prepared by Tully De'Ath. Therefore the two parts of the Exception Test have also been passed.

The Sequential and Exception Tests have been carried out in accordance with the National Planning Policy Framework and Planning Practice Guidance. It has demonstrated that the proposed development would pass both the Sequential and Exception Tests and therefore can be considered suitable for residential development.

1. INTRODUCTION

1.1 This report has been prepared by Boyer on behalf of The Hyde Group in support of the redevelopment of their site at New Salts Farm (Figure 1) for residential development.

Figure 1 - New Salts Farm Illustrative Masterplan

1.2 It is considered that the site could accommodate approximately 455 dwellings together with associated car parking, open space and landscaping, as demonstrated through the illustrative masterplan, and would represent a positive and beneficial contribution towards meeting housing need in Adur District.

1.3 The site is bounded by New Salts Farm road to the east, the railway to the north and Shoreham Airport beyond and existing residential properties, Broadway Park and Brighton Road to the south. It is within Flood Zones 3a and 3b.

1.4 The Council has previously raised concerns over flood risk issues at the site, and a lack of evidence to demonstrate that these can be overcome. The site was excluded from the Council's own Sequential and Exception Test (CD06/10) for the Emerging Adur Local Plan on that basis. We have therefore prepared a site specific Sequential and Exception Test for the development site.

1.5 This report relates to the whole New Salts Farm site for development of 455 homes.

2. POLICY CONTEXT

2.1 The National Planning Policy Framework (NPPF) states at paragraph 100 that:

'inappropriate development in areas at risk of flooding should be avoided by directing development away from areas at highest risk, but where development is necessary, making it safe without increasing flood risk elsewhere'.

2.2 Paragraph 101 continues saying that:

'the aim of the Sequential Test is to steer new development to areas with the lowest probability of flooding. Development should not be allocated or permitted if there are reasonably available sites appropriate for the proposed development in areas with a lower probability of flooding.'

2.3 Para 102 of the NPPF states that:

'if, following application of the Sequential Test, it is not possible, consistent with wider sustainability objectives for the development to be located in zones with a lower probability of flooding, the Exception Text can be applied if appropriate. For the Exception Test to be passed:

it must be demonstrated that the development provides wider sustainability benefits to the community that outweigh flood risk, informed by a Strategic Flood Risk Assessment where one has been prepared; and

a site specific flood risk assessment must demonstrate that the development will be safe for its lifetime taking account of the vulnerability of its users, without increasing flood risk elsewhere, and where possible will reduce flood risk overall.'

2.4 Further guidance at paragraph 103 states that:

'When determining planning applications, local planning authorities should ensure flood risk is not increased elsewhere and only consider development appropriate in areas at risk of flooding where, informed by a site-specific flood risk assessment following the Sequential Test, and if required the Exception Test, it can be demonstrated that:

within the site, the most vulnerable development is located in areas of lowest flood risk unless there are overriding reasons to prefer a different location; and

development is appropriately flood resilient and resistant, including safe access and escape routes where required, and that any residual risk can be safely managed, including by emergency planning; and it gives priority to the use of sustainable drainage systems.'

2.5 Paragraph 033 of the Planning Practice Guidance (PPG) provides further guidance to the application of the Sequential Test. It states that:

'the area to apply the Sequential Test across will be defined by local circumstances relating to the catchment area for the type of development proposed'. It goes on to say that 'when applying the Sequential Test, a pragmatic approach on the availability of alternatives should be taken'.

2.6 Paragraph 023 of the PPG provides guidance on the Exception Test and states that:

'Essentially, the two parts to the test require proposed development to show that it will provide wider sustainability benefits to the community that outweigh flood risk, and that it will be safe for its lifetime, without increasing flood risk elsewhere and where possible reduce flood risk overall.'

2.7 The PPG defines the flood risk vulnerability classifications of which residential development falls within the 'More Vulnerable' classification.

2.8 Table 1 below sets out the flood risk vulnerability and Flood Zone compatibility:

Table1 – Flood Risk Classification

Flood Zones	Flood Zone – Vulnerability Classification				
	Essential Infrastructure	Highly Vulnerable	More Vulnerable	Less Vulnerable	Water Compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test Required	✓	✓	✓
Zone 3a	Exception Test Required	X	Exception Test Required	✓	✓
Zone 3b	Exception Test Required	X	X	X	✓

2.9 This report has been prepared in accordance with the guidance contained in the NPPF and PPG.

3. THE SEQUENTIAL TEST

Background

3.1 The purpose of the Sequential Test is to demonstrate that there are no sequentially preferable available sites at a lower flood risk within a defined search area which could deliver the proposed development.

Housing Target

3.2 In previous years the South East Plan set the housing requirement for Adur, however this was revoked in March 2013. The National Planning Policy Framework now requires that Local Planning Authorities use their evidence base to ensure that their Local Plan meets the full, objectively assessed needs (OAN) for market and affordable housing in the housing market area, as far as is consistent with the policies set out in the Framework.

3.3 The Statutory Development Plan in Adur comprises the Adur Local Plan (1996). All housing allocations in the Adur Local Plan 1996 have been delivered and the policies relating to housing targets are considered out of date.

3.4 The Emerging Adur Local Plan (Submission Adur Local Plan 2016 (CD07/01)) proposes a 'capacity based' target of 3609 dwellings over the plan period (2011 – 2032) equating to 172 dwellings per year. The SHLAA Update 2016 (CD07/20) proposes to review this target to 3718 dwellings between 2011 and 2032, 177 per year.

3.5 Comparatively the Objectively Assessed Housing Need UpdateSeptember 2016 (CD08/1) is the most up to date assessment of housing need in the absence of an adopted, up to date, Local Plan.

3.6 The OAN for housing in Adur has been identified as 6,825 homes over the plan period equating to 325 homes per year.

3.7 The Strategic Housing Land Availability Assessment (SHLAA) 2014 and Update (2016) considers a number of sites within Adur District, the aim to identify a future supply of land which is suitable, available, and achievable for housing uses over the plan period.

3.8 Adurs Housing Implementation Strategy (CD07/21) suggests that including two greenfield site allocations the plan can deliver 3718 new homes over the plan period.

3.9 This leaves a shortfall of 3107 dwellings for the plan period when measured against the 2016 OAN.

3.10 Given the significant shortfall in housing delivery against the OAN, we consider that more sites should be introduced as Strategic Allocations based on the SHLAA as there will continue to be a need for new housing to be delivered within the Local Plan area.

Phasing of Development

3.11 The proposed development at New Salts Farm would be broken down into Phases and delivered over approximately 7 years. The Phasing would take into account the completion of the Adur Tidal Walls Scheme. A suggested phasing plan is set out below in Table 2 with the later phases of development being on land currently within Flood Zone 3b which will be redesignated as 3a following completion of the Adur Tidal Walls.. This demonstrates that the site is available and development is capable of being delivered in a sequential approach in the short and medium term within the plan period providing much needed new homes to contribute towards housing need.

Table 2 – New Salts Farm Potential Phasing Plan

	2018/2019	2019/2020	2020/2021	2021/2022	2022/2023	2023/2024	2024/2025
No. of Homes	25	75	55	75	75	75	75

Strategic Flood Risk Assessment and Sequential and Exceptions Test

3.12 The Adur and Worthing Councils Strategic Flood Risk Assessment (SFRA) (2012) (CD15/1) identifies that 8 of the 10 allocated sites in Adur are at risk of flooding from the River Adur and wave overtopping and are in Flood zone 3a with 6 partially in Flood Zone 3b.

3.13 Recommendations in the SFRA in respect of New Salts Farm identify that the site would need to demonstrate it passes the Exception Test and provide a site specific Flood Risk Assessment, it also suggests a sequential approach to development to minimise flood risk.

3.14 The Sequential and Exception Test for the Amendments to the Proposed Submission Adur Local Plan (2016) (CD06/10) dismisses the site as not sequentially preferable as it is located in Flood Zone 3a and 3b and that no evidence has yet been submitted to overcome concerns regarding surface water and groundwater flooding. Although the report does accept that the flood risk from tidal sources at the site would improve with the Adur Shoreham Tidal Walls.

3.15 This site specific Sequential and Exceptions Test and the accompanying Flood Risk Assessment have been prepared with reference to the Strategic Flood Risk Assessment in order to provide the further evidence to demonstrate that there are no flooding constraints to development of the site and therefore enable the Local Planning Authority to allocate the site for housing.

Defining the Search Area

3.16 It was considered appropriate that the search area in this case should comprise the same boundary as the area defined in the Emerging Adur Local Plan. The reason for this is that the development is proposed to provide residential use towards meeting housing need in the district and therefore this would be an appropriate catchment area.

Applying the Sequential Test - Identifying Potential Alternatives

3.17 To identify potential sites that are available for development, a review of the Council's SHLAA 2014 and SHLAA Update 2016 has been undertaken.

3.18 The SHLAA considered a number of sites within the district and giving consideration to known constraints, neighbouring uses and planning history, determined the availability, suitability and achievability of individual sites. The sites assessed were put into one of four categories: Potential Site; Rejected Site – Monitor; Rejected Site; Committed Site.

3.19 New Salts Farm has been considered in the SHLAA and in past assessments has been identified as 'Rejected Site – Monitor' as a site being *'broadly suitable for housing development but not currently available for development and considered that they may offer development potential in the longer term and as such will be monitored on an annual basis.'*

3.20 However in the recent SHLAA update 2016 it has been assessed as 'Rejected' although there appears to be no clear justification as to why it has been moved to this category.

3.21 The alternative sites we have considered in addition to New Salts Farm for the purposes of this Sequential Test are those within the defined search area which have a similar capacity and therefore could deliver the proposed development of 455 homes and have been identified as 'Potential Sites' or 'Rejected Sites – Monitor' in the SHLAA. This is considered an appropriate approach as the Council has either accepted the site or accepts that there is potential for the site to be allocated for housing.

3.22 The alternative sites selected are noted in Table 3.

Table 3 – Alternative Sites

SHLAA ID	Site Address	Flood Zone	Estimated Approx. Capacity	Allocated in Emerging Local Plan	Potential Constraints
ADC/106/13	New Salts Farm	3	455***	No	Flooding Landscape
ADC/122/13	New Monks Farm	3	600**	Yes	Flooding Landscape

	Lancing				Transport
ADC/125/13	Land at West Sompting	1	480**	Yes	High visibility – design needs to be sensitive to this Transport - Transport Assessment required Ground Water Flooding – mitigation required
**estimated capacity taken from SHLAA 2016 ***estimated capacity based on The Hyde Group masterplan					

3.23 It is evident from viewing the above table that of the comparable sites both have been allocated and that similar to New Salts Farm, New Monks Farm also falls within Flood Zone 3. Only Land at West Sompting has a lower probability of flooding and this site which has already been proposed to be allocated for residential development with an estimated capacity of 480 homes.

3.24 It is noted that New Monks Farm which is one of the two greenfield sites proposed to be allocated for housing in the Emerging Local Plan also falls within Flood Zone 3 and has issues with groundwater flooding. Both sites are outside of the built up area boundary on the Proposals Map 1996 and are designated as Countryside and are within the Lancing / Sompting Strategic Gap.

3.25 In respect of New Monks Farm the SHLAA 2016 states that '*various constraints, including flood risk, transport and landscape impact are currently being addressed*'.

3.26 The same report identified that in respect of New Salts Farm '*constraints, including flood risk and landscape impact have not been addressed to the satisfaction of the local planning authority . . Until it has been demonstrated that these issues can be overcome to the satisfaction of Adur District council, West Sussex County Council (as Lead Local Flood Authority) and the Environment Agency, the site is not considered suitable for residential development'.*

3.27 The New Salts Farm site is being actively promoted by the landowner. Therefore in response to the concerns noted by the Council the landowner has actively engaged relevant technical consultants to prepare detailed reports in order to satisfy the local planning authority that the constraints identified relating to flood risk and landscape impact are capable of being addressed and mitigated and that the site is therefore available and residential development is achievable. This information was submitted to Adur in response to the Call for Site and Regulation 19 Consultation and meetings have been held with the Council, WSCC and EA to discuss the details submitted. We are therefore engaged in ongoing discussion seeking to address the constraints which the Council have identified and to demonstrate that these are not constraints to development of the site.

3.28 As was noted earlier all the sites identified in the SHLAA and proposed to be allocated in the Emerging Local Plan cannot deliver enough housing to meet the OAN in the area. There is therefore a need to look for further suitable sites.

3.29 There are no other sites identified in the SHLAA 2016 as Potential or Rejected Monitor which could deliver the level of proposed development at New Salts Farm that have not yet been proposed to be allocated in the Emerging Local Plan

3.30 It is therefore clearly demonstrated that there are no other suitable, available sites within the defined search area that could deliver the level of proposed development at New Salts Farm and fall into an area at a lower risk of flooding.

3.31 Development at New Salts Farm would make a significant contribution towards meeting Adur's housing need.

Conclusion

3.32 Paragraph 101 of the NPPF seeks that development should not be allocated or permitted if there are reasonable available sites appropriate for the proposed development in areas with a lower probability of flooding.

3.33 The above has demonstrated that following a review of potential alternative sites within Adur district, taking account of the SHLAA 2014 and SHLAA Update 2016, there are no other suitable, available sites within Adur of a similar capacity which could provide the level o development proposed at New Salts Farm and which falls into an area with a lower probability of flooding.

3.34 Further, as noted, additional sites are required to come forward in order to meet Adur’s full objectively assessed housing need and this site is available and deliverable within the Local Plan period.

3.35 On this basis it is considered that the Sequential Test has been passed and the site could be considered suitable for residential development where the Exception Test is also passed.

4. ADUR TIDAL WALLS

4.1 The Environment Agency submitted a planning application to Adur District Council in November 2015 for works known as the ‘Shoreham Adur Tidal Walls’ scheme (ref: AWDM/1614/15). The application was heard at the Adur Planning Committee on the 15th March 2016 and the Planning Committee resolved to grant planning permission subject to conditions. The application was approved on 1st June 2016.

4.2 The scheme involves a range of improvement works on the flood defences along the west and east banks of the River Adur to manage the risk of tidal flooding to the town of Shoreham-by-Sea.

4.3 The proposed works include:

- improvements to 1.8km of defences on the east bank between Coronation Green and the A27 road bridge and 5.4km of defences on the west bank between Shoreham Old Fort and Shoreham Toll Bridge;
- steel sheet piling, concrete walls, flood glass and earth embankments;
- a section of road raising, scour protection in the form of rock revetment, matting, planted terraces and gabions;
- Creation of a 1.3ha intertidal salt marsh; and
- Landscape improvements to Town Quay and Shoreham Old Fort car park.

4.4 If no works are undertaken to the defences then rising sea levels and the continued deterioration of the defences could lead to a catastrophic failure in just 1-in-20 year event. With the proposed new defences in place the residential and commercial properties in Shoreham currently at risk from flooding would be protected into the future from a 1-in-300 year (0.33% AEP) tidal flood event. As sea levels rise the number of properties that the improved defence will protect will increase, up to the 50-year design life of the Scheme.

4.5 In addition once the defences have been constructed, areas designated Flood Zone 3b will be redesignated 3a. This includes areas within the New Salts Farm development site. This would alter the flood risk vulnerability classification of development permissible in the area.

4.6 The Core Strategy site Flood Risk Assessment (2012) states *that ‘these new defences are likely to have a positive affect on the present day and future ‘defended’ flood extents, and future development proposals should give regard to the detailed outputs from the Adur Tidal Walls study’.*

4.7 The works are anticipated to be completed in 2018.

4.8 This is an important consideration for New Salts Farm. The completion of these works would partly address tidal and fluvial flooding at the site. It would also redesignate parts of the site currently in Flood Zone3b to Flood Zone 3a, meaning 'more vulnerable' development (in the flood zone vulnerability classification) in these areas would become appropriate, subject to an Exception Test. Given it is anticipated that the works would be completed in 2018 this would enable the site to be phased appropriately to deliver new housing across the whole site within the plan period, contributing towards housing need in the District.

5. THE EXCEPTION TEST

Background

5.1 In line with the approach set out in the NPPF and PPG and the Flood Zone Classification table, having demonstrated that it is not possible for the development to be located in zones with a lower probability of flooding, the Exception Text has been applied to the site. In doing so we have had regard to the Shoreham Adur Tidal Walls scheme as advised in the Core Strategy Flood Risk Assessment.

5.2 Approximately 60% of the site falls within Flood Zone 3a whereby an Exception Test is required for residential development.

5.3 The remainder of the site currently falls within Flood Zone 3b, which is not considered suitable for residential development. However on completion of the Shoreham Adur Tidal Walls scheme these areas will be redesignated to Flood Zone 3a, and would then be in the same flood risk vulnerability classification as the rest of the site and subject to an Exception Test for residential development.

5.4 In approaching this Exception Test we have had regard to the Shoreham Adur Tidal Walls Scheme and the anticipated timing of completion of these works in 2018. We consider that the proposed redevelopment of New Salts Farm could be sequentially designed and phased so as to deliver residential development taking account of the completion of the Shoreham Adur Tidal Walls works ensuring that no development would be completed within areas currently designated as Flood Zone 3b prior to completion of those works.

5.5 For the Exception Test to be passed it must be demonstrated that the development provides wider sustainability benefits to the community that outweigh flood risk, and a site specific flood risk assessment must demonstrate that the development will be safe for its lifetime without increasing flood risk elsewhere and where possible decrease flood risk overall.

Sustainability Benefits to the community

5.6 The PPG states at paragraph 024 that *'evidence of wider sustainability benefits to the community should be provided, for instance, through the sustainability appraisal'.*

5.7 We have reviewed the potential of the proposed development to provide wider sustainability benefits by considering the scheme alongside the sustainability objectives set out in the Sustainability Appraisal of the Adur Local Plan 2016 (CD07/02), having regard to that document and the Adur and Worthing Strategic Flood Risk Assessment (2012).

5.8 Table 4 below sets out the sustainability objectives identified in the Sustainability Appraisal and how the development could be designed to meet those objectives.

Table 4 – Compliance with Adur Sustainability Objectives

Sustainability Objectives	Compliance
Increase energy efficiency and encourage the use of renewable energy sources	The proposed development will incorporate renewable/low carbon energy sources where demonstrated appropriate and feasible. The proposed development has been designed using the 'fabric first' principle. The dwellings are designed to be highly insulated, reduce heat loss and air leakage, which in turn reduces the heating requirements for the dwellings. The heating that is required will be delivered using energy efficient technologies accompanied with low or zero carbon technologies. The design target for the dwellings is to achieve 19% CO2 reduction beyond Part L 2013.
Protect and enhance water quality and encourage the sustainable use of water Reduce pollution and the risk of pollution to air, land and water	Sustainable Drainage Systems (SuDS) will be incorporated in the development (as detailed later in this report) which shall manage the surface water run-off from the development. To reduce water consumption within the dwellings each dwelling shall be fitted with water efficient sanitaryware to enable the predicted consumption to be no greater than 105l per person per day. To protect the quality of groundwater all surface water run-off from the roof and hard paved areas will receive surface water treatment to satisfy the level of treatment recommended within the SuDS Manual, before discharging into the ground. There is potential for noise impact on the new development from the airport and railway, however this would be capable of being mitigated in any new development through detailed design.
Improve land use efficiency by encouraging the re-use of previously developed land, buildings and materials	The proposed development is on a greenfield site. However Adur have already accepted that some greenfield land would need to be allocated to contribute towards meeting housing need, although it has not allocated enough sites to meet that need. For reasons noted earlier in this report New Salts Farm is considered to be suitable, available and achievable and should be allocated for housing to contribute towards meeting housing need.

Conserve, protect and enhance biodiversity and habitats	The site contains BAP habitats and NERC habitats and supports a number of protected species. However a site specific preliminary ecological appraisal has identified that whilst there are areas of higher ecological interest these can be accommodated within the scheme and maintained and enhanced with potential to also provide ecological benefits on the site. For example the existing ditch network would be maintained in the scheme including a buffer zone in order to maintain the existing water features and supporting habitats. There is also an opportunity to enhance the ditch network by removing invasive species. The wider site illustrative masterplan includes areas of open space and there is potential for these to be left as unmanaged space to maintain some of the grazing floodplain habitat and ensure reptiles can be retained on the site.
Protect and enhance the historic environment including townscapes, buildings, archaeological heritage, parks and landscapes Protect and enhance the countryside Protect and enhance public open space / green infrastructure and accessibility to it	The most recent landscape sensitivity assessment 2016 (CD14/10) carried out on behalf of Adur identifies the site as having a medium-high visual sensitivity and provides a valuable ‘slice of green’ separating urban areas. A landscape and visual appraisal of development proposals at new salts farm (2016) (CD14/22) suggests that development at New Salts Farm would have significant and irreversible landscape and visual effects. Although goes on to suggest options to mitigate these negative impacts. We have substantial concerns regarding the reliability of Adur’s evidence base in this regards which is set out in detail in our Regulation 19 Reps. Our client commissioned a separate study reviewing the work done on behalf of Adur which demonstrates that New Salts Farm has a moderate-high capacity to accommodate housing and would not result in coalescence as a significant areas of space would be retained as a Local Green Gap. A landscape strategy has been development for the proposed site which seeks to integrate the development into the existing landscape by inclusion of soft boundary treatments and tree planting; retention of open boundary treatment to the east and north to retain a sense of openness; incorporation of ditches in the development and informal amenity and recreation areas.

	Whilst the development would result in the loss of a greenfield site, the proposed illustrative layout, along with landscape measures proposed, has the potential to respond to the sensitivities of the local landscape character and safeguard the qualities of the local green gap and provide a number of positive landscape enhancements.
Ensure that all developments have taken into account the changing climate and are adaptable and robust to extreme weather events Avoid, reduce and manage the risk from all sources of flooding to and from the development	A Flood Risk Assessment for the site (discussed further below) has identified how flood risk in the present and future would be managed and mitigated to ensure the development would remain safe for its lifetime and not result in an increase in flood risk elsewhere. The Environment Agency are progressing the Shoreham Adur Tidal Walls Scheme which will improve flood defences and would partly address tidal and fluvial flooding at New Salts Farm. This scheme will also redesignate those parts of the site within Flood Zone 3b as 3a.
Improve health and wellbeing and reduce inequalities in health	The development would provide new areas of accessible open space which would have health benefits for new and existing residents.
Reduce crime, the fear of crime and antisocial behaviour	The layout of the proposed development has sought to design out elements that can contribute towards crime and antisocial behaviour, thereby contributing towards reducing crime, the fear of crime and anti-social behaviour.
Promote sustainable transport and reduce the use of the private car	The site is close to Shoreham Town Centre with a number of local services including supermarkets, doctor and dentist surgeries. There are good pedestrian footways and cycle routes in the vicinity which the proposed development could connect to. Bus routes are located along Brighton Road close to the site. It is therefore considered that the site is well located close to sustainable modes of transport and hence will reduce the need for future residents to travel by car.
Reduce poverty, social exclusion and social inequalities Meet the need for housing and ensure all groups have access to decent / appropriate housing Create and sustain vibrant	The proposed development would deliver new housing of a mix of tenures, including affordable housing, and sizes and hence would go towards meeting the objectively assessed housing needs in Adur District. A mix of homes, including affordable homes, would create a vibrant community in a sustainable location and would contribute towards creating mixed and balanced communities which would help to reduce social exclusion and inequalities.

communities which recognise the needs and contributions of all individuals	
Promote sustainable economic development with supporting infrastructure, and ensure high and stable levels of employment and a diverse economy	The proposed development by providing new homes, including affordable homes, in a sustainable location close to local facilities would help to attract people to live and work in the district thereby supporting this objective. Additionally in the short term construction jobs would be created which would help the economy.
Improve the range, quality and accessibility of key services and facilities and ensure the vitality and viability of existing centres	The site is in a sustainable location with good access to existing local facilities in Shoreham Town Centre by bus, foot and cycle. It would introduce new housing which would utilise local services thereby contributing towards the viability and vitality of existing centres.
Create places, spaces and building that work well, wear well and look good	HGP are high quality architects who have been engaged to develop a scheme on the site. They have prepared an initial illustrative masterplan to demonstrate how the site could be developed to deliver a high quality residential scheme. Further design development will seek to achieve high standards and create places, spaces and buildings that work well, wear well and look good.
Raise educational achievement and skills levels to enable people to remain in work and to access good quality jobs	No education/training facilities are proposed on the site, although financial contributions towards education are likely to be required as part of a planning application.
Reduce the amount of domestic and commercial waste going to landfill as per the waste hierarchy	The waste arrangements for the development will be designed so as to minimise waste and encourage recycling and other waste management prior to sending waste to landfill

5.9 It can be seen from the table above and assessment of the proposed development against the sustainability objectives of the Emerging Adur Local Plan that the development of the site has the potential to give rise to significant sustainability benefits and generally scores positively. In particular it will provide additional homes, including affordable housing, within a sustainable location and set within high quality landscaping and open space, close to local facilities and with the ability to connect to existing sustainable transport modes.

5.10 The proposal scores positively against the aims and objectives of the sustainability appraisal and demonstrates that the sustainability benefits of the development to the community outweigh the flood risk, therefore passes this first part of the Exception Test.

Safe for Its Lifetime

Introduction

5.11 The use proposed at the site is residential which falls within the more vulnerable category. For those areas within Flood Zone 3a the site is suitable for residential development where the Exception Test is passed.

5.12 It is noted that part of the site is currently within Flood Zone 3b and considered not suitable for more vulnerable development. However, the planned improvements to flood defences in the River Adur would remove this area from Flood Zone 3b and redesignate it as Flood Zone 3a. This would make those parts of the site to be redesignated suitable for residential development after that time, where the Exception Test is passed. The proposed new defences are anticipated to be completed in 2018 therefore enabling delivery of housing on this site within the Local Plan period.

5.13 Paragraph 038 of the PPG states that '*the developer must provide evidence to show that the proposed development would be safe and that any residual flood risk . . . can be overcome to the satisfaction of the local planning authority*.' It goes on to say that '*the developer's site-specific flood risk assessment should demonstrate that the site will be safe and that people will not be exposed to hazardous flooding from any source*'.

5.14 A site specific Flood Risk Assessment (FRA) has been prepared for the development site by Tully De'Ath. This report should be read in conjunction with that document, although it has been summarised below.

Flood Risk

5.15 In respect of fluvial flood risk the site is in Flood Zone 3a with a residual risk associated with a breach of the River Adur flood defences. The site is within Flood Zone 3a and 3b for Tidal Flood Risk associated with a residual risk of breaching of defences along the River Adur and wave overtopping along the coastal frontage.

5.16 The site is also susceptible to ground water flooding (ground water emergence is more than 75%). It has a low surface water flood risk with those areas highlighted as susceptible to flooding being aligned to existing drains and ditches on the site. There is no reported incident of sewer flooding within the site.

5.17 Flood model data from the Environment Agency suggests that the 1:200 event with an allowance for climate change was the most onerous with a maximum flood height of 5.391m AOD for the undefended flood event and 5.05m AOD for a defended scenario.

5.18 The existing defences have been assessed as being in relatively good condition. Further, as noted earlier, proposed improvements to the flood defences as part of the Shoreham Adur Tidal Walls scheme will partially address tidal and fluvial flooding at the site and redesignate those parts of the site within Flood Zone 3b as Flood Zone 3a.

Flood Management and Mitigation

5.19 The FRA sets out a number of flood management and mitigation methods which could be incorporated in the final design to address flooding at the site which are summarised below.

5.20 All units would provide accommodation at first floor level only with this floor set above the 1 in 200+CC tidal event. This would equate to a minimum floor level of 5.35m AOD which is 3m above existing ground level. Ground floor levels would also be locally raised by 300mm to mitigate against the risk of ground and surface water flooding. All units would be designed using flood resilient materials and structurally designed to withstand potential flood depths.

5.21 All units would have direct access to first floor which would be the primary area for refuge in the event of a major flood event. All units would be linked to the EA's flood warning system and a site specific Flood Evacuation Plan will be provided and agreed with the Emergency Planning Team which gives guidance and advice to residents with regards to flood risks.

5.22 Surface Water run-off will be restricted to match greenfield run off rates via use of flow control devices.

5.23 Surface water attenuation will be provided in a variety of devices including roof top attenuation, permeable paving and swales.

5.24 Attenuation will be designed to hold a 6hr 100+CC event within the sub-base material with overflow directed to the adjacent swales and ditches should this be exceeded.

Surface Water Drainage Proposals and Sustainable Drainage Systems (SuDS)

5.25 The proposed method of surface water disposal will be via shallow infiltration and sustainable drainage systems (SuDS) will also be incorporated including:

- Water butts which can reduce the total volume of storm water run-off and provide additional attenuation;
- Green roofs on all roofs will provide storm water attenuation and reduce run off;
- Geocelular roof attenuation systems beneath the green roofs will create an additional storage structure which will discharge into the sub-base of permeable roads;
- Permeable paving with base infiltration will be included to all hard paved areas and will allow water to infiltrate and be temporarily stored before infiltrating into the sub-soils;
- New swales / infiltration trenches will be introduced either side of the new access road and linked to the existing ditch system on site;
- Bio retention areas will be introduced in the landscaping to provide additional exceedence event storage.

5.26 All surface water run-off from the roof and hard paved areas will receive an element of surface water treatment before discharging into the ground to satisfy the level of treatment recommended in the SuDS Manual.

Flood Risk Assessment Conclusion

5.27 The FRA has identified the current and future flood risk to the site and demonstrated how this would be managed and mitigated over the developments lifetime to demonstrate that the development can be designed so as to remain safe for its lifetime and would not increase flood risk elsewhere.

5.28 The principles for management and mitigation of flood risk will be incorporated across the site, although those areas which are currently within Flood Zone 3b will not be developed until the Shoreham Adur Tidal Walls scheme is completed and they have been redesignated.

5.29 The proposals have therefore demonstrated that the second part of the Exception Test has also been passed.

Conclusion

5.30 This chapter has demonstrated that the development site offers wider sustainability benefits to the community that outweigh flood risk, and that the development can be designed to incorporate measures to mitigate and manage flood risk now and for the lifetime of the development and not increase flood risk elsewhere.

5.31 While some parts of the site are currently within Flood Zone 3b and would not be considered appropriate for residential development at this time these would be redesignated once the Shoreham Adur Tidal Walls scheme is complete to Flood Zone 3a and would not be developed until after this time.

5.32 It is concluded that the Exception Test has been passed, and the site can be considered appropriate for residential redevelopment.

6. CONCLUSION

6.1 The NPPF sets out tests to protect people and property from flooding. It requires a sequential approach to site selection to ensure development is as far as possible directed to the areas at lowest risk of flooding. Where development needs to be in locations at risk from flooding it should demonstrate that it provides sustainability benefits to the wider community and would be safe for its lifetime.

6.2 It has been demonstrated that Adur District Emerging Local Plan does not currently allocate enough development sites to meet its objectively assessed housing need. It is therefore clear that further development sites should be brought forward to meet this need.

6.3 This report has demonstrated that the development site at New Salts Farm would pass the Sequential Test as there are no other available sites within a lower Flood Zone that could provide the development proposed.

6.4 It is relevant that the Shoreham Adur Tidal Walls proposals to improve flood defences in the River Adur would have a positive impact on the development site in terms of flooding and would open up areas of the site currently not considered suitable for residential development. These defences are anticipated to be completed in 2018 and would enable deliverability of new housing on the site within the plan period.

6.5 In respect of the Exception Test, this report has demonstrated that the proposed development would provide sustainability benefits to the wider community that outweigh flood risk, particularly through the provision of new housing, including affordable housing, to meet objectively assessed need. Further the Flood Risk Assessment has demonstrated that the proposed development would incorporate through its design, measures to manage and mitigate flood risk at the site to demonstrate that it would be safe for its lifetime without increasing flood risk elsewhere.

6.6 In accordance with the NPPF and PPG it has been demonstrated, informed by a site specific Flood Risk Assessment, taking account of the future Adur Tidal Walls Scheme and following the Sequential and Exception Tests that the development is appropriately flood resilient and resistant, any residual risk can be safely managed and sustainable drainage systems have been incorporated and there is no increase in flood risk elsewhere.

6.7 The Sequential and Exception Tests have been passed and the development can therefore be considered appropriate and be permitted in line with the NPPF.

Boyer

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Appendix G – SFRA Historical Flood Maps

Core Strategy Site Flood Risk Assessment: summary and recommendations

Site Details

Site Name	Land North East of the Hasler Estate
Site Location (OS NGR)	TQ200046
Site Area (ha)	30.4
Proposed use	Residential
Flood risk vulnerability classification (PPS25 Table D2):	More Vulnerable
Brown/Greenfield	Greenfield

Flood Risk

Flood Zones (Fluvial & Tidal)		Comments
Flood Type	Fluvial and Tidal	River Adur, tidal estuary and coastline
Percentage of site in Flood Zone 3b	39%	
Percentage of site in Flood Zone 3a	61%	
Percentage of site in Flood Zone 2	0%	This excludes any area contained within Flood Zone 3
Percentage of site in Flood Zone 1	0%	Flood Zone 1 indicates the area lying outside of Flood Zones 2 and 3
Defended?	Formal defences along the River Adur and the coastline.	Maintainer: Local Authority, private and EA
		Standard of Protection: Less than 1 in 20 year.

Surface water flooding

Susceptibility	The susceptibility to surface water flooding during a 1 in 200 year event for the majority of the site is shown to be less to intermediate .
Flood map for surface water	There are small pockets of flooding, some deep, associated with the 1 in 30 year and 1 in 200 year event across the site according to FMfSW.

Other sources of flood risk

Groundwater Flood Risk	The site is underlain by the Newhaven Chalk Formation, and is within the EA's major aquifer high vulnerability zone. Consequently the area may be susceptible to groundwater emergence. According to the EA groundwater susceptibility map, the site resides in a series of 1km squares where the proportion of each 1 km square that is susceptible to groundwater flood emergence is more than 75%.
Sewer Flood Risk	No reported incidents of sewer flooding within the site. Reported incident to south west of the site (West Way)

Residual risk

Fluvial Residual Risk	Yes - there would be a residual risk associated with breach of the River Adur west bank defences.
Tidal Residual Risk	The Flood Zones show the site would be inundated if undefended, therefore there is a residual risk associated with breach of the defences along the River Adur. Also, detailed modelling has been undertaken to assess the impact of wave overtopping along the coastal frontage. The results show that the site is at a high risk of inundation as a result of wave overtopping in both the 1 in 20 and 1 in 200 year events.

Effect of climate change

Detailed modelling undertaken to assess the impact of climate change of the tidal flood extent show that the entire site would suffer inundation in the future (2115) 1 in 200 year event. The impact of climate change on surface water or groundwater has not been assessed as part of this SFRA.

Is a site specific Flood Risk Assessment required?

FRA required?	Yes	Site is over 1ha and has significant areas within Flood Zone 3a. Small areas at residual risk from wave overtopping. Additional high risk of groundwater emergence and surface water flooding. The site is at significant risk from the affects of climate change.
Exception test required for proposed use?	Yes	The majority of the site is within Flood Zone 3a. The exception test would need to be met for more vulnerable development within the site. Notably, to meet the exception test the FRA would need to demonstrate that the development is 'safe'.

Recommendations for Development

The site is within Flood Zones 3a, and 3b and has a history of flooding. All development proposals should be accompanied by a FRA. Flood Zone 3b is not considered suitable for less, more, or highly vulnerable developments. Flood Zone 3a is not suitable for highly vulnerable developments. The Exception Test is required for essential infrastructure and more vulnerable proposals. Water compatible land uses are considered compatible. Future development should be mindful of the various sources of flood risk, and where possible implement sequential design throughout the site to try to reduce flood risk within the development.

The effect of climate change should be considered for all new development, at present it is shown that the risk of flooding will increase in the future if the current defences remain unchanged as a consequence of reduced SoP.

There is also a risk from wave-overtopping, an assessment should be carried out on the impact of wave overtopping so that any future development can be designed with this in mind. Future developments should be resilient to the effects of wave overtopping and the site should be sequentially designed ensuring the development remains safe in the event of wave overtopping i.e. situating resilient uses on the ground floor.

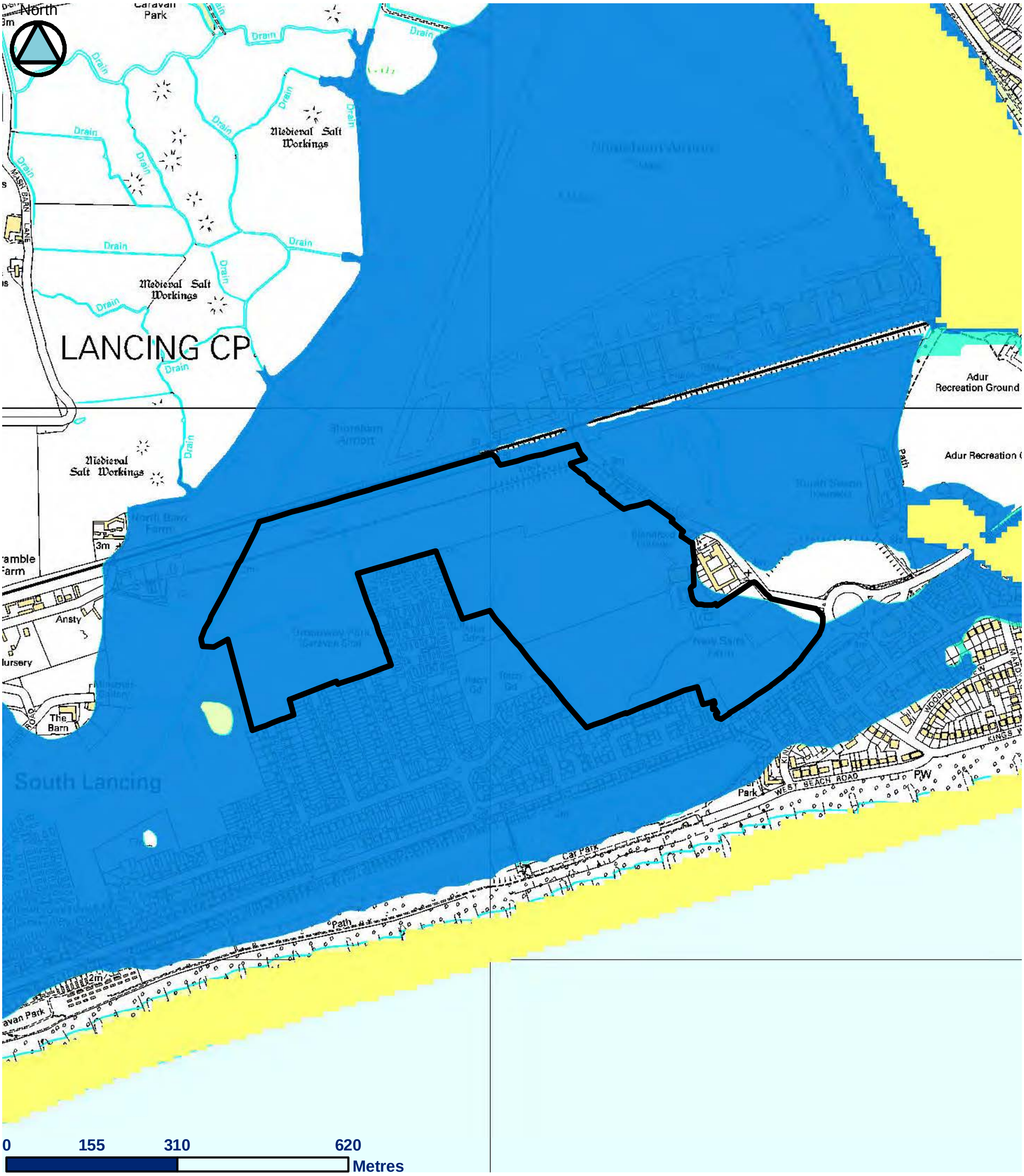
The site is also at risk of groundwater and surface water flooding, therefore steps should be taken to reduce the consequence of flooding. Any future development should ensure that it would not increase the surface water flood risk elsewhere, to achieve this any existing flow paths would need to maintained. The site is greenfield so surface water drainage techniques should be built into any new design to ensure the runoff rate does not increase.

Improvements to the tidal walls along the River Adur in the vicinity of the site have been proposed (see section 4.3.4 of the main report for more information). When these improvements occur the floodplain designation will change, and areas designated 3b will be redesignated 3a. This will alter the flood risk vulnerability classification of development permissible in the area. These new defences are likely to have a positive affect on the present day and future ‘defended’ flood extents, and future development proposals should give regard to the detailed outputs from the Adur Tidal Walls study. At present available information indicates that the planned improvements are to the present day 1 in 200 year standard. This standard of protection is expected to decrease in the future with climate change and this should be considered early in the design of the development, including directing the highest vulnerability land uses to areas of lowest risk. Any new development should be resilient to future climate change, as well as the effect of wave overtopping and a failure in the defence. A detailed FRA will be required to assess these aspects.

Worthing

Adur and Worthing SFRA update
Flood Risk Summary Map

ADUR



Legend

- Land to the North East of the Hasler Estate
- Flood Zone 3b
- Flood Zone 3a
- Flood Zone 2

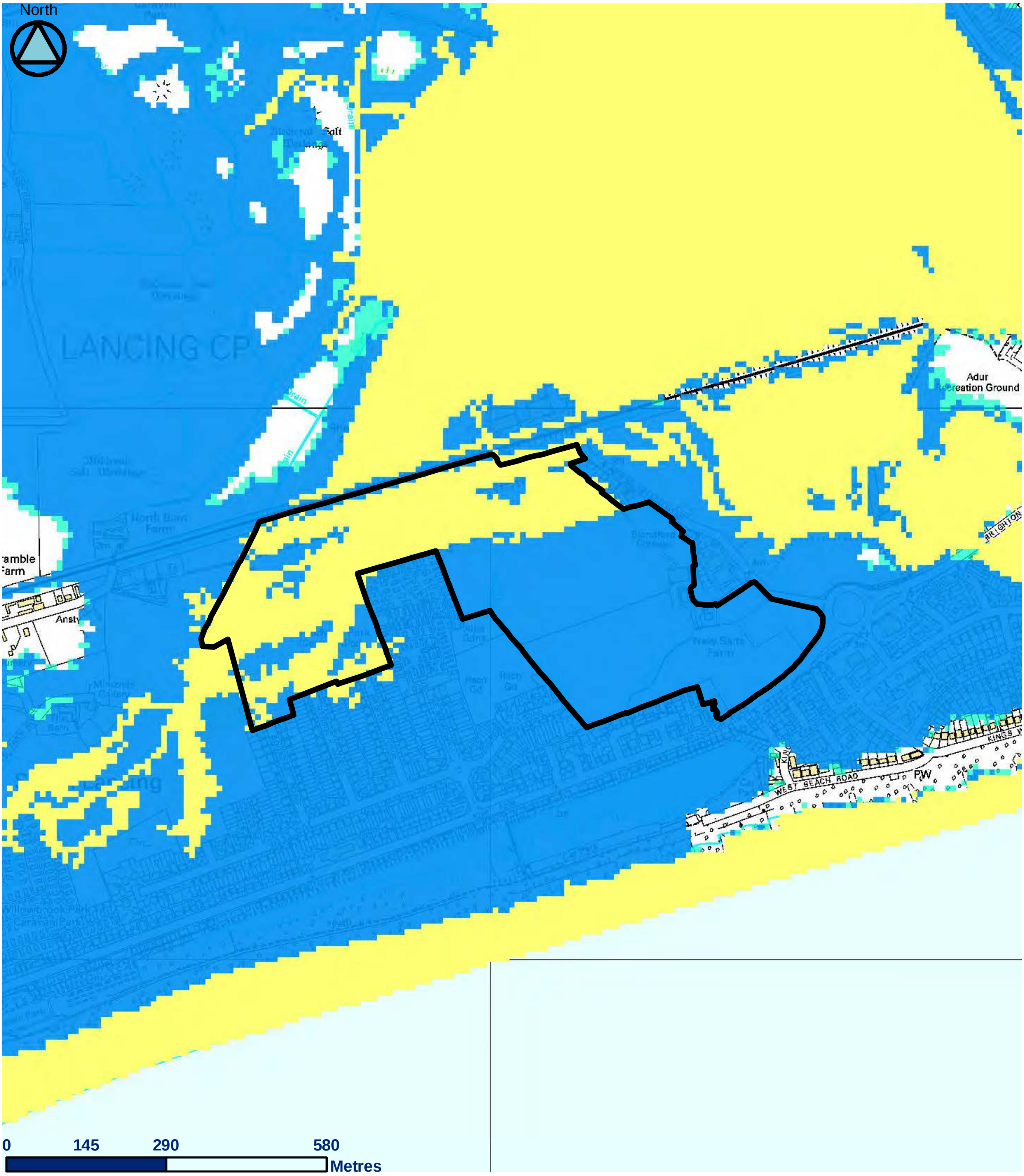
JBA consulting

Land to the North East of the Hasler Estate

Fluvial Flood Risk

(Floodplain Delineation - Undefended)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

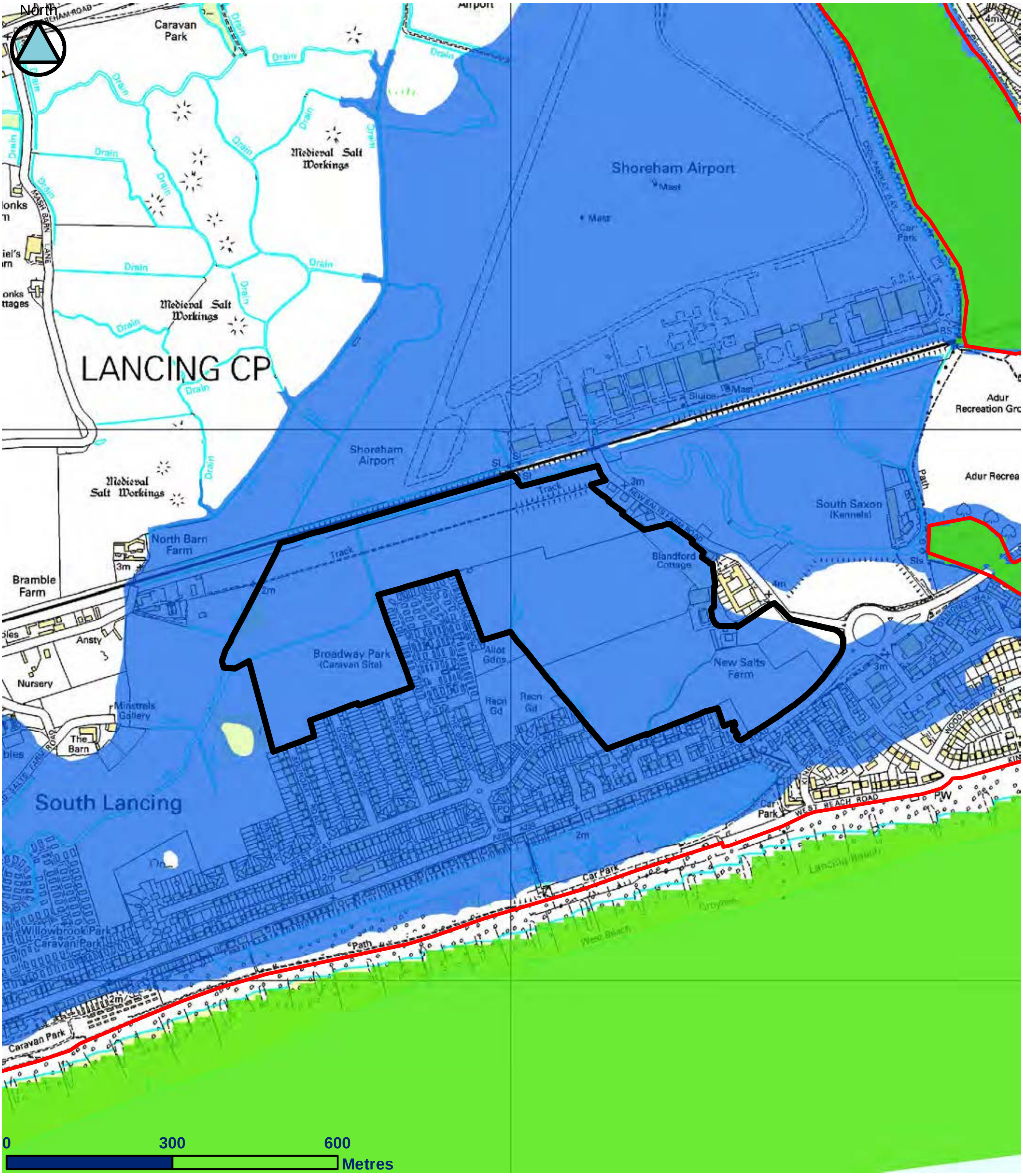
- Land to the North East of the Hasler Estate
- Flood Zone 3b
- Flood Zone 3a
- Flood Zone 2

Land to the North East of the Hasler Estate

Tidal Flood Risk

(Floodplain Delineation - Undefended)

Adur and Worthing SFRA update
Flood Risk Summary Map



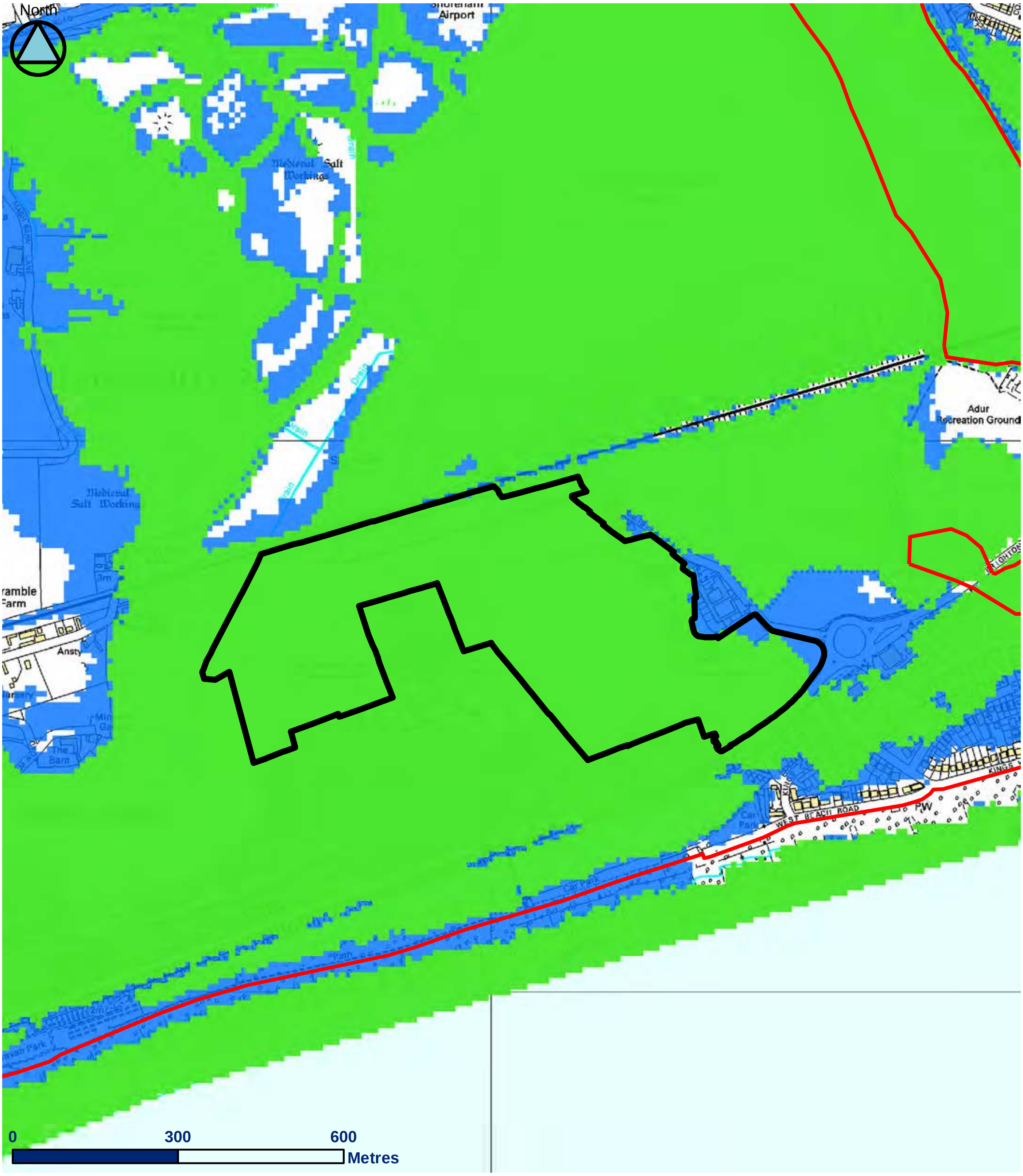
Legend

- Land to the North East of the Hasler Estate
- Adur and Worthing Defences
- 1% AEP Defended
- Flood Zone 3a

Land to the North East of the Hasler Estate

Actual Fluvial Flood Risk (with defences)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

Land to the North East of the Hasler Estate
Adur and Worthing Defences
0.5% AEP Defended
Flood Zone 3a

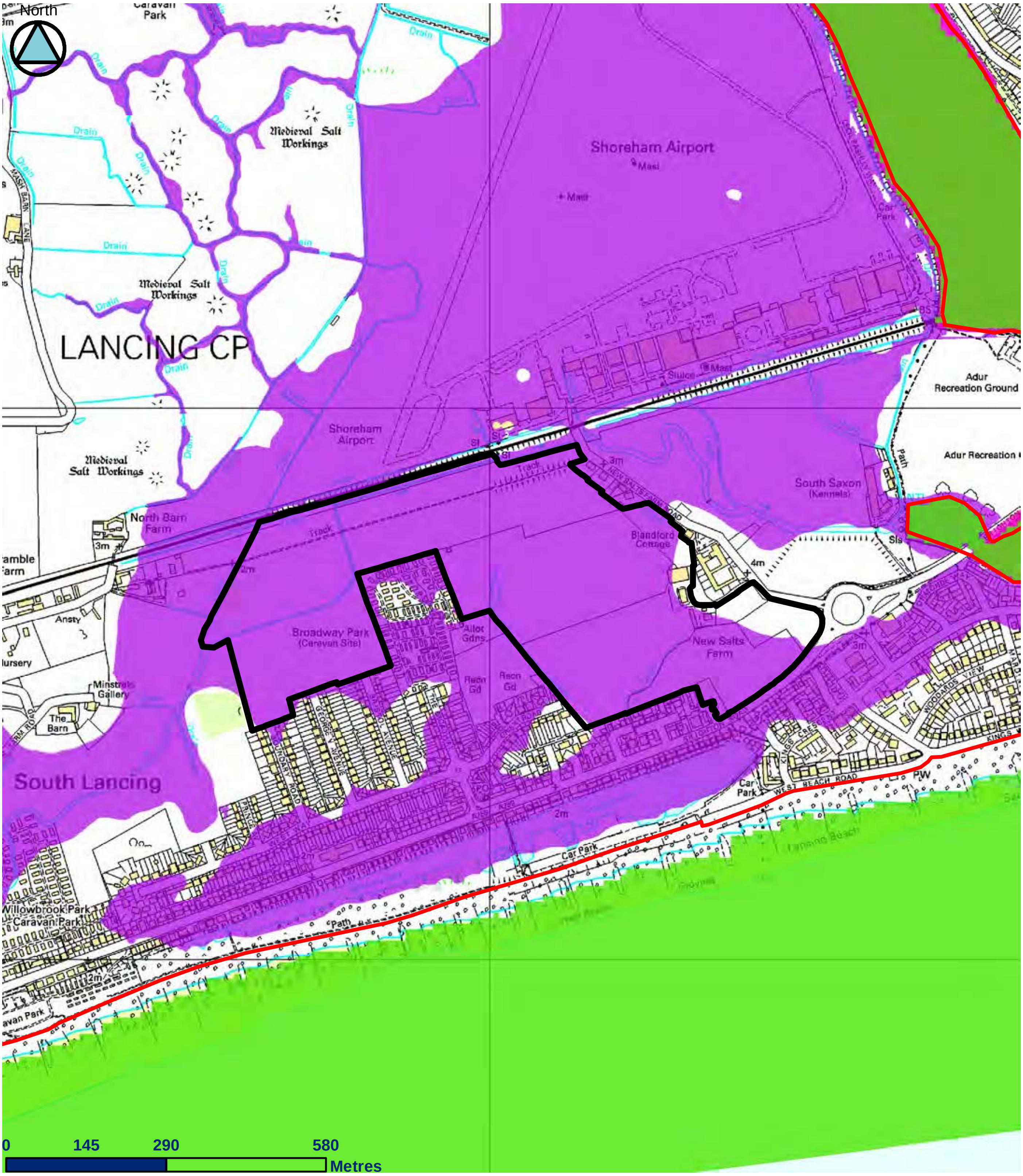
Land to the North East of the Hasler Estate

Actual Tidal Flood Risk (with defences)

Worthing

Adur and Worthing SFRA update
Flood Risk Summary Map

ADUR



Legend

- Land to the North East of the Hasler Estate
- Adur and Worthing Defences

1% AEP Defended

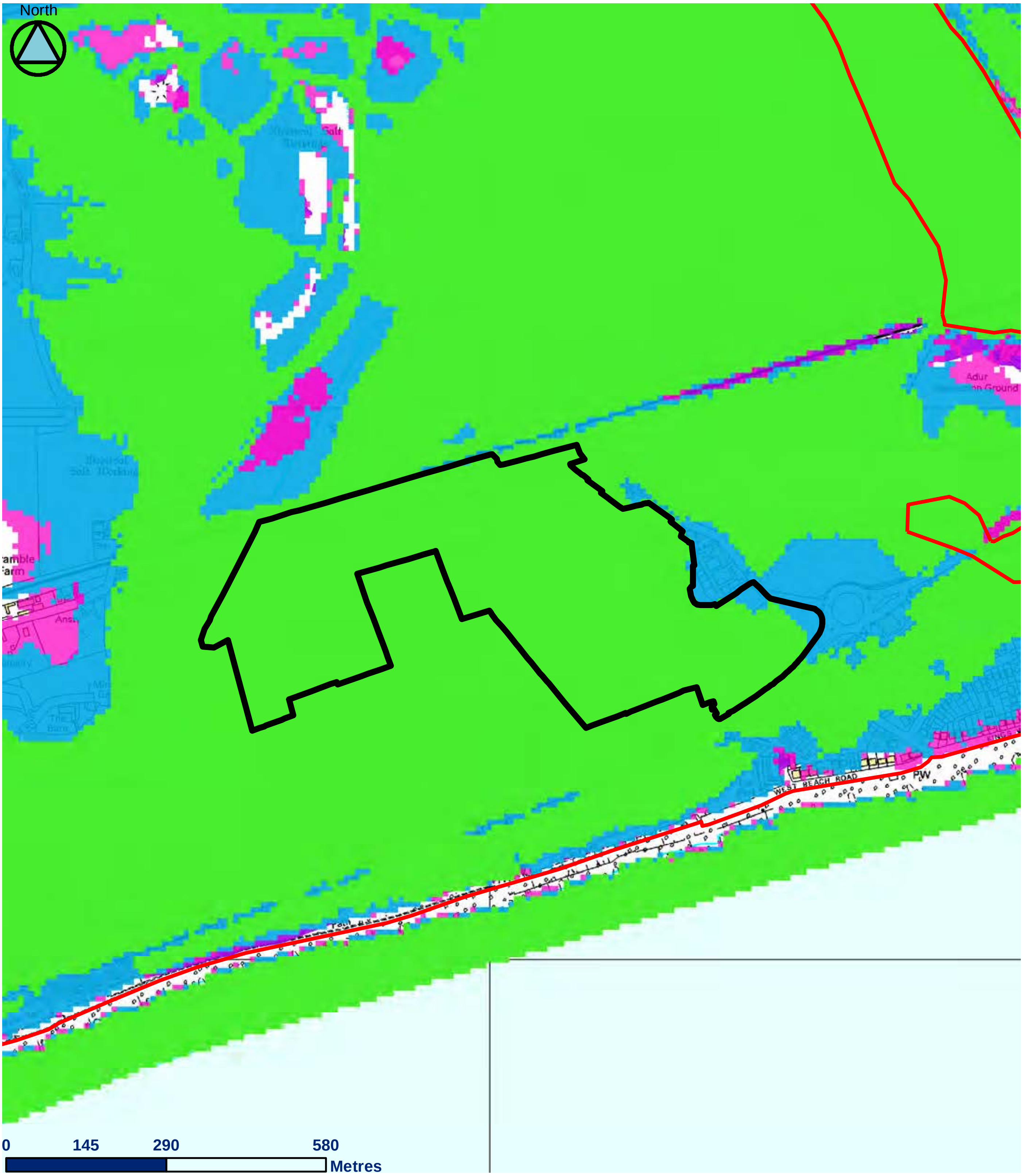
- Present Day
- Climate Change (2056)
- Climate Change (2106)

JBA consulting

Land to the North East of the Hasler Estate

Future Fluvial Flood Risk (with defences)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

- Land to the North East of the Hasler Estate
- Adur and Worthing Defences

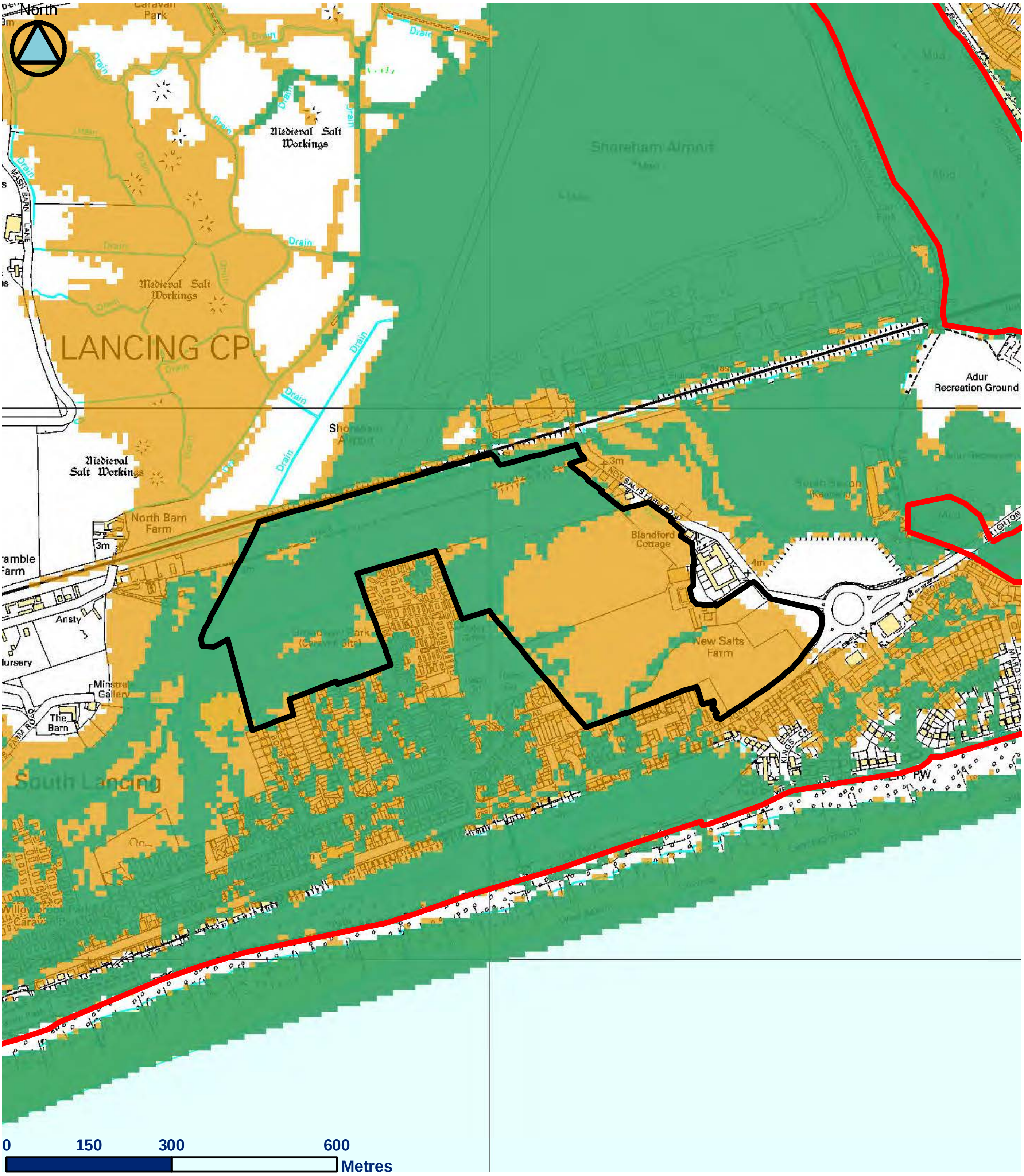
0.5% AEP Defended

- Present Day
- Future 2070
- Future 2115

Land to the North East of the Hasler Estate

Future Tidal Flood Risk (with defences)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

Land to the North East of the Hasler Estate

Defences

Defended present day with wave overtopping

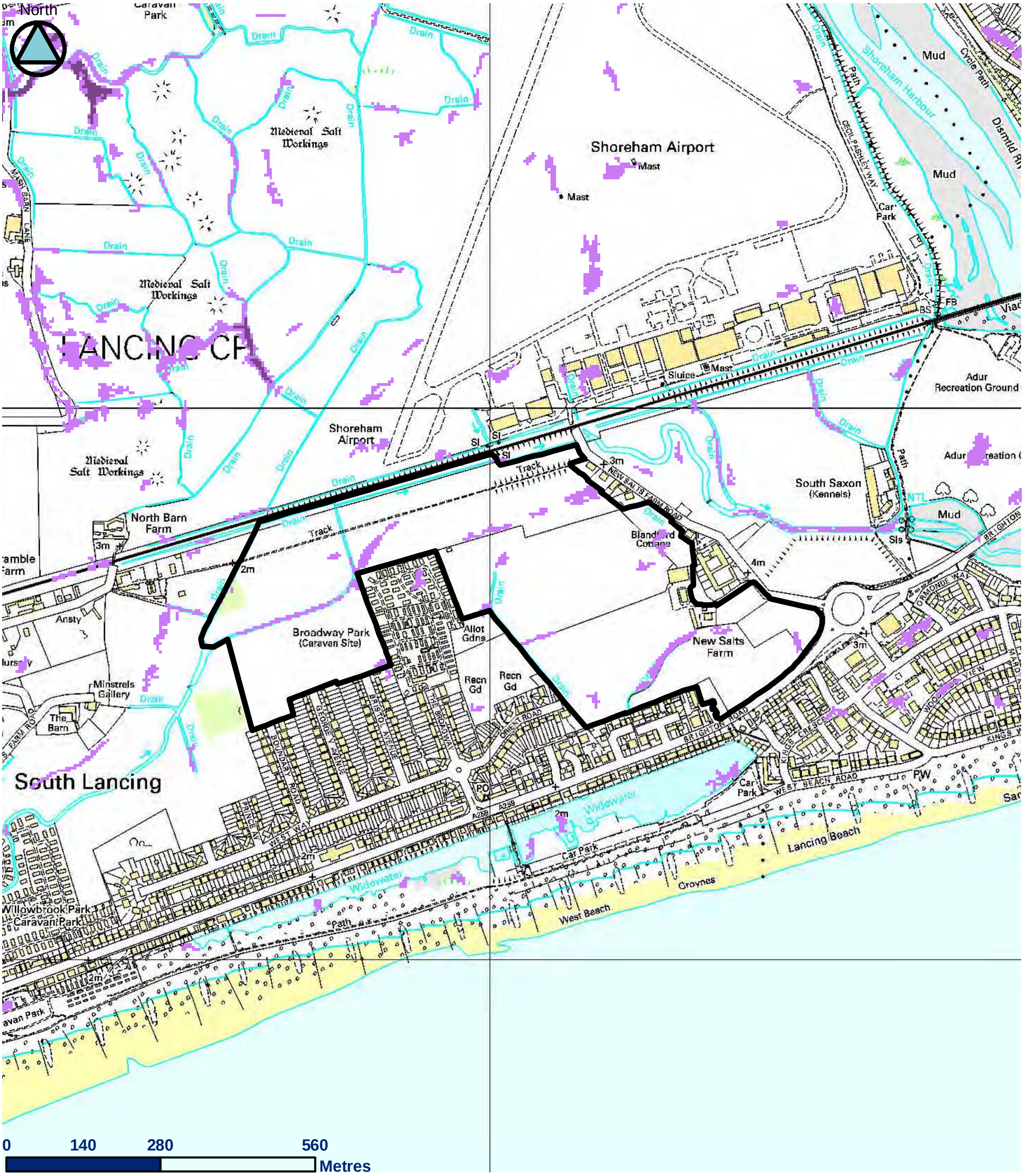
5% AEP

0.5% AEP

Land to the North East of the Hasler Estate

Residual Tidal Flood Risk
Wave Overtopping

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

Land to the North East of the Hasler Estate

Depth

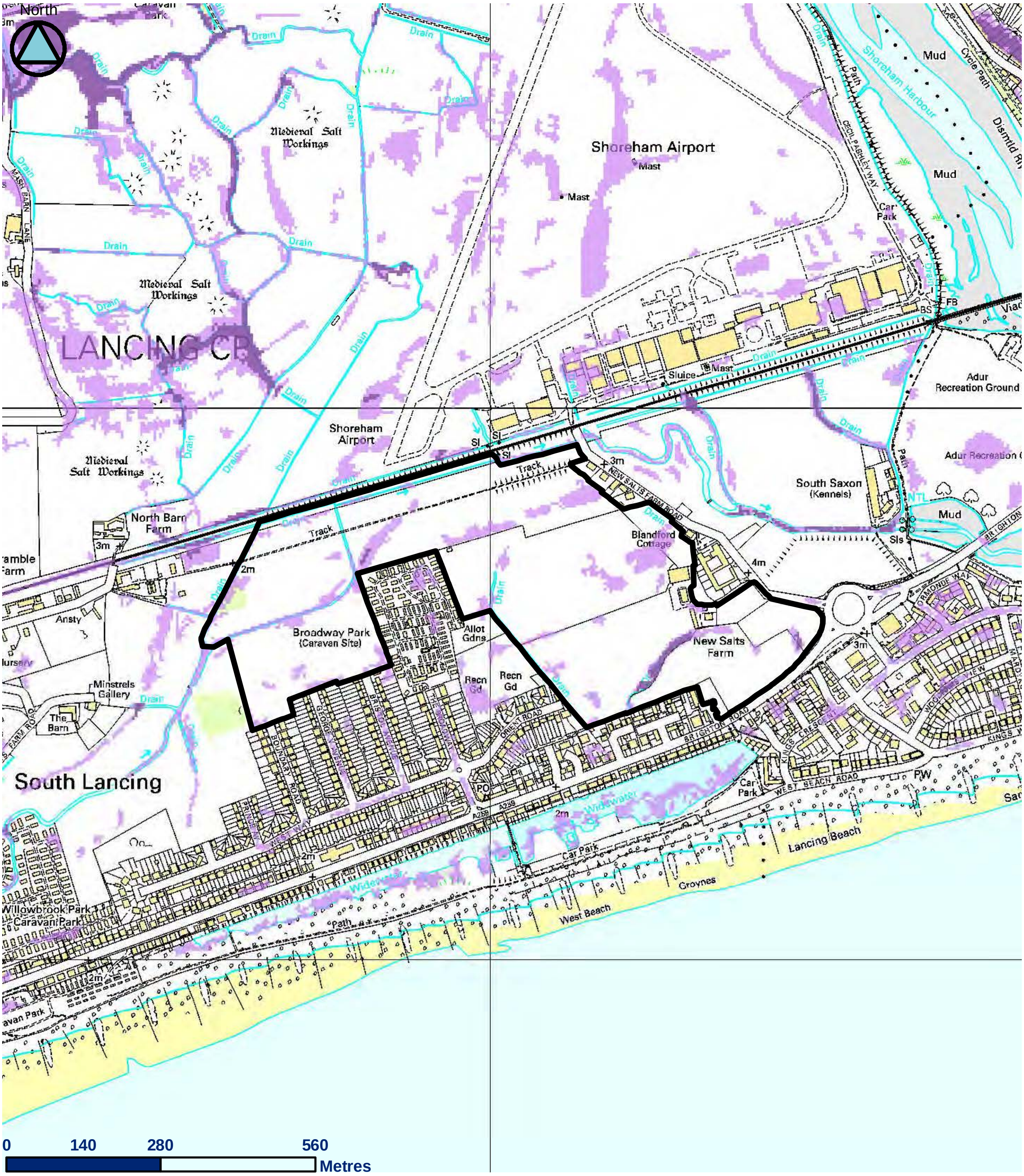
> 0.1m

> 0.3m

Land to the North East of the Hasler Estate

Surface Water Flood Risk
(Flood Map for Surface Water 1 in 30 year)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

Land to the North East of the Hasler Estate

Depth

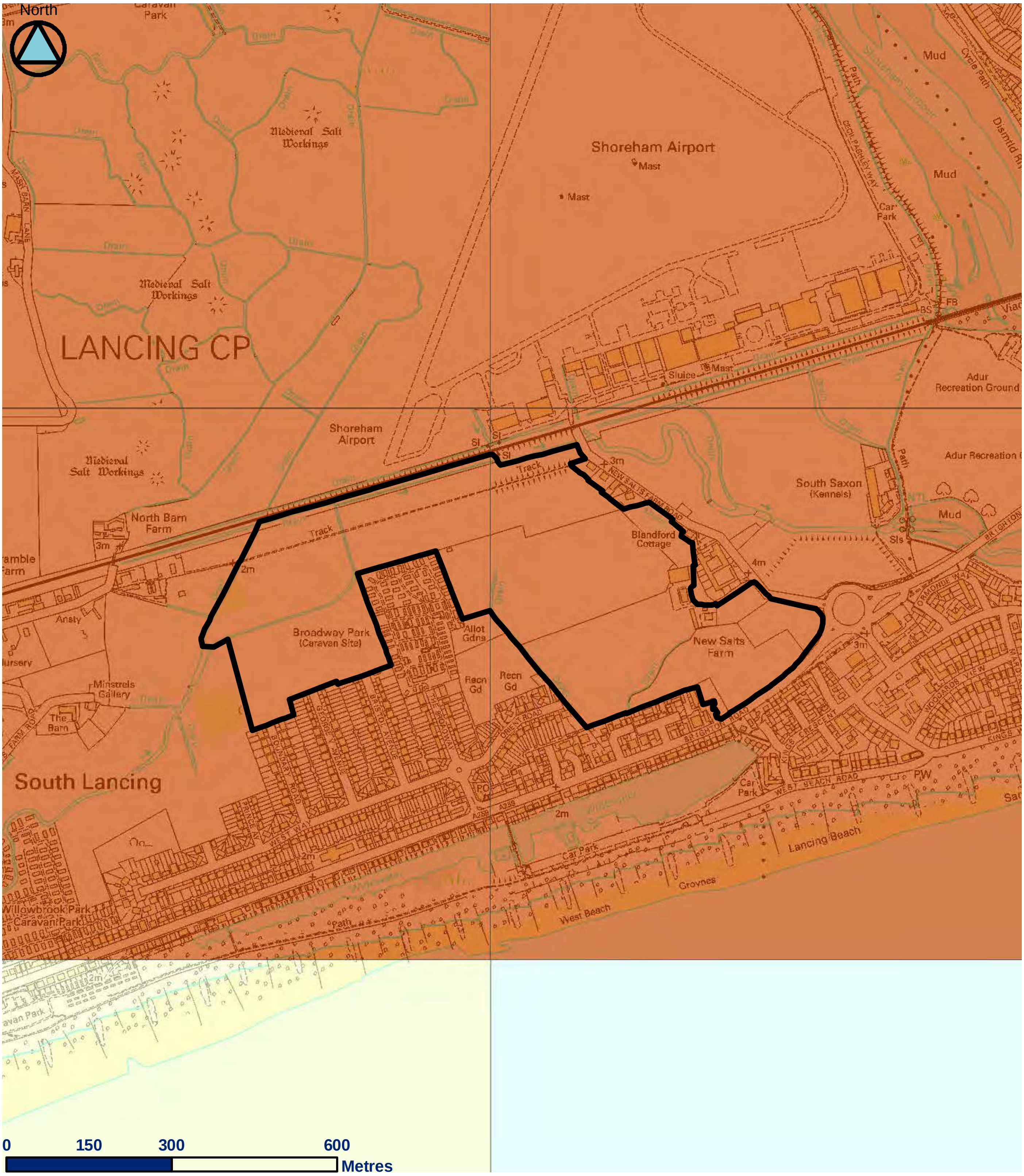
- > 0.3m
- > 0.1m

JBA consulting

Land to the North East of the Hasler Estate

Surface Water Flood Risk
(Flood Map for Surface Water 1 in 200 year)

Adur and Worthing SFRA update
Flood Risk Summary Map



Legend

Land to the North East of the Hasler Estate

AStGWF

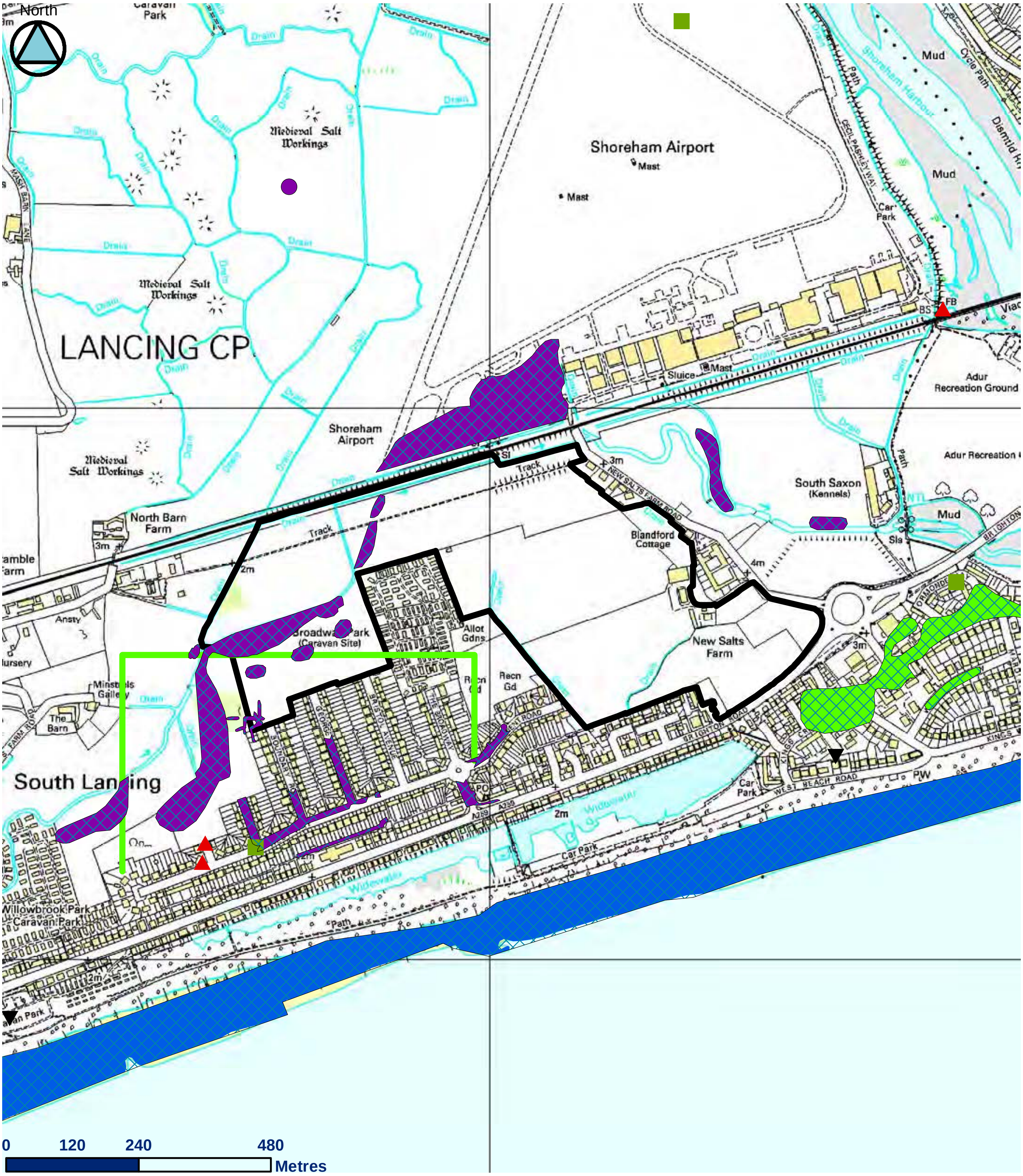
RISK

- >= 75%
- >= 50% <75%
- >= 25% <50%
- < 25%

Land to the North East of the Hasler Estate

Groundwater Flood Risk (Areas Susceptible to Groundwater Flooding)

Adur and Worthing SFRA update
Flood Risk Summary Map

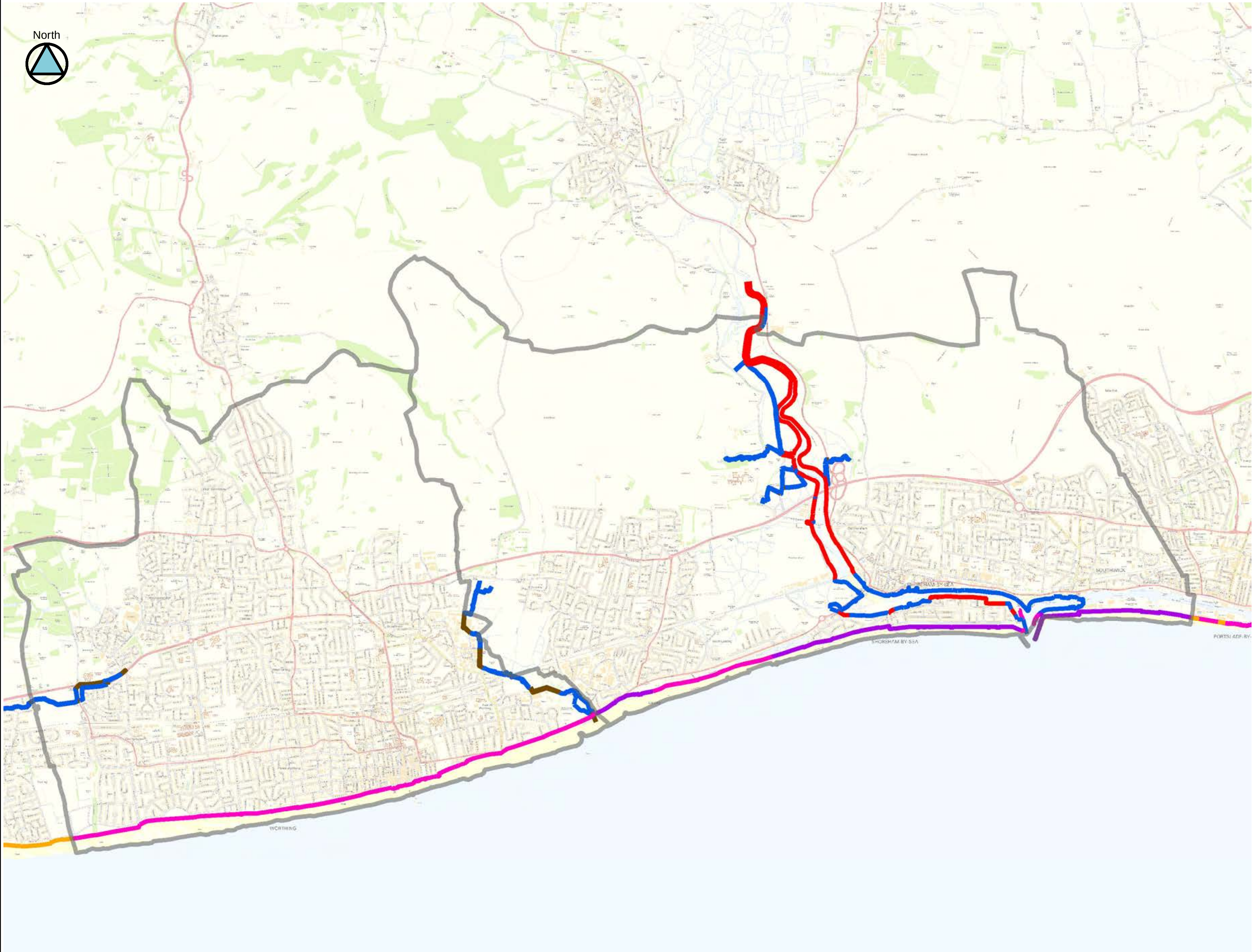


Legend

- Land to the North East of the Hasler Estate
- EA Historic Flood Map
- Fluvial Flooding
- Sewer Flooding
- Unknown Flooding
- Coastal Flooding
- Surface Water Flooding
- Surface Water Flooding
- Surface Water Flooding
- Tidal Flooding
- Tidal Flooding

Land to the North East of the Hasler Estate

Historic Flooding



LEGEND

Adur and Worthing Boundary

Flood Defence Type

- coastal protection (man-made)
- coastal protection (natural)
- culverted channel
- maintained channel
- raised defence (man-made)
- raised defence (natural)
- sea defence (man-made)
- sea defence (natural)

Map 16
Flood Defences

Scale 1:55,000

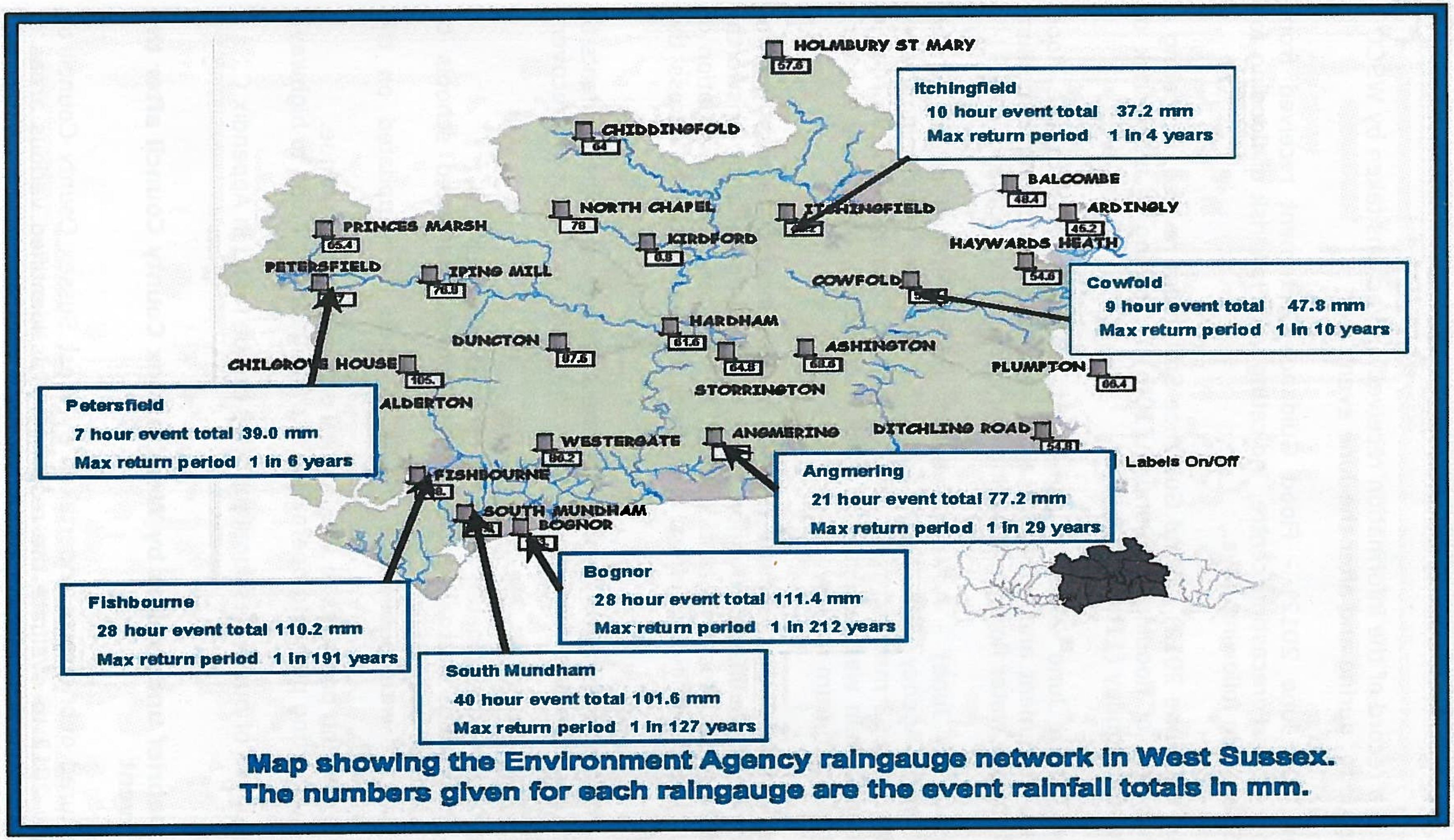


Figure 3 – Environment Agency rain gauge network across West Sussex.

Appendix H – Lancing SWMP Extracts

SECTION 4

Flooding history

4.1 Summary of flooding

There is good anecdotal evidence of flooding within Lancing from the wet winters of 2012/13 and 2013/14, and ongoing reporting from local residents throughout 2014 and 2015. Local residents have provided detailed information on the timing, location and impacts of flooding in Lancing. This has enabled a comprehensive picture of flooding to be established over the past two to three years. Flooding in Lancing has been a long-standing problem, but the best anecdotal evidence of flooding is from the last two to three years. Given that 2013/14 was the wettest winter on record it is reasonable to assume that the available anecdotal evidence from the past two to three years provides a good basis to assess the flooding impacts. Table 4-1 provides an overview of the key locations affected by flooding in Lancing.

Table 4-1 Locations affected by flooding in Lancing

Location	No. properties flooded internally[16]	Other impacts	Dates of flooding
Grinstead Lane, Manor Way, Manor Close	Two garages flooded in Manor Way	Extensive flooding on Grinstead Lane (impassable), restricted toilet use, garden flooding, and overpumping of foul network into ditch network	December 2012 and December 2013 January 2015 although flooding impacts significantly reduced
Old Shoreham Road[17]	None	Flooding on Old Shoreham Road Garden flooding	December 2012 and December 2013, January 2014 and 2015
Barfield Park and Monks Avenue	1 home affected on Barfield Park 1 property flooded near Monks Avenue/Hadlow Way	Garden flooding in other locations	December 2013, Summer 2014
The Paddocks	None, but some garages affected	Flooding on the highway	Flooding occurred regularly following heavy rainfall (until work completed (see Section 2.4.3)
West Beach Estate	None	Flooding across most of The Broadway, and parts of Westway and Prince Avenue	Flooding occurs regularly
A27	None	Northern carriageway of A27 flooded	December 2013
Shoreham Airport	None	Airport flooded, although main runway was still operational	December 2013

[16] Defined as flooding within a building, and includes the main buildings / garages of a property

[17] This refers to the cul-de-sacs south of the A27 (NB: The A27 is also known as Old Shoreham Road)

SECTION 5

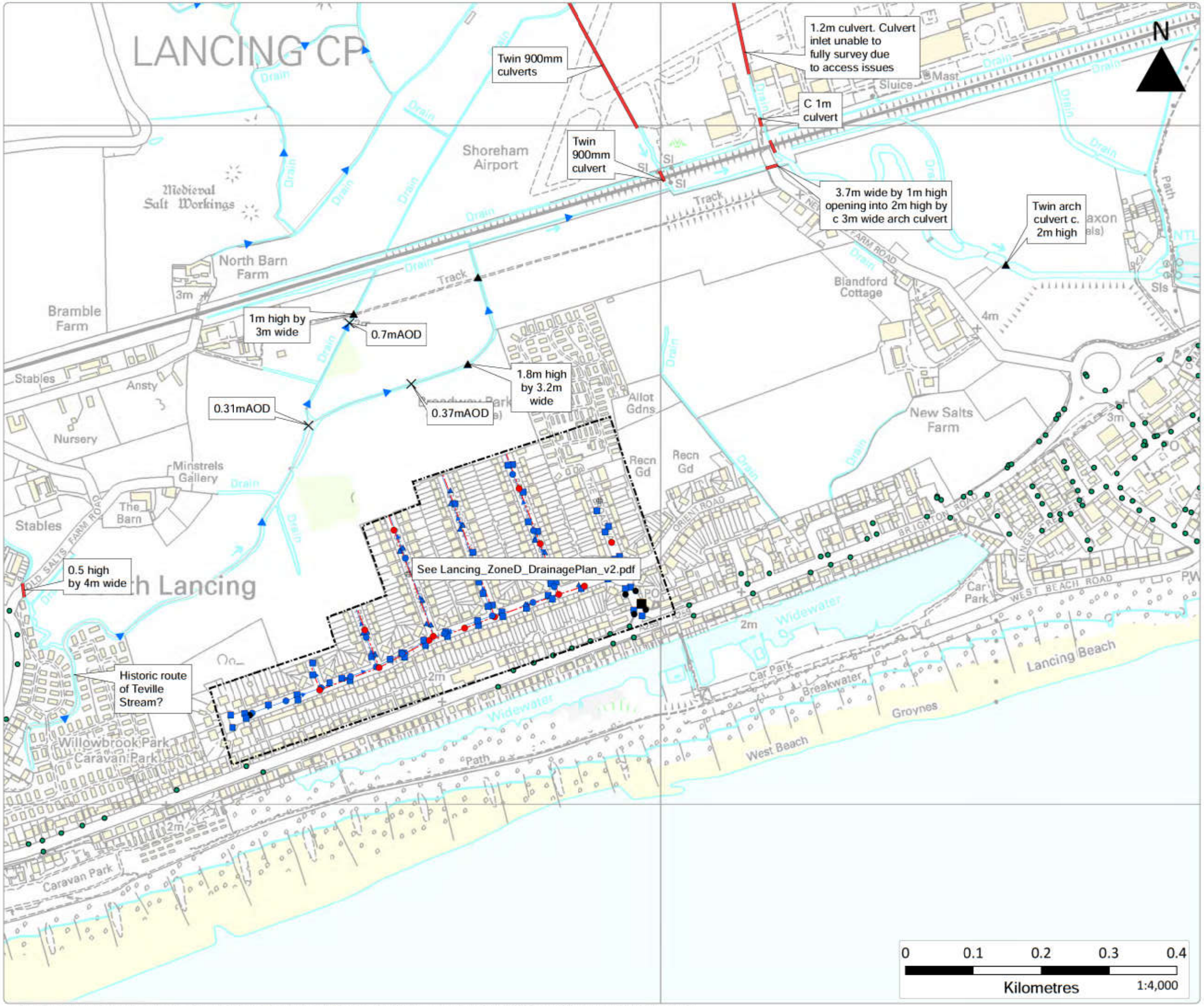
Table 5-1 *Summary of flooding reports from residents of West Beach Estate (bold & italic text identifies a linked event)*

ID	Date/Time flooding	Date/time high tide	What is flooded? And to what extent? (All are quotes from local residents)		Other useful information	Level of high tide (mAOD)	Groundwater level at Sussex PAD (mAOD)	Rainfall total (Applesham Farm)	Dry / Wet	Tide level at time of flood
			The Broadway	The Westway						
1	05/11/2013 2:30pm	05/11/2013 12:05pm				6.61	1.36	25.6mm two days earlier2.2mm	Wet	High
2	27/12/2013 7:00am	27/12/2013 05:43am	The Broadway is continually flooded from the entrance to the crossroads 6-9 inches deep		The flood Plain behind West Beach is pooling on Old Salts Farm	5.06	2.17	4.2mm on 27/11, 10.9mm on 26/11	Wet	Low
3	01/01/2014 2:00pm	01/01/2014 10:49am	Was flooded to the centre of the road but numerous cars including a lowered mini was able to enter/exit the estate area		All areas were deep, making it difficult to drive through	6.51	2.29	12.2mm on 01/01, 11.5mm on 31/12	Wet	High
4	10/08/2014 Time Unknown		The Broadway is continually flooded	West beach is continuing to flood badly all this week		Tide >6m	1.31	17.8mm on 08/08 and 4.5mm on 09/08 and 7.5mm on 10/08	Wet	High
4a	***13/08/2014 4:10pm***	***13/08/2014 1:59pm***	***Unknown***	***Flooding either side of Westway above ankle deep, but not to middle of road***		***6.96***	***1.30***	***1mm on 13/08, 12/08 dry***	***Dry***	***High***
5	25/08/2014 7:00am				Flooding of woodland area	Tide <6m	1.31	29.2mm on 25/08	Wet	Low
6	11/09/2014 3:00pm	11/09/2014 1:38pm		Flooding either side of Westway limited to one side of the road	Water bubbling up onto Westway (video). Water drained away by 5.30	6.96	1.27	Dry	Dry	High
7	08/10/2014 1:00PM	08/10/2014 11:50am	Unknown	Flooding across most of width of Westway. Fairly deep in places	The flood plain behind west beach is now flooded. Prince Avenue flooded	6.74	1.32	5.3mm 08/10 and 20.0mm on 07/10	Wet	High
8	22/10/2014 1:30PM	22/10/2014 11:06am			Prince Avenue flooded across whole width of road	5.66	1.37	Unknown	Unknown	Low
9	03/11/2014 09:00am	03/11/2014 7:50am	Broadway flooded, extent / depths unknown	Westway also flooded, extent unknown?	Roads were at 9am. About a foot of concrete was not under water. Water still sitting in Westway and Broadway at 6pm	5.62	1.40	18.3mm on 02/11 and 13.8mm 03/11	Wet	Low
9a	***07/11/2014 12:15pm***	***07/11/2014 11:13am***	***Broadway flooded, extent / depths unknown***	***Westway flooded, half way across the road but quite deep. Also looks to be flowing***		***6.54***	***1.42***	***5.1mm on 06/11 and 5.2mm on 07/11***	***Dry***	***High***
9b	***08/11/2014 1:30pm***	***08/11/2014 11:54am***	***Broadway flooded, extent / depths unknown***	***Flooding across most of width of Westway. Fairly deep in places***	***Also flooded 9th November 2014, high tide was 6.32m AOD***	***6.49***	***1.43***	***13.7mm on 08/11***	***Wet***	***High***

ID	Date/Time flooding	Date/time high tide	What is flooded? And to what extent? (All are quotes from local residents)		Other useful information	Level of high tide (mAOD)	Groundwater level at Sussex PAD (mAOD)	Rainfall total (Applesham Farm)	Dry / Wet	Tide level at time of flood
			The Broadway	The Westway						
9c	***10/11/2014 3.25pm***	***10/11/2014 1:13pm***	***Broadway badly flooded across whole length of road, buses couldn't stop at edge of road***		***Orient Road flooded near junction with Broadway***	***6.05***	***1.51***	***0.3mm on 10/11***	***Dry***	***High***
10	23/11/2014 1:00pm	23/11/2014 11:26am	The Broadway / Orient Rd are flooding badly again. Flooding almost across the total width of the road	Limited flooding on Westway?	Broadway north of junction also flooded one side of the road	6.24	1.83	9.4 on 22/11 and 15.3mm on 23/11	Wet	High
10a	***28/11/2014 Time Unknown***		***Refer to previous photographs of the flooding on the Broadway and crossroads of Westway / Orient Road***		***The flooding has been constant and not abated ,yesterday it was across the entire road and causing problems with the flow of traffic especially smaller cars which have to travel down the centre of the road***	***Tide <6m in afternoon***	***1.91***	***10.8mm on 25/11, .88mm on 26/11 and 0.3mm on 28/11***	***Wet***	***Low***
11	12/12/2014 09:30am		Broadway and crossroads Orient Road and Westway almost across road near junction with A259		Just to keep you informed The Broadway is still partially flooded 2 days after the precipitation	Not linked to high tide	1.85	14.7mm on 11/12 and 0.2mm on 12/11	Wet	Low
11a	***17/12/2014 3:30pm***		***The Broadway is badly flooded nearly to the middle of the road but still passable. Corner of Orient Flooded***	***No flooding on Westway***	***Low tide but heavy precipitation.***	***Tide <5m***	***1.76***	***13.2mm on 16/12 and 1.4mm on 17/12***	***Wet***	***Low***
11b	***19/12/2014 12:00pm***		***Broadway flooded, but less water compared to 2 days earlier. Still across most of road***			***Tide <5.5m***	***1.72***	***6.8mm on 18/12 and 0.1mm on 19/12***	***Dry***	***Low***
12	03/01/2015 Time Unknown		The Broadway badly flooded , cascading water off A 259 , reaching middle of road etc as previous	No flooding on Westway	Unsure of time of flood, email was as 12:47pm	Tide <5.75m	1.81	9.4mm on 02/01 and 5.6mm on 03/01	Wet	Low
13	08-01-2015 11:00am		The Broadway badly flooded , cascading water off A 259 , reaching middle of road etc as previous	Flooding Westway half way across road, focussed on area between George V and Bristol Avenue		Not related to tide level	1.88	23.6mm on 07/01 and 17.9mm on 08/01	Wet	Low
14	14-01-2015 3:35pm		The Broadway junction floods in all four directions (into Orient Rd, up and down Broadway as well as into the Westway). If it rains heavily, the whole junction floods heavily	Not flooded		Low tide 2 hours earlier	2.50	11.3mm on 14/01	Wet	Low
14a	***17-01-2015 11.30am***		***Majority of Broadway flooded***	***Not flooded***		***High tide @ 8am, only 5.2m AOD***	***2.79***	***4.7mm on 17/01***	***Dry***	***Low***
15	23-01-15 2.30pm	23-01-15 @1.30pm	Flooding but still receding	Water bubbling up through Westway, but only isolated flooding		6.83	2.60	6.6mm on 23/01	Dry	High

ID	Date/Time flooding	Date/time high tide	What is flooded? And to what extent? (All are quotes from local residents)		Other useful information	Level of high tide (mAOD)	Groundwater level at Sussex PAD (mAOD)	Rainfall total (Applesham Farm)	Dry / Wet	Tide level at time of flood
			The Broadway	The Westway						
16	13-02-2015 4.00pm		The Broadway is flooded to about a quarter due to surface water runoff from the A 259 after precipitation this	pooling significantly along the curbs to a quarter of the Broadway and orient / Westway		High tide 4.97 @ 5pm	Data not yet available	Data not yet available	Unknown	Unknown

Lancing SWMP



Legend

- Manhole Data from WSCC
- Bed Level
- Farm Crossing
- Gullies
- Soakaway
- Culvert
- Open Channel
- Catchpit
- Foul Chamber
- Gully
- Kerb Inlet
- Surface Chamber
- Unknown Chamber
- Assumed Foul Pipe
- Assumed Surface Water Pipe

West Beach

CH2MHILL.

Created by: Helen Winter (02/04/2015)
Checked by: Ali Cotton (02/04/2015)

Document Path: W:\we\Urban Water Market Sectors\Urban Water UK\Projects\West Sussex SWMP framework\Lancing SWMP\GIS\Drainage_Plans_individual_Lancing_A2.mxd

TECHNICAL NOTE

CH2MHILL®

Lancing Site Notes - Zone D

PREPARED FOR:	Ali Cotton Mike Barker
COPY TO:	Ammar Ahmad
PREPARED BY:	Pearce, Toby/UKS
DATE:	November 13, 2014
PROJECT NUMBER:	488929

The following technical note has been prepared to summarise the findings of the Lancing SWMP site walkover.

Site Overview

Zone D is comprised of the privately owned West Beach Estate. The estate is bounded to the south by the A259. The A259 is typically 400mm higher than the highways in West Beach Estate. The northern boundary of zone D is the drainage ditch downstream of zone B and C. Figure 1 shows the zone D boundary.

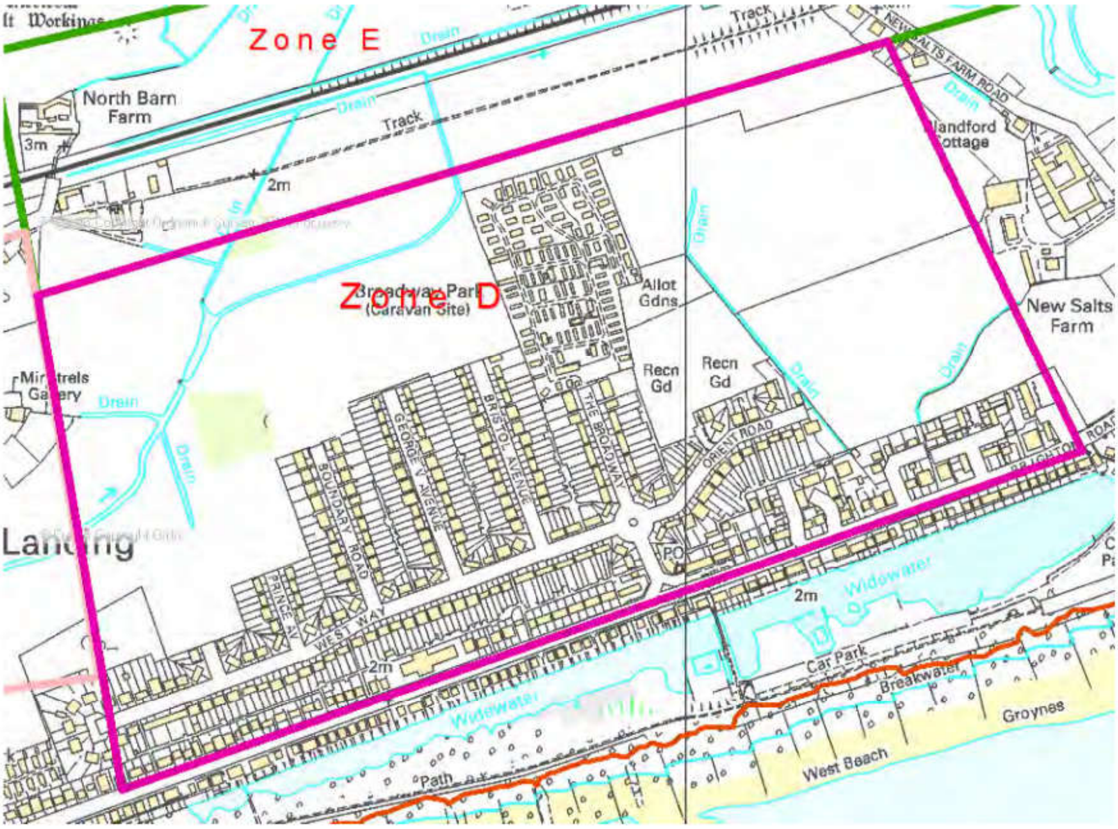


Fig.1. Zone D boundary

Ditch Network

The main ditch flows from west to east along the northern boundary of the study area. No connection was found between the surface water sewer network and the ditch network. The ditch network was heavily overgrown with reeds where it entered the zone in the west to the point where the ditch turns north adjacent to the caravan park. From this point to the northern boundary the ditch was clear of vegetation. Anecdotal evidence suggests the ditch is maintained by the landowner (farmer) in this location. Figure 2 shows the ditch where it is overgrown with reeds. Figure 3 shows the ditch where it has been cleared of vegetation.

Fig.2. vegetated section of ditch

Fig.3. Ditch maintained winter 2013

Surface Water Sewer Network

A record was collected of the location of all manholes and gullies within zone D. The survey was carried out the day after an approximately 6 hour rainfall. Siginificant flooding of the highway was observed at the entrance to West Beach Estate from the A259 'Brighton Road' as shown in figure 4. It was not possible to view inside gullies at this point due to the high water level. Although not accurately measured there was no discernable change in water level here throughout the day despite the changing tide levels. These gullies at the entrance to the estate drain to a soakaway. This area at the entrance is the lowest topographical point in the estate according to the digital terrain map (DTM).

The survey of manholes and gullies in the remainder of the estate found that surface water sewers ran in a northerly direction along all the minor roads coming of West Way. Surface water sewers were found in West Way however the connectivity of these sewers is unknown. Figure 5 shows a typical manhole along West Way at the head of Boundary Road.

Fig.4. Surface water flooding at entrance to West Beach Estate

Fig.5. Manhole at head of Boundary Road

Outlets to these surface water sewers have recently been excavated at the head of Bristol Avenue and George V Avenue. No outlets were found at the end of the other roads within the estate. Figure 6 shows the outlet at the head of George V Avenue. The existing ground level at the end the roads in West Beach Estate is typically 650 mm higher than the ground level adjacent to the ditch at the north of site D.

Fig.6. Surface water sewer outlet at end of George V Avenue

in the aquifers during the period of groundwater measurement. The monitoring showed that the aquifers layers respond separately to the influence of recharge and discharge, with a time lag in response between the two layers. Additionally, due to the clay/alluvium acting as an aquiclude the pressure in the Chalk aquifer was found to be artesian after the period of heavy rainfall during February. The Chalk aquifer in this location was therefore behaving as a confined aquifer with groundwater in the superficial layer acting as a perched water table. Based on this evidence, there is no significant contribution to surface water flows from the underlying Chalk in this location. There was found to be a tidal influence in the Chalk aquifer at BH07. The groundwater level was found to react quickly to the diurnal tidal cycle. Elevated levels of sodium and chloride were recorded for BH07D only. It was concluded that this was representative of a more direct and deeper hydraulic link to groundwater in the Chalk beneath the Adur estuary or beneath the coast.

The report recommended that the development site was not at risk of groundwater flooding, provided that the development did not disturb the geological units (i.e. development did not extend into the Chalk formation. As part of the review for the SWMP there is no evidence that would counter the conclusions of the New Monks Farm hydrogeological investigations. A summary description of the geology in the area can be found in Section 3.4.

2.4 Actions taken to alleviate flooding in the catchment

2.4.1 Clearance of Lancing Brooks

Collectively, the Monson and Royal Haskoning studies have considered the drainage ditch network in considerable detail to understand pinch points and remedial works required. Significant ditch clearance work was carried out by Adur and Worthing Councils and landowners in 2010 and 2013. In 2010 extensive ditch clearance was undertaken on the northern floodplain east of Mash Barn Lane[12], and on the southern floodplain south of the railway line (downstream of Barfield Park). Furthermore in 2013 the ditch sections which run through residential areas were dredged and cleared (beds lowered by up to 500mm)[13]. The ditch clearance work addresses most of the recommendations of the Monson and Royal Haskoning reports.

As part of the SWMP new and comprehensive cross-section survey of the ditch network was undertaken in in December 2014 and January 2015 to understand the current flow regime and levels of siltation and vegetation. The purpose of this was to assist WSCC and Adur and Worthing District Councils in identifying an optimal maintenance regime. The findings of the cross-section survey are described in Section 6. In January 2015 the landowner of the golf course development undertook a comprehensive clearance of the ditches.

2.4.2 Improvements to foul sewerage network

Since the winters of 2013/13 and 2013/14 Southern Water have undertaken a number of actions to reduce the risk of foul sewer flooding, including:

- developing an Infiltration Reduction Plan (IRP) for North Lancing which sets out the strategy for managing infiltration into the sewer network;
- sealing of the sewer network to reduce infiltration;
- installation of a level alert system which triggers a tanker call out when sewer levels go above a certain threshold, and;
- production of an Emergency Action Plan (EAP) which identifies trigger levels and associated actions depending on sewer levels and forecast flooding.

12 In January 2015 the landowner also undertook ditch clearance of the northern floodplain within the golf course area

13 Ken Argent, *pers. comm.*

In this area groundwater levels in the Chalk will be influenced by a number of factors including:

- The regional southerly and easterly groundwater flow toward the sea.
- Diurnal variations caused by the pressure response from tidal influence.
- Confined conditions from the cover of Lambeth Group and finer superficial deposits.
- Possible discharge at the Lambeth Group Boundary and through permeable windows in the superficial deposits.

Groundwater levels in the Superficial Deposits will be influenced by:

- Upward groundwater movement from the Chalk brought on by a response to changing groundwater pressure.
- Local recharges and discharges (including drainage from soakaways).
- The amount of groundwater storage available based on the extent and nature of the superficial deposits.
- Response to tidal influence, preventing seaward discharge through the gravels and backing up groundwater levels inland

Seasonal groundwater level variation in the area (particularly in the Chalk) is most likely to occur as a pressure response, but as this is an area primarily of discharge of regional groundwater flow , these seasonal variations are likely to be less marked than further inland (i.e. there will be a lesser variation between maximum and minimum levels). Supported both by upward leakage from the Chalk and local recharge, groundwater levels in the superficial deposits are likely to remain high most of the year, although there will be some recession through the autumn months.

On this basis, groundwater levels recorded at Sussex Pad may not be wholly representative of groundwater behaviour in the Chalk beneath the site. However, there remains value in comparing monitored groundwater levels with tidal levels, rainfall and the occurrence of flooding (see Figure 5.3 below).

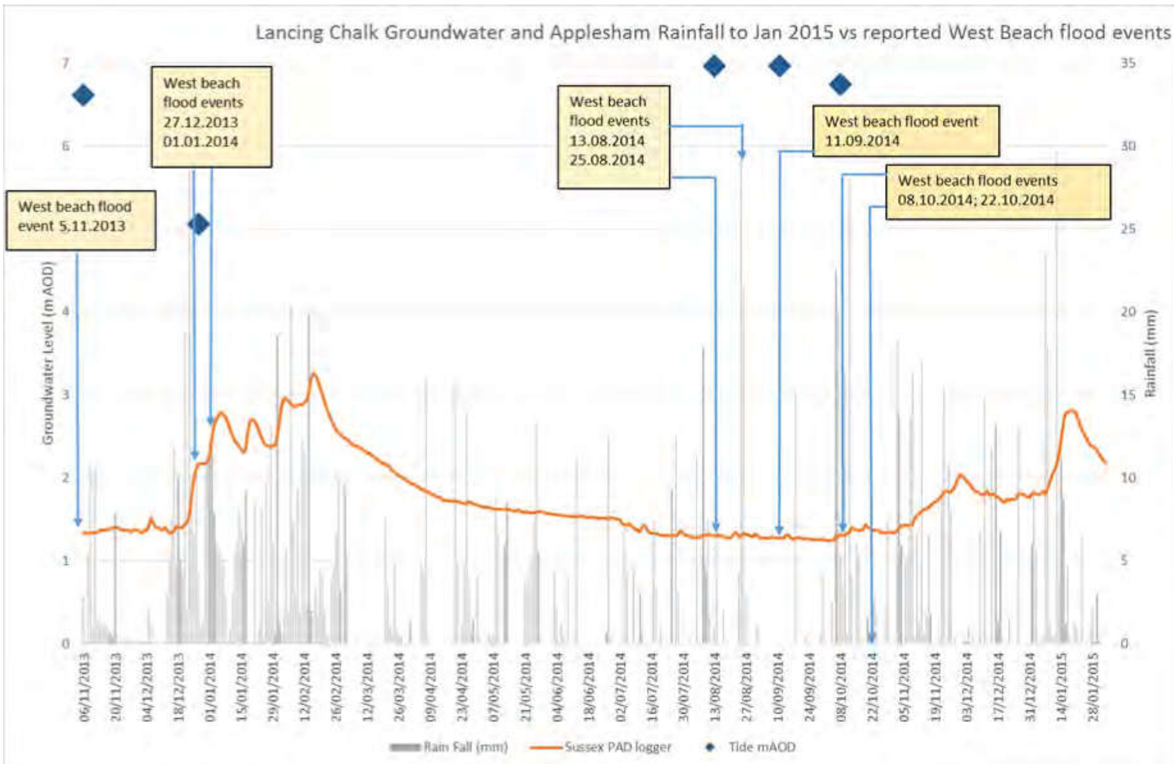
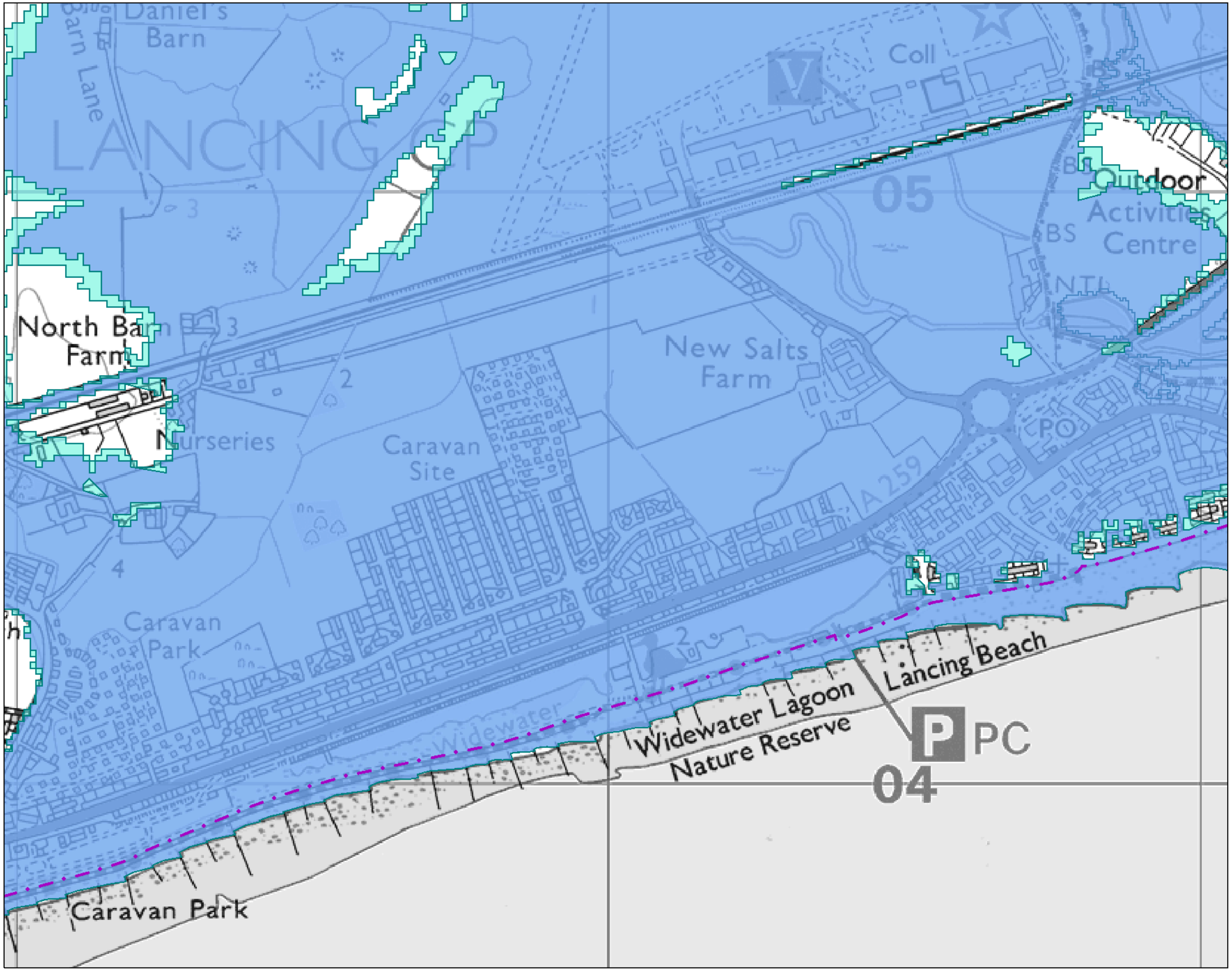


Figure 5.3 Groundwater and Tidal Levels and Rainfall vs flooding recorded in West Beach Area

Appendix I– EA Flood Maps

Flood Map for Planning (Rivers & Sea) centred on New Salts Farm, Shoreham - created 19/05/2015



Environment Agency

Scale 1:10,001

Flood Map for Planning (Rivers and Sea)

- Flood Map - Defences
- Areas Benefiting from Flood Defences
- Flood Map - Flood Storage Areas
- Flood Map - Flood Zone 3
- Flood Map - Flood Zone 2

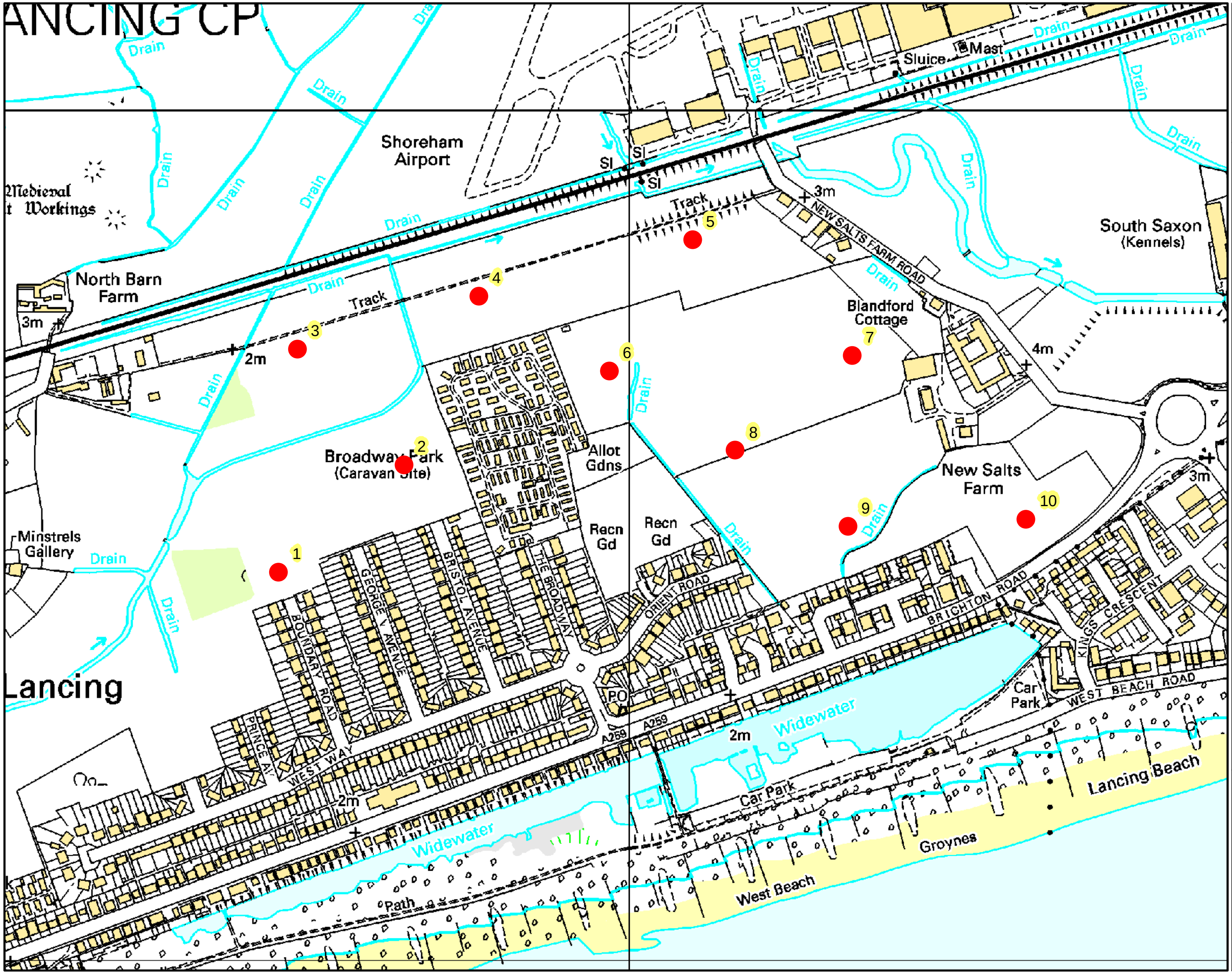
Flood Map for Planning (Rivers and Sea) (assuming no defences)

Flood Zone 3 shows the area that could be affected by flooding:
- from the sea with a 1 in 200 or greater chance of happening each year
- or from a river with a 1 in 100 or greater chance of happening each year.

Flood Zone 2 shows the extent of an extreme flood from rivers or the sea with up to a 1 in 1000 chance of occurring each year.

Contact Us: National Customer Contact Centre, PO Box 544, Rotherham, S60 1BY. Tel: 03708 506 506 (Mon-Fri 8-6). Email: enquiries@environment-agency.gov.uk

Flood Levels Location Points



Environment Agency

Legend

● Location Points

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*when printed at A4.

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Metres

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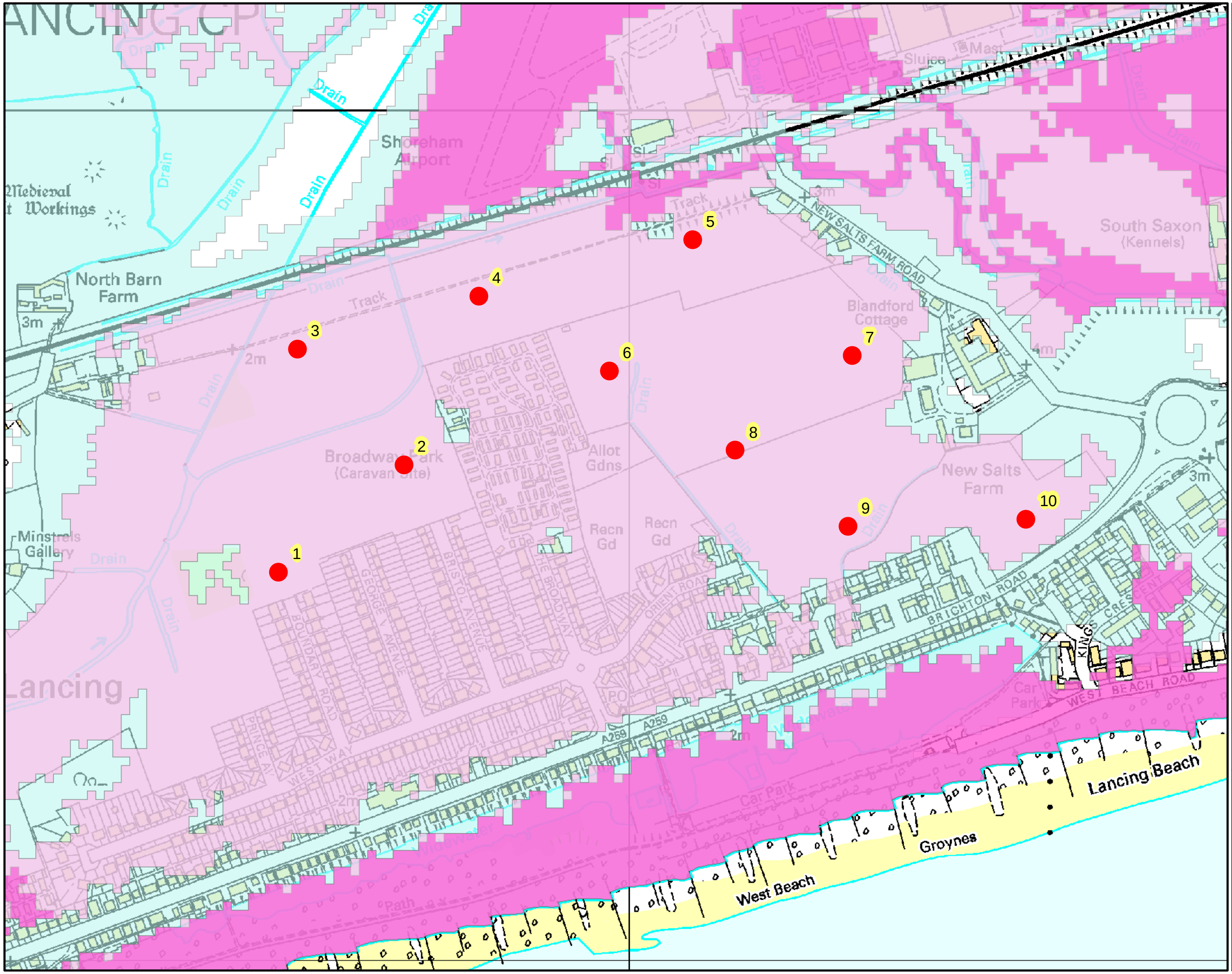
			1 in 75 undefended		1 in 200 undefended		1 in 200+cc undefended		1 in 1000 undefended	
Point	Easting	Northing	depth	height	depth	height	depth	height	depth	height
1	519,585	104,456	2.468	4.111	2.607	4.250	3.756	5.399	2.835	4.478
2	519,735	104,581	2.405	4.113	2.544	4.252	3.687	5.395	2.770	4.479
3	519,609	104,716	2.388	4.116	2.526	4.255	3.667	5.395	2.753	4.481
4	519,823	104,779	2.996	4.117	3.136	4.256	4.270	5.391	3.360	4.481
5	520,075	104,847	2.541	4.119	2.681	4.258	3.810	5.388	2.904	4.482
6	519,976	104,693	2.548	4.116	2.688	4.256	3.823	5.390	2.912	4.480
7	520,261	104,709	2.430	4.116	2.571	4.257	3.704	5.390	2.794	4.481
8	520,125	104,600	2.376	4.114	2.517	4.255	3.653	5.392	2.741	4.479
9	520,257	104,508	2.563	4.113	2.705	4.255	3.842	5.393	2.929	4.479
10	520,468	104,520	2.252	4.115	2.395	4.258	3.529	5.391	2.618	4.481

depth = metres height = mAOD

			1 in 75 defended		1 in 200 defended		1 in 200+cc defended		1 in 1000 defended	
Point	Easting	Northing	depth	height	depth	height	depth	height	depth	height
1	519,587	104,452	0.336	1.970	0.786	2.420	3.415	5.050	1.649	3.284
2	519,736	104,581	0.261	1.970	0.712	2.420	3.341	5.050	1.575	3.283
3	519,612	104,717	0.304	1.970	0.755	2.420	3.383	5.049	1.617	3.283
4	519,823	104,780	0.775	1.970	1.226	2.420	3.854	5.049	2.087	3.282
5	520,073	104,847	0.398	1.970	0.849	2.420	3.477	5.048	1.711	3.282
6	519,976	104,693	0.402	1.970	0.852	2.420	3.481	5.049	1.715	3.283
7	520,263	104,710	0.358	1.970	0.808	2.420	3.437	5.049	1.672	3.284
8	520,125	104,595	0.248	1.970	0.698	2.420	3.327	5.049	1.561	3.284
9	520,255	104,509	0.420	1.970	0.870	2.420	3.499	5.049	1.734	3.285
10	520,467	104,519	0.107	1.970	0.558	2.420	3.187	5.050	1.423	3.286

depth = metres height = mAOD

Flood Extents - Taken from Arun to Adur Flood Modelling (2012)



Environment Agency

Legend

- Location Points
- 1 in 20 year defended
- 1 in 75 year defended
- 1 in 75 year undefended

1:6,544 *

*when printed at A4.

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Metres

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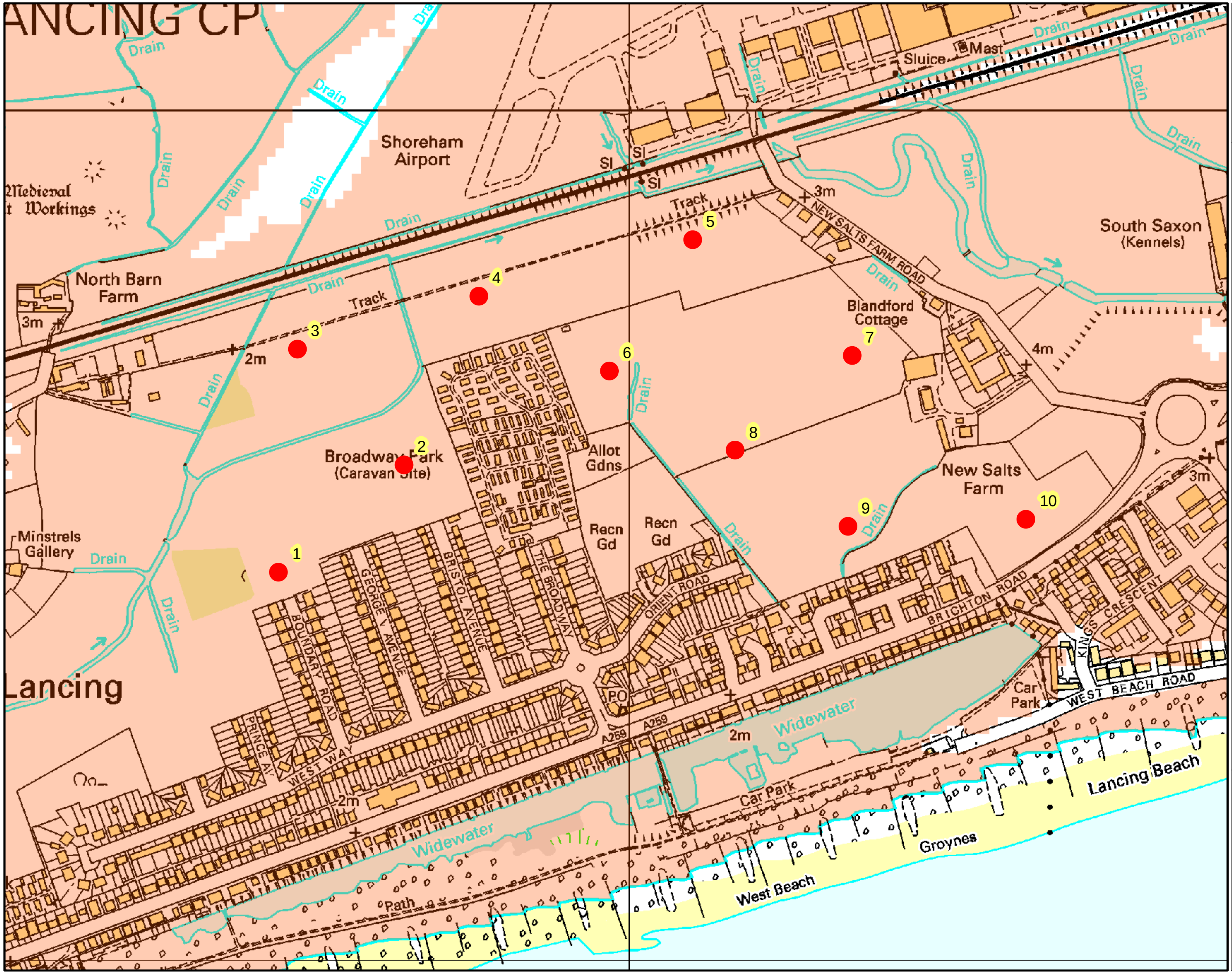
Flood Extents - Taken from Arun to Adur Flood Modelling (2012)



Environment Agency

Legend

- Location Points
- 1 in 200 year undefended

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Metres

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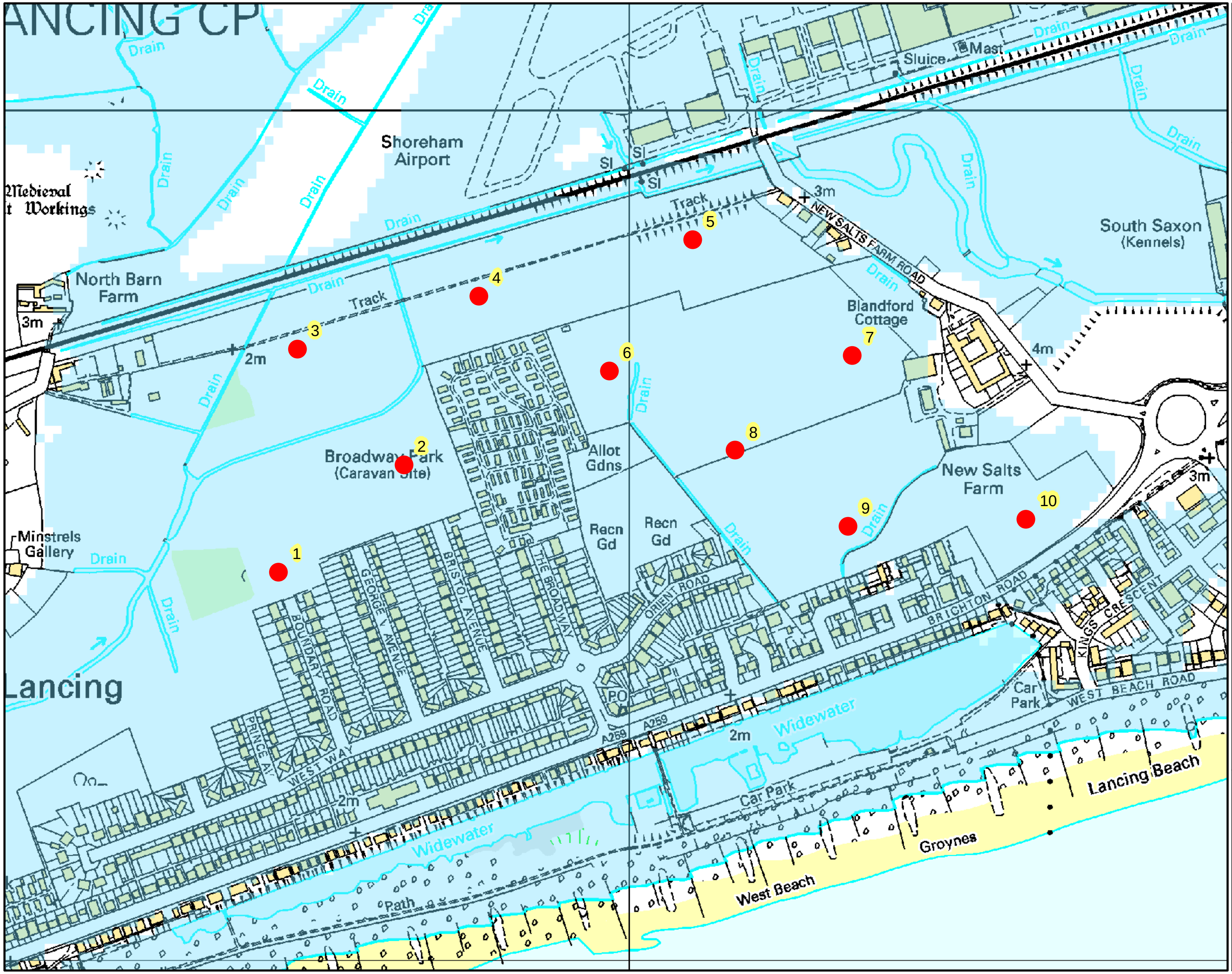

Environment Agency

Legend

- Location Points
- 1 in 200 year defended

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Metres

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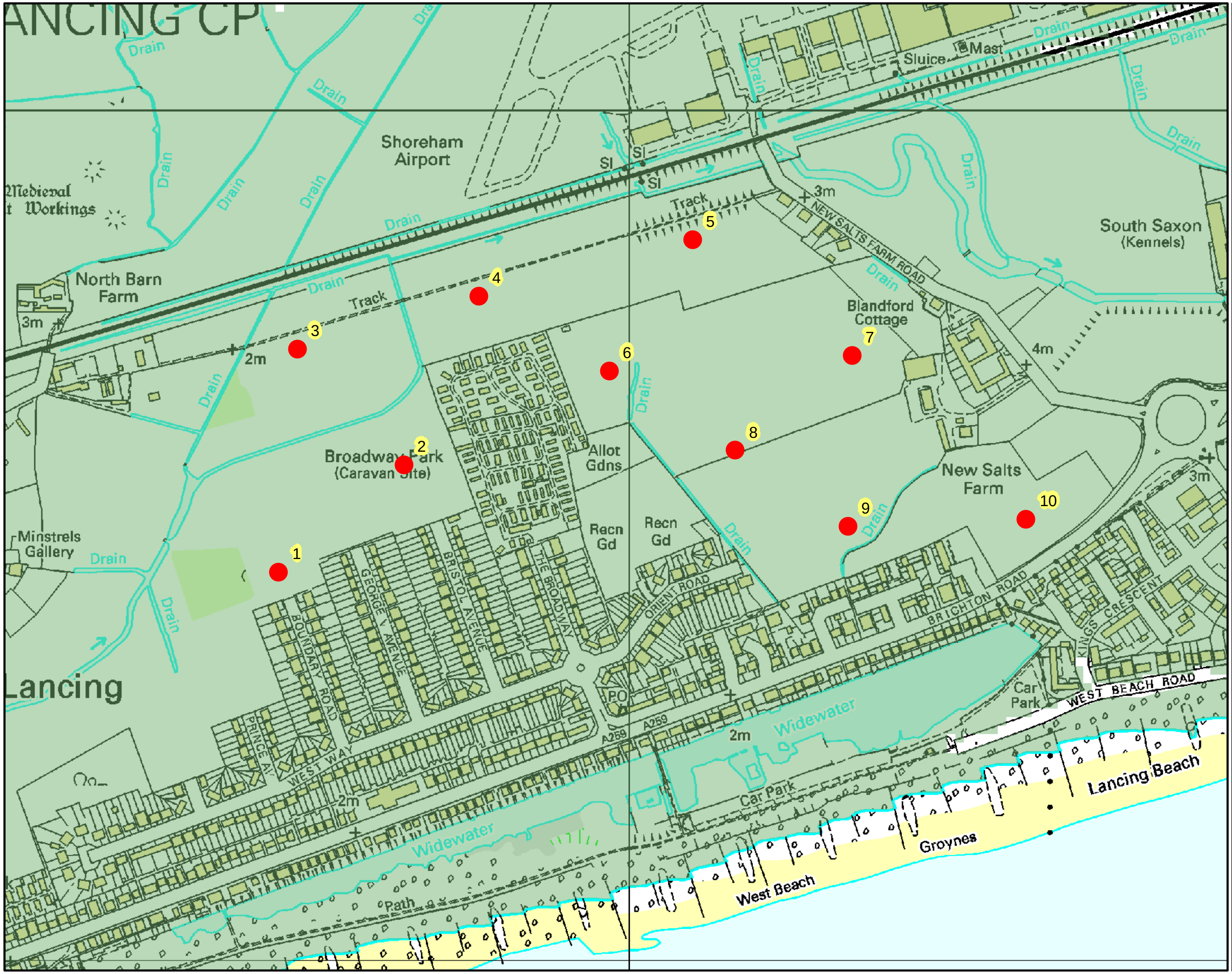

Environment Agency

Legend

- Location Points
- 1 in 200 year+cc undefended

1:6,544 *

*when printed at A4.

N

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Metres

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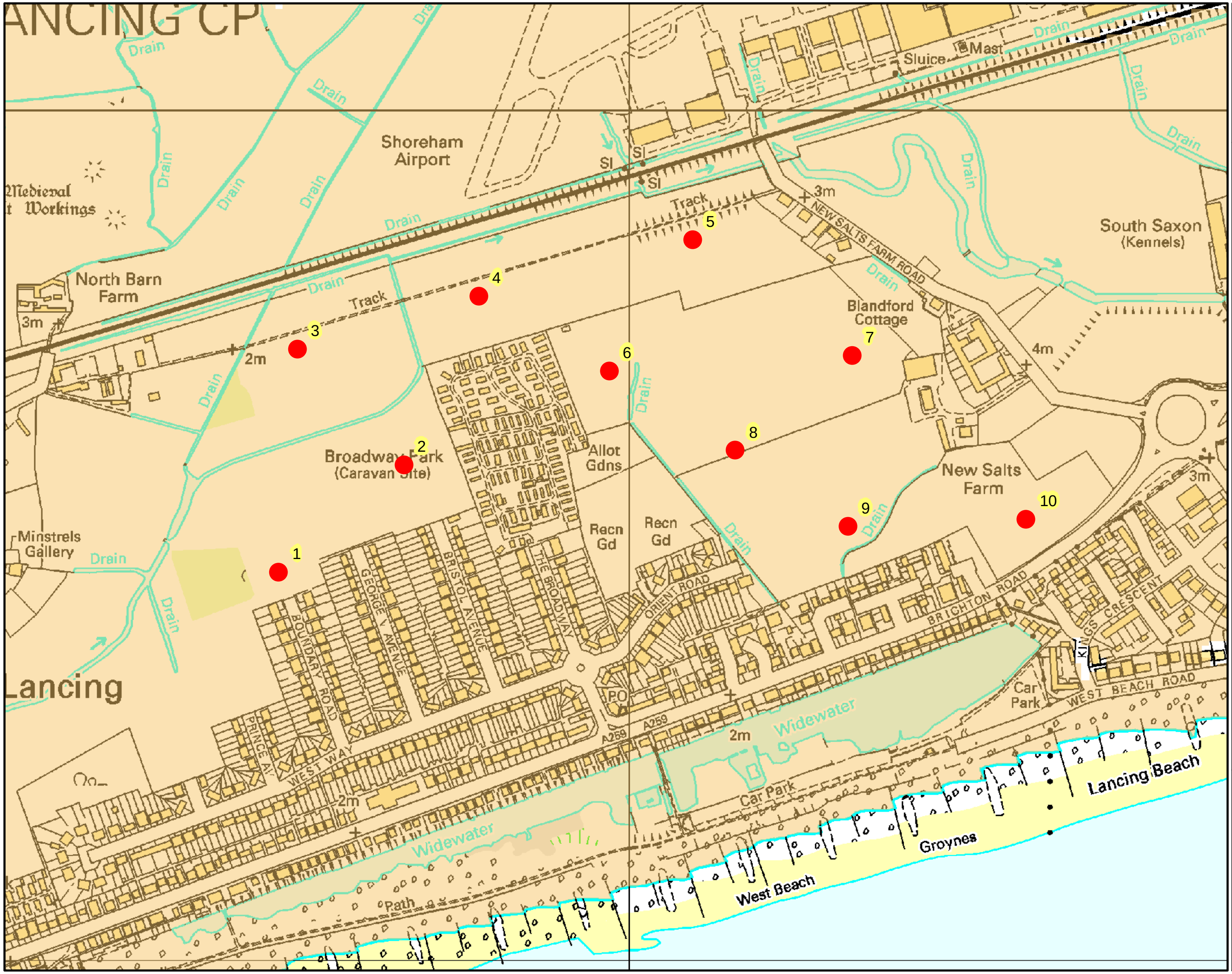
Flood Extents - Taken from Arun to Adur Flood Modelling (2012)



Legend

- Location Points
- 1 in 200 year+cc defended

1:6,544 *

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Metres

N

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8	520,125	104,600	2.376	4.114	2.517	4.255	3.653	5.392	2.741	4.479
9	520,257	104,508	2.563	4.113	2.705	4.255	3.842	5.393	2.929	4.479
10	520,468	104,520	2.252	4.115	2.395	4.258	3.529	5.391	2.618	4.481

depth = metres height = mAOD

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4	519,823	104,780	0.775	1.970	1.226	2.420	3.854	5.049	2.087	3.282
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7	520,263	104,710	0.358	1.970	0.808	2.420	3.437	5.049	1.672	3.284
8	520,125	104,595	0.248	1.970	0.698	2.420	3.327	5.049	1.561	3.284
9	520,255	104,509	0.420	1.970	0.870	2.420	3.499	5.049	1.734	3.285
10	520,467	104,519	0.107	1.970	0.558	2.420	3.187	5.050	1.423	3.286

depth = metres height = mAOD

Appendix J – Indicative Drainage Drawing and Calculations

Bioretention Basin

Contributing area:		
Roads/Driveways	–	1.60 Ha
Blue Roofs	–	1.20 Ha
Detention Basin	–	3.00 Ha
Greenfield	–	7.60 Ha
Base Level	–	1.100m AOD
Top water Level	–	1.359m AOD
Top of Bank Level	–	1.600m AOD
Volume	–	7445m^3

Drainage Strategy

The eastern half of the site has been analysed (Area 1) as the most onerous section of the site. The analysis area covers 14.69Ha of the total 28Ha site coverage.

The surface water flows from Area 1 have been modelled to replicate high ground water conditions for the 100yr + 40% climate change critical storm event. The model doesn't allow for any natural infiltration.

The rainfall model is based on FEH Catchment data provided by Centre for Ecology & Hydrology.

The proposed discharge rates for the site have been limited to Greenfield run–off (Qbar).

Qbar for the 28Ha site is calculated as 95.4l/s.

The discharge rate for Area 1 is set at 52.4l/s.

Attenuation requirement are determined by a maximum allowable discharge rate of 52.4l/s for the 100yr + 40% climate change critical storm event.

Contributing areas to the drainage network is as follows:

- Roads/Driveways (assumed 100% impermeable) 1.60Ha
- Blue Roofs, to all buildings (assumed 100% impermeable) 1.20Ha
- Detention Basin (assumed 100% impermeable) 3.00Ha
- Greenfield (remainder of site area assumed rate at Qbar) 7.60Ha

General Notes

Legend

Analysis Area

Road Levels (A.O.D)
+ 1.80m

Swale/Piped ditch
(Conveyance/Infiltration – swale 400–500mm deep)
(Conveyance – piped at road crossings

Bioretention Basin
(Infiltration – exceedance storage set 200–300mm below adjacent ground levels)

Existing Watercourse/ditch
Drain

Drainage Channel
(Drainage channel 15m wide x 1.5m deep)

All dwellings to have a blue roof discharging into permeable paving sub–base at <0.5l/s.

All highways, parking areas and private drives to be formed of permeable paving (full infiltration). Sub–base structure to acomodate storage requirements of 100yr +40% climate change critical storm event.

Testing should be undertaken for each development phase to establish the infiltration rate. In the event of infiltration not being suitable in localised areas surface water will be discharged to swales and ditches.

INFORMATION

REV	DATE	DESCRIPTION	BY	CHK'D
B	12.07.17	Architect's Layout updated and drainage amended to suit.	CH	JCT
A	10.03.17	Detention basin updated in accordance with revised hydraulic calculations	CH	.

hyde

TITLE:
Area 1
High Ground Water Scenario
(No Infiltration)

PROJECT:
New Salts Farm

SCALE: 1:1000@A1 DATE: Feb'17 DRAWN: CH CHK'D: JT

DRG NO. 11649–CIV–120 REV. B

Tully De'Ath consultants
Engineering at its Best
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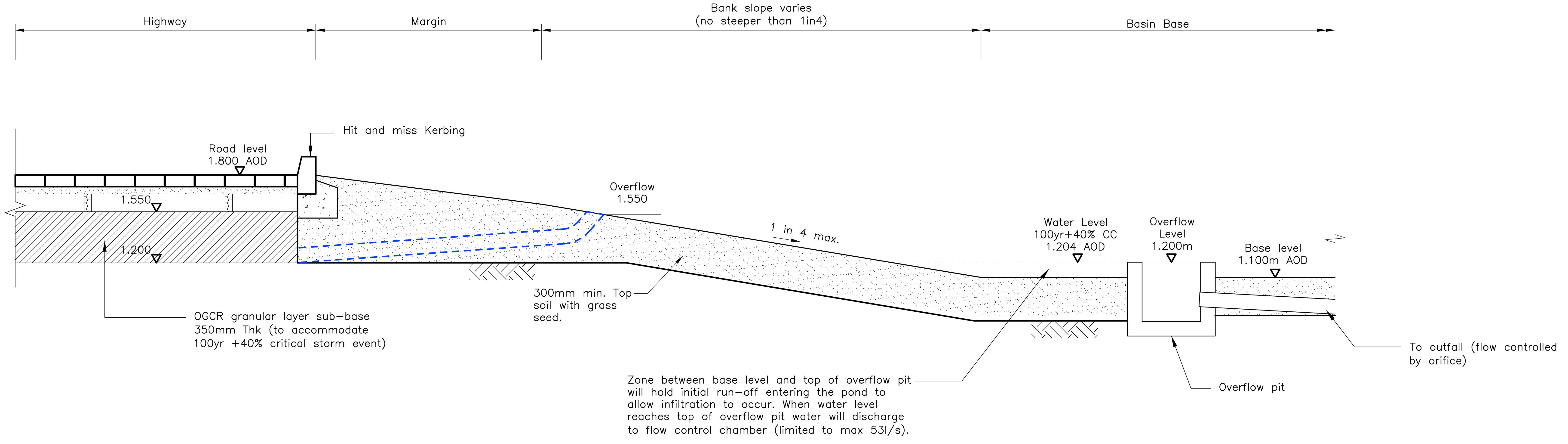
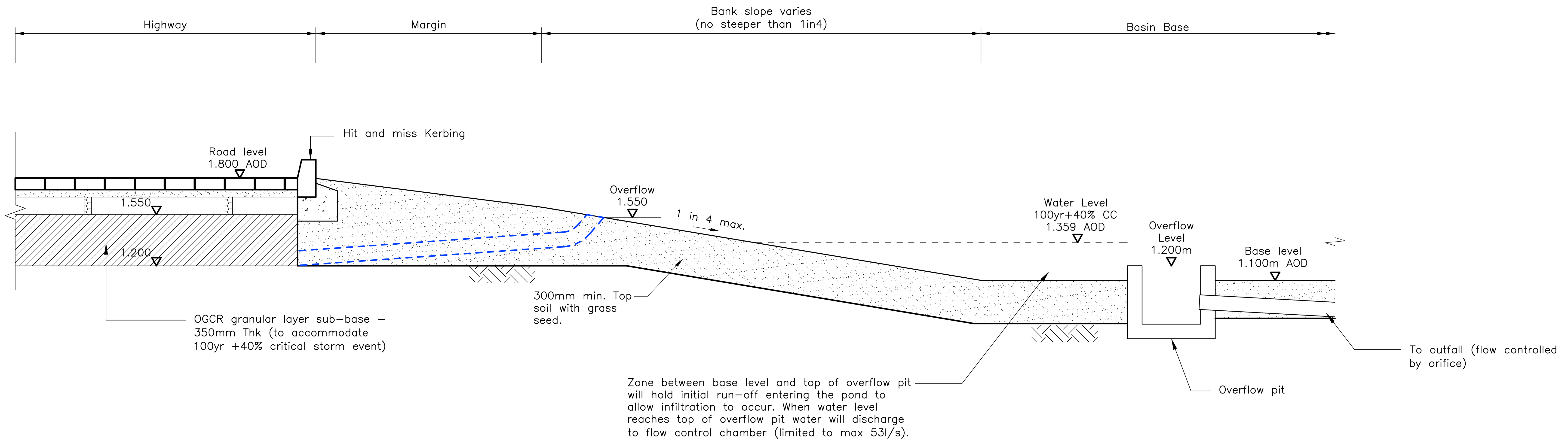
W: www.tullydeath.com



Bioretention Basin – Infiltration Scenario



Bioretention Basin – Non-Infiltration Scenario

General Notes

1. All dimensions are in millimetres unless otherwise stated.
2. All buried concrete used in works shall comply with BRE Special Digest SD1 for the following aggressive conditions in the ground:
   – Design sulphate class DS–1s.
   – Aggressive chemical environment class AC–1s.
   – Design concrete class DC–1.
   – No additional protective measures (APMs).
3. All road and hardstanding works to be carried out in accordance with the current edition of the Specification for Highway Works.
4. All hardstanding construction to be founded in sub-soil free of any organic material.

REV	DATE	DESCRIPTION	BY	CHK'D
A	12.07.17	Details amended in accordance with latest drainage calculations.	JSR	JCT

hyde

TITLE:
Drainage Details
Bioretention Basin

PROJECT:
New Salts Farm

SCALE: 1:20@A1 DATE: Feb'17 DRAWN: CH CHK'D: JCT

DRG NO.: 11649–CIV–SK200 REV.: A

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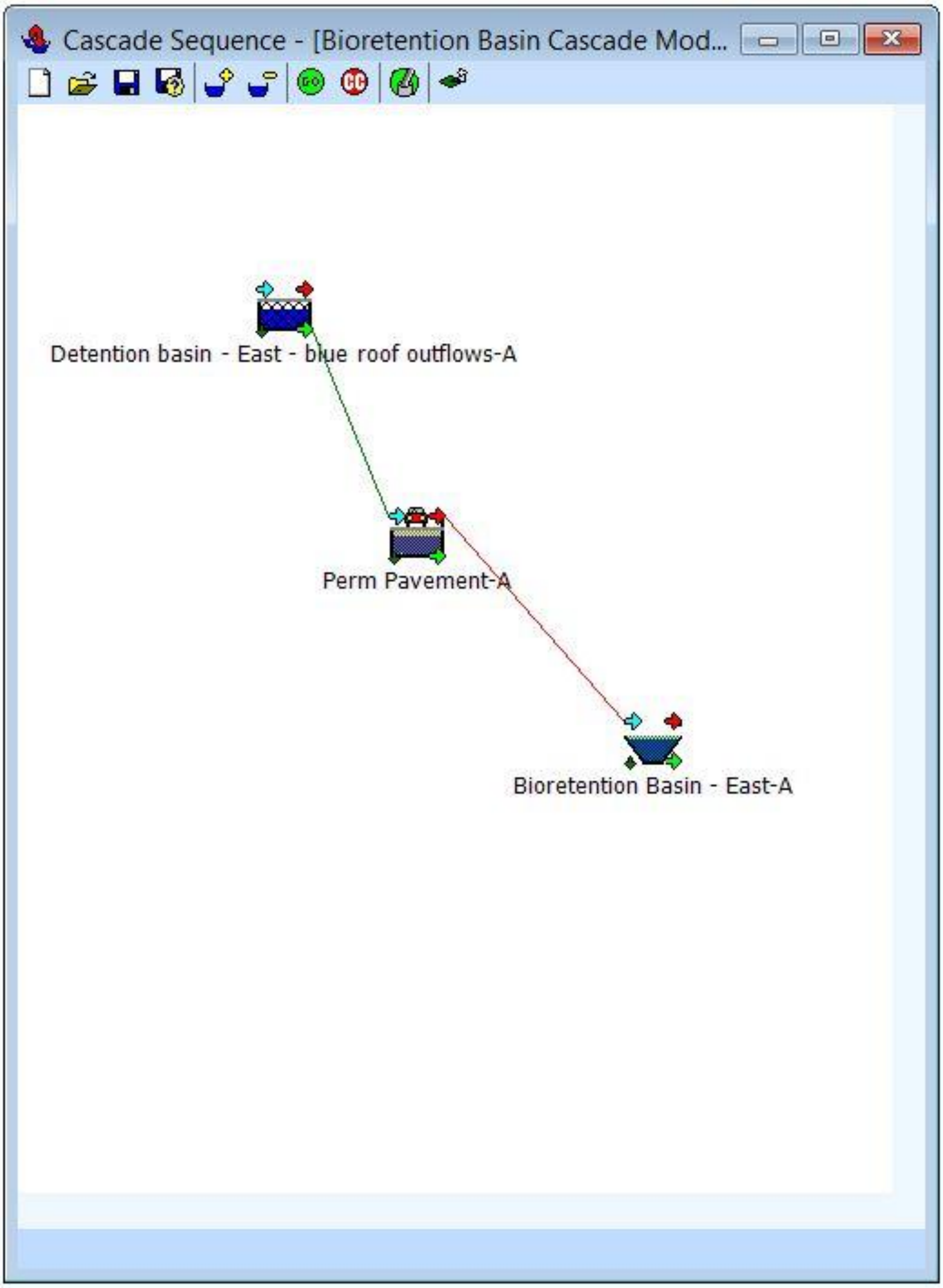
New Salts Farm   11649

Microdrainage Calculations

Revision A

11th July 2017



Cascade Schematic

Infiltration scenario

Infiltration based on recommend design rate of $7x10^{-6}$m/s derived from BRE 365 infiltration testing undertaken on 7th and 8th April 2017. Factor of safety of 2 applied.

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Upstream Structures	Outflow To	Overflow To
(None)	Perm Pavement-A.srcx	(None)

Half Drain Time : 67 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	4.936	0.036	0.0	70.0	70.0	411.4	Flood Risk
30 min Summer	4.940	0.040	0.0	70.0	70.0	457.0	Flood Risk
60 min Summer	4.941	0.041	0.0	70.0	70.0	468.7	Flood Risk
120 min Summer	4.939	0.039	0.0	70.0	70.0	448.8	Flood Risk
180 min Summer	4.937	0.037	0.0	70.0	70.0	427.3	Flood Risk
240 min Summer	4.935	0.035	0.0	70.0	70.0	403.2	Flood Risk
360 min Summer	4.930	0.030	0.0	70.0	70.0	346.9	Flood Risk
480 min Summer	4.926	0.026	0.0	70.0	70.0	292.5	Flood Risk
600 min Summer	4.921	0.021	0.0	70.0	70.0	240.7	Flood Risk
720 min Summer	4.917	0.017	0.0	70.0	70.0	195.7	Flood Risk
960 min Summer	4.910	0.010	0.0	70.0	70.0	108.4	Flood Risk
1440 min Summer	4.902	0.002	0.0	70.0	70.0	18.8	Flood Risk
2160 min Summer	4.901	0.001	0.0	52.5	52.5	9.6	Flood Risk
2880 min Summer	4.901	0.001	0.0	52.5	52.5	9.6	Flood Risk
4320 min Summer	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
5760 min Summer	4.901	0.001	0.0	52.5	52.5	9.7	Flood Risk
7200 min Summer	4.901	0.001	0.0	52.5	52.5	9.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	473.6	17
30 min Summer	107.772	0.0	575.4	31
60 min Summer	65.491	0.0	703.5	58
120 min Summer	39.798	0.0	856.8	88
180 min Summer	29.739	0.0	961.4	124
240 min Summer	24.185	0.0	1038.8	158
360 min Summer	18.072	0.0	1164.4	226
480 min Summer	14.697	0.0	1264.0	290
600 min Summer	12.519	0.0	1346.2	354
720 min Summer	10.982	0.0	1418.1	414
960 min Summer	8.733	0.0	1504.6	530
1440 min Summer	6.322	0.0	1638.7	738
2160 min Summer	4.577	0.0	1779.5	1132
2880 min Summer	3.640	0.0	1887.2	1448
4320 min Summer	2.681	0.0	2085.5	2084
5760 min Summer	2.159	0.0	2261.9	2824
7200 min Summer	1.825	0.0	2387.1	3520

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	4.901	0.001	0.0	52.5	52.5	9.4	Flood Risk
10080 min Summer	4.900	0.000	0.0	16.7	16.7	0.0	Flood Risk
15 min Winter	4.936	0.036	0.0	70.0	70.0	412.3	Flood Risk
30 min Winter	4.940	0.040	0.0	70.0	70.0	460.6	Flood Risk
60 min Winter	4.942	0.042	0.0	70.0	70.0	475.6	Flood Risk
120 min Winter	4.939	0.039	0.0	70.0	70.0	443.8	Flood Risk
180 min Winter	4.936	0.036	0.0	70.0	70.0	406.4	Flood Risk
240 min Winter	4.932	0.032	0.0	70.0	70.0	361.3	Flood Risk
360 min Winter	4.924	0.024	0.0	70.0	70.0	272.0	Flood Risk
480 min Winter	4.916	0.016	0.0	70.0	70.0	186.9	Flood Risk
600 min Winter	4.910	0.010	0.0	70.0	70.0	117.6	Flood Risk
720 min Winter	4.905	0.005	0.0	70.0	70.0	59.8	Flood Risk
960 min Winter	4.901	0.001	0.0	70.0	70.0	12.9	Flood Risk
1440 min Winter	4.901	0.001	0.0	52.5	52.5	8.2	Flood Risk
2160 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
2880 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
4320 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
5760 min Winter	4.900	0.000	0.0	16.4	16.4	0.0	Flood Risk
7200 min Winter	4.900	0.000	0.0	13.8	13.8	0.0	Flood Risk
8640 min Winter	4.900	0.000	0.0	12.1	12.1	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	2480.0	4368
10080 min Summer	1.416	0.0	2569.2	0
15 min Winter	177.348	0.0	473.6	17
30 min Winter	107.772	0.0	578.2	31
60 min Winter	65.491	0.0	700.9	58
120 min Winter	39.798	0.0	857.2	94
180 min Winter	29.739	0.0	960.8	132
240 min Winter	24.185	0.0	1037.0	168
360 min Winter	18.072	0.0	1167.1	238
480 min Winter	14.697	0.0	1264.5	302
600 min Winter	12.519	0.0	1348.0	360
720 min Winter	10.982	0.0	1418.7	412
960 min Winter	8.733	0.0	1508.8	486
1440 min Winter	6.322	0.0	1638.9	722
2160 min Winter	4.577	0.0	1779.9	1304
2880 min Winter	3.640	0.0	1886.8	1336
4320 min Winter	2.681	0.0	2085.2	2140
5760 min Winter	2.159	0.0	2238.3	0
7200 min Winter	1.825	0.0	2364.8	0
8640 min Winter	1.590	0.0	2473.4	0

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
10080 min Winter	4.900	0.000	0.0	10.7	10.7	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
10080 min Winter	1.416	0.0	2569.2	0

Cascade Rainfall Details for Detention basin - East - blue roof outflows-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.200

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.200

Cascade Summary of Results for Perm Pavement-A.srcx

Upstream Structures	Outflow To	Overflow To
Detention basin - East - blue roof outflows-A.srcx	(None)	Bioretention Basin - East-A.srcx

Half Drain Time : 231 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	1.379	0.179	56.0	0.0	56.0	666.9	O K
30 min Summer	1.414	0.214	56.0	0.0	56.0	834.9	O K
60 min Summer	1.457	0.257	56.0	0.0	56.0	1039.5	O K
120 min Summer	1.505	0.305	56.0	0.0	56.0	1273.5	Flood Risk
180 min Summer	1.535	0.335	56.0	0.0	56.0	1415.4	Flood Risk
240 min Summer	1.554	0.354	56.0	7.8	63.8	1506.6	Flood Risk
360 min Summer	1.561	0.361	56.0	38.0	94.0	1538.6	Flood Risk
480 min Summer	1.561	0.361	56.0	38.0	94.0	1539.8	Flood Risk
600 min Summer	1.561	0.361	56.0	40.7	96.7	1540.8	Flood Risk
720 min Summer	1.561	0.361	56.0	40.7	96.7	1540.9	Flood Risk
960 min Summer	1.548	0.348	56.0	0.0	56.0	1480.2	Flood Risk
1440 min Summer	1.498	0.298	56.0	0.0	56.0	1240.2	O K
2160 min Summer	1.432	0.232	56.0	0.0	56.0	922.0	O K
2880 min Summer	1.377	0.177	56.0	0.0	56.0	660.0	O K
4320 min Summer	1.310	0.110	56.0	0.0	56.0	334.1	O K
5760 min Summer	1.281	0.081	56.0	0.0	56.0	197.7	O K
7200 min Summer	1.271	0.071	49.9	0.0	49.9	151.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	0.0	114
30 min Summer	107.772	0.0	0.0	139
60 min Summer	65.491	0.0	0.0	172
120 min Summer	39.798	0.0	0.0	218
180 min Summer	29.739	0.0	0.0	258
240 min Summer	24.185	0.0	9.8	292
360 min Summer	18.072	0.0	127.2	348
480 min Summer	14.697	0.0	131.2	404
600 min Summer	12.519	0.0	115.7	458
720 min Summer	10.982	0.0	93.9	510
960 min Summer	8.733	0.0	0.0	626
1440 min Summer	6.322	0.0	0.0	894
2160 min Summer	4.577	0.0	0.0	1280
2880 min Summer	3.640	0.0	0.0	1644
4320 min Summer	2.681	0.0	0.0	2324
5760 min Summer	2.159	0.0	0.0	2944
7200 min Summer	1.825	0.0	0.0	3648

Cascade Summary of Results for Perm Pavement-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	1.263	0.063	44.3	0.0	44.3	119.7	O K
10080 min Summer	1.254	0.054	38.0	0.0	38.0	88.4	O K
15 min Winter	1.379	0.179	56.0	0.0	56.0	666.2	O K
30 min Winter	1.414	0.214	56.0	0.0	56.0	833.4	O K
60 min Winter	1.456	0.256	56.0	0.0	56.0	1036.1	O K
120 min Winter	1.505	0.305	56.0	0.0	56.0	1273.7	Flood Risk
180 min Winter	1.536	0.336	56.0	0.0	56.0	1420.5	Flood Risk
240 min Winter	1.555	0.355	56.0	11.2	67.2	1511.0	Flood Risk
360 min Winter	1.562	0.362	56.0	43.4	99.4	1543.6	Flood Risk
480 min Winter	1.562	0.362	56.0	43.4	99.4	1543.8	Flood Risk
600 min Winter	1.562	0.362	56.0	46.2	102.2	1546.8	Flood Risk
720 min Winter	1.556	0.356	56.0	16.8	72.8	1518.3	Flood Risk
960 min Winter	1.523	0.323	56.0	0.0	56.0	1360.6	Flood Risk
1440 min Winter	1.449	0.249	56.0	0.0	56.0	1001.6	O K
2160 min Winter	1.357	0.157	56.0	0.0	56.0	561.4	O K
2880 min Winter	1.296	0.096	56.0	0.0	56.0	268.6	O K
4320 min Winter	1.267	0.067	46.7	0.0	46.7	133.9	O K
5760 min Winter	1.253	0.053	37.3	0.0	37.3	85.8	O K
7200 min Winter	1.248	0.048	31.9	0.0	31.9	67.8	O K
8640 min Winter	1.245	0.045	28.0	0.0	28.0	59.5	O K
10080 min Winter	1.242	0.042	25.0	0.0	25.0	53.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	0.0	4384
10080 min Summer	1.416	0.0	0.0	5136
15 min Winter	177.348	0.0	0.0	114
30 min Winter	107.772	0.0	0.0	140
60 min Winter	65.491	0.0	0.0	172
120 min Winter	39.798	0.0	0.0	216
180 min Winter	29.739	0.0	0.0	254
240 min Winter	24.185	0.0	21.0	288
360 min Winter	18.072	0.0	147.2	334
480 min Winter	14.697	0.0	136.4	378
600 min Winter	12.519	0.0	100.1	436
720 min Winter	10.982	0.0	45.7	504
960 min Winter	8.733	0.0	0.0	670
1440 min Winter	6.322	0.0	0.0	946
2160 min Winter	4.577	0.0	0.0	1312
2880 min Winter	3.640	0.0	0.0	1620
4320 min Winter	2.681	0.0	0.0	2248
5760 min Winter	2.159	0.0	0.0	2944
7200 min Winter	1.825	0.0	0.0	3584
8640 min Winter	1.590	0.0	0.0	4376
10080 min Winter	1.416	0.0	0.0	5120

Cascade Rainfall Details for Perm Pavement-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.600

Time (mins) From:	To:	Area (ha)
0	4	1.600

Cascade Model Details for Perm Pavement-A.srcx

Storage is Online Cover Level (m) 1.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.02520	Width (m)	20.0
Membrane Percolation (mm/hr)	1000	Length (m)	800.0
Max Percolation (l/s)	4444.4	Slope (1:X)	10000.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	1.200	Membrane Depth (m)	0

Weir Overflow Control

Discharge Coef 0.544 Width (m) 20.000 Invert Level (m) 1.550

Cascade Summary of Results for Bioretention Basin - East-A.srcx

Upstream Structures	Outflow To	Overflow To
Perm Pavement-A.srcx Detention basin - East - blue roof outflows-A.srcx	(None)	(None)

Half Drain Time : 205 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	1.140	0.040	79.8	21.6	0.0	101.4	1138.8	O K
30 min Summer	1.147	0.047	93.7	21.9	0.0	115.6	1339.7	O K
60 min Summer	1.154	0.054	99.2	22.2	0.0	121.4	1534.3	O K
120 min Summer	1.161	0.061	99.2	22.5	0.0	121.7	1728.5	O K
180 min Summer	1.166	0.066	99.2	22.7	0.0	121.9	1876.5	O K
240 min Summer	1.174	0.074	99.2	23.1	0.0	122.3	2108.8	O K
360 min Summer	1.191	0.091	99.2	23.8	0.0	123.0	2582.5	O K
480 min Summer	1.199	0.099	99.2	24.2	0.0	123.4	2831.4	O K
600 min Summer	1.204	0.104	99.2	24.4	0.0	123.6	2953.1	O K
720 min Summer	1.204	0.104	99.2	24.4	0.0	123.6	2967.7	O K
960 min Summer	1.196	0.096	99.2	24.1	0.0	123.2	2749.5	O K
1440 min Summer	1.187	0.087	99.2	23.7	0.0	122.8	2481.2	O K
2160 min Summer	1.174	0.074	99.2	23.1	0.0	122.3	2094.1	O K
2880 min Summer	1.162	0.062	99.2	22.6	0.0	121.7	1755.4	O K
4320 min Summer	1.148	0.048	95.7	22.0	0.0	117.7	1374.0	O K
5760 min Summer	1.141	0.041	81.8	21.7	0.0	103.5	1176.4	O K
7200 min Summer	1.136	0.036	71.9	21.4	0.0	93.3	1025.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	2065.9	0.0	18
30 min Summer	107.772	0.0	2552.9	0.0	33
60 min Summer	65.491	0.0	3252.3	0.0	62
120 min Summer	39.798	0.0	4000.2	0.0	122
180 min Summer	29.739	0.0	4522.8	0.0	182
240 min Summer	24.185	0.0	4948.6	0.0	340
360 min Summer	18.072	0.0	5720.7	0.0	420
480 min Summer	14.697	0.0	6246.9	0.0	488
600 min Summer	12.519	0.0	6672.5	0.1	600
720 min Summer	10.982	0.0	7036.4	0.2	672
960 min Summer	8.733	0.0	7401.2	0.0	788
1440 min Summer	6.322	0.0	8101.5	0.0	1022
2160 min Summer	4.577	0.0	9068.9	0.0	1384
2880 min Summer	3.640	0.0	9665.6	0.0	1732
4320 min Summer	2.681	0.0	10804.1	0.0	2420
5760 min Summer	2.159	0.0	12090.7	0.0	3160
7200 min Summer	1.825	0.0	12841.9	0.0	3888

Cascade Summary of Results for Bioretention Basin - East-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	1.132	0.032	64.0	21.3	0.0	85.2	908.9	O K
10080 min Summer	1.129	0.029	57.0	21.1	0.0	78.1	811.0	O K
15 min Winter	1.140	0.040	79.8	21.6	0.0	101.4	1139.7	O K
30 min Winter	1.147	0.047	93.7	21.9	0.0	115.6	1342.5	O K
60 min Winter	1.154	0.054	99.2	22.2	0.0	121.4	1539.4	O K
120 min Winter	1.161	0.061	99.2	22.5	0.0	121.7	1735.5	O K
180 min Winter	1.166	0.066	99.2	22.8	0.0	121.9	1878.0	O K
240 min Winter	1.174	0.074	99.2	23.1	0.0	122.3	2108.8	O K
360 min Winter	1.191	0.091	99.2	23.8	0.0	123.0	2579.3	O K
480 min Winter	1.198	0.098	99.2	24.2	0.0	123.3	2796.3	O K
600 min Winter	1.201	0.101	99.2	24.3	0.0	123.5	2888.3	O K
720 min Winter	1.200	0.100	99.2	24.3	0.0	123.4	2859.5	O K
960 min Winter	1.192	0.092	99.2	23.9	0.0	123.0	2617.9	O K
1440 min Winter	1.178	0.078	99.2	23.3	0.0	122.5	2223.1	O K
2160 min Winter	1.158	0.058	99.2	22.4	0.0	121.6	1656.7	O K
2880 min Winter	1.147	0.047	92.7	21.9	0.0	114.6	1331.3	O K
4320 min Winter	1.136	0.036	71.9	21.4	0.0	93.3	1024.0	O K
5760 min Winter	1.129	0.029	58.0	21.1	0.0	79.1	823.4	O K
7200 min Winter	1.124	0.024	48.1	20.9	0.0	69.0	681.3	O K
8640 min Winter	1.120	0.020	40.2	20.7	0.0	60.9	576.2	O K
10080 min Winter	1.118	0.018	35.2	20.6	0.0	55.8	496.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	13494.7	0.0	4584
10080 min Summer	1.416	0.0	14074.6	0.0	5328
15 min Winter	177.348	0.0	2065.9	0.0	18
30 min Winter	107.772	0.0	2552.5	0.0	32
60 min Winter	65.491	0.0	3251.8	0.0	60
120 min Winter	39.798	0.0	4000.2	0.0	118
180 min Winter	29.739	0.0	4523.8	0.0	178
240 min Winter	24.185	0.0	4959.6	0.0	340
360 min Winter	18.072	0.0	5742.5	0.0	420
480 min Winter	14.697	0.0	6253.3	0.0	488
600 min Winter	12.519	0.0	6658.1	0.0	588
720 min Winter	10.982	0.0	6989.3	0.0	678
960 min Winter	8.733	0.0	7401.0	0.0	790
1440 min Winter	6.322	0.0	8101.4	0.0	1036
2160 min Winter	4.577	0.0	9070.2	0.0	1384
2880 min Winter	3.640	0.0	9666.4	0.0	1720
4320 min Winter	2.681	0.0	10804.3	0.0	2448
5760 min Winter	2.159	0.0	12091.1	0.0	3168
7200 min Winter	1.825	0.0	12842.2	0.0	3888
8640 min Winter	1.590	0.0	13494.7	0.0	4656
10080 min Winter	1.416	0.0	14074.4	0.0	5344

Cascade Rainfall Details for Bioretention Basin - East-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 3.000

Time From:	**(mins) To:**	**Area (ha)**
0	4	3.000

Cascade Model Details for Bioretention Basin - East-A.srcx

Storage is Online Cover Level (m) 2.000

Bio-Retention Area Structure

Invert Level (m) 1.100 Infiltration Coefficient Base (m/hr) 0.02520
Porosity 1.00 Infiltration Coefficient Side (m/hr) 0.00000
Safety Factor 2.0

Depth (m)	**Area (m²)**	**Perimeter (m)**	**Depth (m)**	**Area (m²)**	**Perimeter (m)**
0.000	28334.0	839.000	1.300	146900.0	2010.000
0.100	28671.0	841.500	1.400	146900.0	2010.000
0.200	29008.0	844.000	1.500	146900.0	2010.000
0.300	29346.0	846.500	1.600	146900.0	2010.000
0.400	29685.0	849.000	1.700	146900.0	2010.000
0.500	30025.0	851.500	1.800	146900.0	2010.000
0.600	30366.0	854.000	1.900	146900.0	2010.000
0.700	30708.0	856.500	2.000	146900.0	2010.000
0.800	146900.0	2010.000	2.100	146900.0	2010.000
0.900	146900.0	2010.000	2.200	146900.0	2010.000
1.000	146900.0	2010.000	2.300	146900.0	2010.000
1.100	146900.0	2010.000	2.400	146900.0	2010.000
1.200	146900.0	2010.000	2.500	146900.0	2010.000

Filtration Outflow Control

Permeability Coefficient (m/s) 0.000007 Area (m²) 28334.000
Safety Factor 10.000 Invert Level (m) 1.100
Bed Depth (m) 0.450

Orifice Overflow Control

Diameter (m) 0.270 Discharge Coefficient 0.600 Invert Level (m) 1.200

Cascade Model Details for Detention basin - East - blue roof outflows-A.srcx

Storage is Online Cover Level (m) 5.000

Cellular Storage Structure

Invert Level (m) 4.900 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	12000.0	12000.0	0.075	12000.0	12000.0
0.025	12000.0	12000.0	0.100	12000.0	12000.0
0.050	12000.0	12000.0			

Depth/Flow Relationship Outflow Control

Invert Level (m) 4.900

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.001	70.0000	0.050	70.0000	0.075	70.0000	0.100	70.0000

Non-Infiltration scenario

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Upstream Structures	Outflow To	Overflow To
(None)	Perm Pavement-A.srcx	(None)

Half Drain Time : 67 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	4.936	0.036	0.0	70.0	70.0	411.4	Flood Risk
30 min Summer	4.940	0.040	0.0	70.0	70.0	457.0	Flood Risk
60 min Summer	4.941	0.041	0.0	70.0	70.0	468.7	Flood Risk
120 min Summer	4.939	0.039	0.0	70.0	70.0	448.8	Flood Risk
180 min Summer	4.937	0.037	0.0	70.0	70.0	427.3	Flood Risk
240 min Summer	4.935	0.035	0.0	70.0	70.0	403.2	Flood Risk
360 min Summer	4.930	0.030	0.0	70.0	70.0	346.9	Flood Risk
480 min Summer	4.926	0.026	0.0	70.0	70.0	292.5	Flood Risk
600 min Summer	4.921	0.021	0.0	70.0	70.0	240.7	Flood Risk
720 min Summer	4.917	0.017	0.0	70.0	70.0	195.7	Flood Risk
960 min Summer	4.910	0.010	0.0	70.0	70.0	108.4	Flood Risk
1440 min Summer	4.902	0.002	0.0	70.0	70.0	18.8	Flood Risk
2160 min Summer	4.901	0.001	0.0	52.5	52.5	9.6	Flood Risk
2880 min Summer	4.901	0.001	0.0	52.5	52.5	9.6	Flood Risk
4320 min Summer	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
5760 min Summer	4.901	0.001	0.0	52.5	52.5	9.7	Flood Risk
7200 min Summer	4.901	0.001	0.0	52.5	52.5	9.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	473.6	17
30 min Summer	107.772	0.0	575.4	31
60 min Summer	65.491	0.0	703.5	58
120 min Summer	39.798	0.0	856.8	88
180 min Summer	29.739	0.0	961.4	124
240 min Summer	24.185	0.0	1038.8	158
360 min Summer	18.072	0.0	1164.4	226
480 min Summer	14.697	0.0	1264.0	290
600 min Summer	12.519	0.0	1346.2	354
720 min Summer	10.982	0.0	1418.1	414
960 min Summer	8.733	0.0	1504.6	530
1440 min Summer	6.322	0.0	1638.7	738
2160 min Summer	4.577	0.0	1779.5	1132
2880 min Summer	3.640	0.0	1887.2	1448
4320 min Summer	2.681	0.0	2085.5	2084
5760 min Summer	2.159	0.0	2261.9	2824
7200 min Summer	1.825	0.0	2387.1	3520

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	4.901	0.001	0.0	52.5	52.5	9.4	Flood Risk
10080 min Summer	4.900	0.000	0.0	16.7	16.7	0.0	Flood Risk
15 min Winter	4.936	0.036	0.0	70.0	70.0	412.3	Flood Risk
30 min Winter	4.940	0.040	0.0	70.0	70.0	460.6	Flood Risk
60 min Winter	4.942	0.042	0.0	70.0	70.0	475.6	Flood Risk
120 min Winter	4.939	0.039	0.0	70.0	70.0	443.8	Flood Risk
180 min Winter	4.936	0.036	0.0	70.0	70.0	406.4	Flood Risk
240 min Winter	4.932	0.032	0.0	70.0	70.0	361.3	Flood Risk
360 min Winter	4.924	0.024	0.0	70.0	70.0	272.0	Flood Risk
480 min Winter	4.916	0.016	0.0	70.0	70.0	186.9	Flood Risk
600 min Winter	4.910	0.010	0.0	70.0	70.0	117.6	Flood Risk
720 min Winter	4.905	0.005	0.0	70.0	70.0	59.8	Flood Risk
960 min Winter	4.901	0.001	0.0	70.0	70.0	12.9	Flood Risk
1440 min Winter	4.901	0.001	0.0	52.5	52.5	8.2	Flood Risk
2160 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
2880 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
4320 min Winter	4.901	0.001	0.0	52.5	52.5	7.8	Flood Risk
5760 min Winter	4.900	0.000	0.0	16.4	16.4	0.0	Flood Risk
7200 min Winter	4.900	0.000	0.0	13.8	13.8	0.0	Flood Risk
8640 min Winter	4.900	0.000	0.0	12.1	12.1	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	2480.0	4368
10080 min Summer	1.416	0.0	2569.2	0
15 min Winter	177.348	0.0	473.6	17
30 min Winter	107.772	0.0	578.2	31
60 min Winter	65.491	0.0	700.9	58
120 min Winter	39.798	0.0	857.2	94
180 min Winter	29.739	0.0	960.8	132
240 min Winter	24.185	0.0	1037.0	168
360 min Winter	18.072	0.0	1167.1	238
480 min Winter	14.697	0.0	1264.5	302
600 min Winter	12.519	0.0	1348.0	360
720 min Winter	10.982	0.0	1418.7	412
960 min Winter	8.733	0.0	1508.8	486
1440 min Winter	6.322	0.0	1638.9	722
2160 min Winter	4.577	0.0	1779.9	1304
2880 min Winter	3.640	0.0	1886.8	1336
4320 min Winter	2.681	0.0	2085.2	2140
5760 min Winter	2.159	0.0	2238.3	0
7200 min Winter	1.825	0.0	2364.8	0
8640 min Winter	1.590	0.0	2473.4	0

Cascade Summary of Results for Detention basin - East - blue roof outflows-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m^3)	Status
10080 min Winter	4.900	0.000	0.0	10.7	10.7	0.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Discharge Volume (m^3)	Time-Peak (mins)
10080 min Winter	1.416	0.0	2569.2	0

Cascade Rainfall Details for Detention basin - East - blue roof outflows-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.200

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.200

Cascade Model Details for Detention basin - East - blue roof outflows-A.srcx

Storage is Online Cover Level (m) 5.000

Cellular Storage Structure

Invert Level (m) 4.900 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	12000.0	12000.0	0.075	12000.0	12000.0
0.025	12000.0	12000.0	0.100	12000.0	12000.0
0.050	12000.0	12000.0			

Depth/Flow Relationship Outflow Control

Invert Level (m) 4.900

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.001	70.0000	0.050	70.0000	0.075	70.0000	0.100	70.0000

Cascade Summary of Results for Perm Pavement-A.srcx

Upstream Structures	Outflow To	Overflow To
Detention basin - East - blue roof outflows-A.srcx	(None)	Bioretention Basin - East-A.srcx

Half Drain Time exceeds 7 days.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	1.455	0.255	0.0	0.0	0.0	1031.4	O K
30 min Summer	1.505	0.305	0.0	0.0	0.0	1270.3	Flood Risk
60 min Summer	1.561	0.361	0.0	38.0	38.0	1540.8	Flood Risk
120 min Summer	1.566	0.366	0.0	70.6	70.6	1565.0	Flood Risk
180 min Summer	1.572	0.372	0.0	109.3	109.3	1592.7	Flood Risk
240 min Summer	1.573	0.373	0.0	116.9	116.9	1598.0	Flood Risk
360 min Summer	1.578	0.378	0.0	157.5	157.5	1620.2	Flood Risk
480 min Summer	1.580	0.380	0.0	179.3	179.3	1634.1	Flood Risk
600 min Summer	1.581	0.381	0.0	188.3	188.3	1638.6	Flood Risk
720 min Summer	1.582	0.382	0.0	192.8	192.8	1639.7	Flood Risk
960 min Summer	1.581	0.381	0.0	183.8	183.8	1635.4	Flood Risk
1440 min Summer	1.579	0.379	0.0	166.1	166.1	1625.4	Flood Risk
2160 min Summer	1.574	0.374	0.0	124.7	124.7	1601.2	Flood Risk
2880 min Summer	1.571	0.371	0.0	101.9	101.9	1588.0	Flood Risk
4320 min Summer	1.567	0.367	0.0	73.9	73.9	1569.3	Flood Risk
5760 min Summer	1.566	0.366	0.0	67.4	67.4	1562.1	Flood Risk
7200 min Summer	1.564	0.364	0.0	58.0	58.0	1557.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	0.0	116
30 min Summer	107.772	0.0	0.0	141
60 min Summer	65.491	0.0	76.6	172
120 min Summer	39.798	0.0	431.0	180
180 min Summer	29.739	0.0	672.1	176
240 min Summer	24.185	0.0	855.8	188
360 min Summer	18.072	0.0	1145.8	228
480 min Summer	14.697	0.0	1373.1	284
600 min Summer	12.519	0.0	1561.0	338
720 min Summer	10.982	0.0	1723.8	392
960 min Summer	8.733	0.0	1916.6	508
1440 min Summer	6.322	0.0	2207.7	736
2160 min Summer	4.577	0.0	2512.5	1092
2880 min Summer	3.640	0.0	2739.2	1456
4320 min Summer	2.681	0.0	3153.8	2176
5760 min Summer	2.159	0.0	3486.6	2944
7200 min Summer	1.825	0.0	3732.5	3592

Cascade Summary of Results for Perm Pavement-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	1.564	0.364	0.0	54.9	54.9	1554.0	Flood Risk
10080 min Summer	1.561	0.361	0.0	40.7	40.7	1541.0	Flood Risk
15 min Winter	1.455	0.255	0.0	0.0	0.0	1031.4	O K
30 min Winter	1.505	0.305	0.0	0.0	0.0	1273.1	Flood Risk
60 min Winter	1.561	0.361	0.0	38.0	38.0	1539.8	Flood Risk
120 min Winter	1.566	0.366	0.0	70.6	70.6	1565.0	Flood Risk
180 min Winter	1.573	0.373	0.0	116.9	116.9	1596.5	Flood Risk
240 min Winter	1.575	0.375	0.0	132.7	132.7	1606.7	Flood Risk
360 min Winter	1.579	0.379	0.0	170.5	170.5	1627.6	Flood Risk
480 min Winter	1.580	0.380	0.0	179.3	179.3	1632.6	Flood Risk
600 min Winter	1.580	0.380	0.0	174.9	174.9	1630.7	Flood Risk
720 min Winter	1.579	0.379	0.0	170.5	170.5	1627.7	Flood Risk
960 min Winter	1.577	0.377	0.0	149.1	149.1	1616.1	Flood Risk
1440 min Winter	1.572	0.372	0.0	113.1	113.1	1594.3	Flood Risk
2160 min Winter	1.568	0.368	0.0	80.6	80.6	1572.8	Flood Risk
2880 min Winter	1.566	0.366	0.0	67.4	67.4	1563.3	Flood Risk
4320 min Winter	1.563	0.363	0.0	52.0	52.0	1551.8	Flood Risk
5760 min Winter	1.561	0.361	0.0	38.0	38.0	1539.1	Flood Risk
7200 min Winter	1.560	0.360	0.0	32.8	32.8	1533.9	Flood Risk
8640 min Winter	1.559	0.359	0.0	27.9	27.9	1529.4	Flood Risk
10080 min Winter	1.558	0.358	0.0	25.5	25.5	1526.7	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	3922.3	4384
10080 min Summer	1.416	0.0	4091.1	5056
15 min Winter	177.348	0.0	0.0	116
30 min Winter	107.772	0.0	0.0	142
60 min Winter	65.491	0.0	74.0	172
120 min Winter	39.798	0.0	431.4	202
180 min Winter	29.739	0.0	671.5	170
240 min Winter	24.185	0.0	854.0	180
360 min Winter	18.072	0.0	1148.6	234
480 min Winter	14.697	0.0	1373.5	288
600 min Winter	12.519	0.0	1562.8	344
720 min Winter	10.982	0.0	1724.4	398
960 min Winter	8.733	0.0	1920.9	506
1440 min Winter	6.322	0.0	2208.0	734
2160 min Winter	4.577	0.0	2512.9	1100
2880 min Winter	3.640	0.0	2739.0	1472
4320 min Winter	2.681	0.0	3154.5	2248
5760 min Winter	2.159	0.0	3464.8	2920
7200 min Winter	1.825	0.0	3712.9	3480
8640 min Winter	1.590	0.0	3919.2	4384
10080 min Winter	1.416	0.0	4095.5	5000

Cascade Rainfall Details for Perm Pavement-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.600

Time From:	**(mins) To:**	**Area (ha)**
0	4	1.600

Cascade Model Details for Perm Pavement-A.srcx

Storage is Online Cover Level (m) 1.800

Porous Car Park Structure

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	20.0
Membrane Percolation (mm/hr)	1000	Length (m)	800.0
Max Percolation (l/s)	4444.4	Slope (1:X)	10000.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	1.200	Membrane Depth (m)	0

Weir Overflow Control

Discharge Coef 0.544 Width (m) 20.000 Invert Level (m) 1.550

Cascade Summary of Results for Bioretention Basin - East-A.srcx

Upstream Structures	Outflow To	Overflow To
Perm Pavement-A.srcx Detention basin - East - blue roof outflows-A.srcx	(None)	(None)

Half Drain Time : 2193 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	1.153	0.053	0.0	22.2	0.0	22.2	1501.9	O K
30 min Summer	1.169	0.069	0.0	22.9	0.0	22.9	1957.5	O K
60 min Summer	1.191	0.091	0.0	23.8	0.0	23.8	2587.7	O K
120 min Summer	1.226	0.126	0.0	25.4	0.7	26.1	3610.1	O K
180 min Summer	1.250	0.150	0.0	26.5	2.0	28.5	4290.9	O K
240 min Summer	1.268	0.168	0.0	27.2	3.7	30.9	4804.1	O K
360 min Summer	1.294	0.194	0.0	28.4	6.9	35.3	5572.4	O K
480 min Summer	1.314	0.214	0.0	29.3	9.8	39.0	6138.8	O K
600 min Summer	1.329	0.229	0.0	29.9	12.5	42.5	6579.9	O K
720 min Summer	1.341	0.241	0.0	30.5	15.5	46.0	6934.2	O K
960 min Summer	1.351	0.251	0.0	30.9	18.0	48.9	7220.0	O K
1440 min Summer	1.355	0.255	0.0	31.1	19.0	50.1	7346.3	O K
2160 min Summer	1.354	0.254	0.0	31.0	18.8	49.8	7315.6	O K
2880 min Summer	1.352	0.252	0.0	30.9	18.2	49.1	7238.3	O K
4320 min Summer	1.351	0.251	0.0	30.9	18.0	48.9	7228.0	O K
5760 min Summer	1.350	0.250	0.0	30.8	17.7	48.5	7180.2	O K
7200 min Summer	1.346	0.246	0.0	30.7	16.8	47.5	7083.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
15 min Summer	177.348	0.0	1845.1	0.0	340
30 min Summer	107.772	0.0	1900.8	0.0	363
60 min Summer	65.491	0.0	3329.6	0.0	394
120 min Summer	39.798	0.0	3993.9	12.9	434
180 min Summer	29.739	0.0	4193.7	63.0	468
240 min Summer	24.185	0.0	4373.6	136.1	502
360 min Summer	18.072	0.0	4699.5	313.9	570
480 min Summer	14.697	0.0	4986.9	501.8	642
600 min Summer	12.519	0.0	5238.7	681.8	712
720 min Summer	10.982	0.0	5470.0	861.5	780
960 min Summer	8.733	0.0	5691.5	1052.8	966
1440 min Summer	6.322	0.0	5918.1	1293.9	1416
2160 min Summer	4.577	0.0	10355.1	1591.5	1764
2880 min Summer	3.640	0.0	10462.2	1709.4	2160
4320 min Summer	2.681	0.0	10463.9	1934.6	2980
5760 min Summer	2.159	0.0	15581.5	2060.3	3808
7200 min Summer	1.825	0.0	16575.1	2085.0	4616

Cascade Summary of Results for Bioretention Basin - East-A.srcx

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Overflow (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
8640 min Summer	1.342	0.242	0.0	30.5	15.8	46.3	6958.3	O K
10080 min Summer	1.338	0.238	0.0	30.3	14.6	45.0	6828.1	O K
15 min Winter	1.153	0.053	0.0	22.2	0.0	22.2	1501.9	O K
30 min Winter	1.169	0.069	0.0	22.9	0.0	22.9	1956.5	O K
60 min Winter	1.191	0.091	0.0	23.8	0.0	23.8	2584.5	O K
120 min Winter	1.226	0.126	0.0	25.4	0.7	26.1	3610.9	O K
180 min Winter	1.250	0.150	0.0	26.5	2.0	28.5	4290.8	O K
240 min Winter	1.268	0.168	0.0	27.2	3.7	30.9	4802.3	O K
360 min Winter	1.294	0.194	0.0	28.4	6.9	35.3	5573.7	O K
480 min Winter	1.314	0.214	0.0	29.3	9.8	39.0	6137.2	O K
600 min Winter	1.329	0.229	0.0	29.9	12.5	42.5	6580.6	O K
720 min Winter	1.341	0.241	0.0	30.5	15.5	46.0	6934.1	O K
960 min Winter	1.352	0.252	0.0	31.0	18.3	49.2	7247.8	O K
1440 min Winter	1.359	0.259	0.0	31.2	19.9	51.1	7445.8	O K
2160 min Winter	1.356	0.256	0.0	31.1	19.3	50.4	7376.8	O K
2880 min Winter	1.353	0.253	0.0	31.0	18.5	49.5	7280.8	O K
4320 min Winter	1.349	0.249	0.0	30.8	17.5	48.3	7170.9	O K
5760 min Winter	1.344	0.244	0.0	30.6	16.1	46.7	7000.8	O K
7200 min Winter	1.337	0.237	0.0	30.3	14.4	44.7	6800.5	O K
8640 min Winter	1.329	0.229	0.0	29.9	12.5	42.5	6586.5	O K
10080 min Winter	1.322	0.222	0.0	29.6	10.9	40.6	6362.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Overflow Volume (m³)	Time-Peak (mins)
8640 min Summer	1.590	0.0	17243.2	2047.6	5448
10080 min Summer	1.416	0.0	16761.7	1984.0	6272
15 min Winter	177.348	0.0	1845.1	0.0	340
30 min Winter	107.772	0.0	1900.7	0.0	364
60 min Winter	65.491	0.0	3326.9	0.0	394
120 min Winter	39.798	0.0	3993.3	13.0	434
180 min Winter	29.739	0.0	4192.5	63.1	468
240 min Winter	24.185	0.0	4371.6	136.4	502
360 min Winter	18.072	0.0	4698.7	315.9	572
480 min Winter	14.697	0.0	4984.3	504.1	644
600 min Winter	12.519	0.0	5235.4	685.1	720
720 min Winter	10.982	0.0	5464.7	865.2	792
960 min Winter	8.733	0.0	5683.1	1059.3	962
1440 min Winter	6.322	0.0	5897.6	1298.4	1386
2160 min Winter	4.577	0.0	10334.5	1591.7	1844
2880 min Winter	3.640	0.0	10434.0	1704.1	2264
4320 min Winter	2.681	0.0	10425.1	1883.2	3172
5760 min Winter	2.159	0.0	15551.3	1949.0	4088
7200 min Winter	1.825	0.0	16555.1	1917.0	4976
8640 min Winter	1.590	0.0	17180.5	1826.3	5848
10080 min Winter	1.416	0.0	16689.8	1710.4	6728

Cascade Rainfall Details for Bioretention Basin - East-A.srcx

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 520800 104800 TQ 20800 04800
C (1km)	-0.026
D1 (1km)	0.401
D2 (1km)	0.323
D3 (1km)	0.366
E (1km)	0.309
F (1km)	2.424
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.900
Cv (Winter)	0.900
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 3.000

Time From:	**(mins) To:**	**Area (ha)**
0	4	3.000

Cascade Model Details for Bioretention Basin - East-A.srcx

Storage is Online Cover Level (m) 2.000

Bio-Retention Area Structure

Invert Level (m) 1.100 Infiltration Coefficient Base (m/hr) 0.00000
Porosity 1.00 Infiltration Coefficient Side (m/hr) 0.00000
Safety Factor 2.0

Depth (m)	**Area (m²)**	**Perimeter (m)**	**Depth (m)**	**Area (m²)**	**Perimeter (m)**
0.000	28334.0	839.000	1.300	146900.0	2010.000
0.100	28671.0	841.500	1.400	146900.0	2010.000
0.200	29008.0	844.000	1.500	146900.0	2010.000
0.300	29346.0	846.500	1.600	146900.0	2010.000
0.400	29685.0	849.000	1.700	146900.0	2010.000
0.500	30025.0	851.500	1.800	146900.0	2010.000
0.600	30366.0	854.000	1.900	146900.0	2010.000
0.700	30708.0	856.500	2.000	146900.0	2010.000
0.800	146900.0	2010.000	2.100	146900.0	2010.000
0.900	146900.0	2010.000	2.200	146900.0	2010.000
1.000	146900.0	2010.000	2.300	146900.0	2010.000
1.100	146900.0	2010.000	2.400	146900.0	2010.000
1.200	146900.0	2010.000	2.500	146900.0	2010.000

Filtration Outflow Control

Permeability Coefficient (m/s) 0.000007 Area (m²) 28334.000
Safety Factor 10.000 Invert Level (m) 1.100
Bed Depth (m) 0.450

Orifice Overflow Control

Diameter (m) 0.270 Discharge Coefficient 0.600 Invert Level (m) 1.200

Appendix K – SuDS Manual Extracts

TABLE 26.2 Pollution hazard indices for different land use classifications

Land use	Pollution hazard level	Total suspended solids (TSS)	Metals	Hydro-carbons
Residential roofs	Very low	0.2	0.2	0.05
Other roofs (typically commercial/ industrial roofs)	Low	0.3	0.2 (up to 0.8 where there is potential for metals to leach from the roof)	0.05
Individual property driveways, residential car parks, low traffic roads (eg cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change (eg schools, offices) ie < 300 traffic movements/day	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (eg hospitals, retail), all roads except low traffic roads and trunk roads/motorways[1]	Medium	0.7	0.6	0.7
Sites with heavy pollution (eg haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways[1]	High	0.8[2]	0.8[2]	0.9[2]

Notes

1 Motorways and trunk roads should follow the guidance and risk assessment process set out in Highways Agency (2009).

2 These should only be used if considered appropriate as part of a detailed risk assessment – required for all these land use types (Table 4.3). When dealing with high hazard sites, the environmental regulator should first be consulted for pre-permitting advice. This will help determine the most appropriate approach to the development of a design solution.

Where a site land use falls outside the defined categories, the indices should be adapted (and agreed with the drainage approving body) or else the more detailed risk assessment method should be adopted.

Where nutrient or bacteria and pathogen removal is important for a particular receiving water, equivalent indices should be developed for these pollutants (if acceptable to the drainage approving body) or the risk assessment method adopted.

Where the mitigation index of an individual component is insufficient, two components (or more) in series will be required, where:

$$\textbf{Total SuDS mitigation index} = \textbf{mitigation index}_1 + 0.5\,(\textbf{mitigation index}_2)$$

Where:

mitigation Index_n = mitigation index for component n

A factor of 0.5 is used to account for the reduced performance of secondary or tertiary components associated with already reduced inflow concentrations.

TABLE 26.4 Indicative SuDS mitigation indices for discharges to groundwater

Characteristics of the material overlying the proposed infiltration surface, through which the runoff percolates[1]	TSS	Metals	Hydrocarbons
A layer of dense vegetation underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.6[4]	0.5	0.6
A soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.4[4]	0.3	0.3
Infiltration trench (where a suitable depth of filtration material is included that provides treatment, ie graded gravel with sufficient smaller particles but not single size coarse aggregate such as 20 mm gravel) underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.4[4]	0.4	0.4
Constructed permeable pavement (where a suitable filtration layer is included that provides treatment, and including a geotextile at the base separating the foundation from the subgrade) underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.7	0.6	0.7
Bioretention underlain by a soil with good contaminant attenuation potential[2] of at least 300 mm in depth[3]	0.8[4]	0.8	0.8
Proprietary treatment systems[5, 6]	These must demonstrate that they can address each of the contaminant types to acceptable levels for inflow concentrations relevant to the contributing drainage area.		

Notes

1. All designs must include a minimum of 1 m unsaturated depth of aquifer material between the infiltration surface and the maximum likely groundwater level (as required in infiltration design – Chapter 25).
2. For example as recommended in Sniffer (2008a and 2008b), Scott Wilson (2010) or other appropriate guidance.
3. Alternative depths may be considered where it can be demonstrated that the combination of the proposed depth and soil characteristics will provide equivalent protection to the underlying groundwater – see note 1.
4. If significant volumes of sediment are allowed to enter an infiltration system, there will be a high risk of rapid clogging and subsequent system failure.
5. See Chapter 14 for approaches to demonstrate product performance. Note: a British Water/Environment Agency assessment code of practice is currently under development that will allow manufacturers to complete an agreed test protocol for systems intended to treat contaminated surface water runoff. Full details can be found at: **www.britishwater.co.uk/Publications/codes-of-practise.aspx**
6. SEPA only considers proprietary treatment systems as appropriate in exceptional circumstances where other types of SuDS component are not practicable. Proprietary treatment systems may also be considered appropriate for existing sites that are causing pollution, where there is a requirement to retrofit treatment. WAT-RM-08 (SEPA, 2014) also provides a flowchart with a summary of checks on suitability of a proprietary system.

The following should be noted:

- Where the indices are not considered representative by the designer, a risk assessment can be undertaken (Section 26.7.3).
- Components should always be designed for treatment, as described in the design guidance set out in the individual component chapters. If they are undersized, incorrectly designed or constructed or inadequately maintained, their treatment performance could be significantly affected. Component checklists (Appendix B) can be used to confirm design and construction adequacy and set appropriate maintenance regimes.
- Where the infiltration component itself does not provide sufficient pollution mitigation, the design should include upstream SuDS components that are lined to prevent infiltration from occurring. The mitigation indices set out in Table 26.3 (for discharges to surface water) should be used for any upstream treatment.

Appendix L – Adur Tidal Wall Scheme

Notes

Key to Symbols

Legend

- Soldiers Point - Reach W1
- Emerald Quay and South Wharf - Reach W2
- Riverside Road - Reach W3
- Ferry Bridge - Reach W4
- Riverbank Reach - Reach W5
- Recreation Ground - Reach W6
- Shoreham Airport - Reach W7
- Old Railway Reach - Reach E3
- Ropetackle - Reach E2
- Tollhouse Bridge - Reach E1

Reference drawings

Rev	Date	Drawn	Description	Ch'k'd	App'd

Mott MacDonald
Mott MacDonald House
8-10 Sydenham Road
Croydon CR0 2EE
United Kingdom

T +44 (0)20 8774 2000
F +44 (0)20 8681 5706
W www.mottmac.com

Client

Environment Agency
Guildbourne House
Chatsworth Road
Worthing
BN11 1LD

T +44 (0)3708 506 506
E shorehamwestbank@ea.gov.uk
W www.gov.uk/environment-agency

Title

Shoreham Adur Tidal Walls

Reach Design Names

Designed	NC		Eng Check	DC	
Drawn	NC		Coordination	NC	
GIS Check	BR		Approved	PP	
Scale at A1 1:5,000		Status INF		Rev 00	

Drawing No.
MM - 339663 - DES - GIS - 000 - 0001 - 00

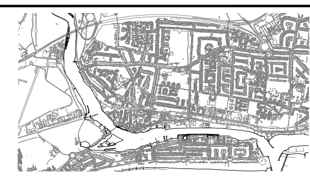
Notes

1. All dimensions are in metres unless noted otherwise.
2. All levels are in metres relative to Ordnance Datum Newlyn (OD).
3. All positions are in metres relative to National Grid.
4. Limit of deviation is extended 1.0m from the toe of embankment or sheet pile walls on each side.
5. The Boundary of Works lines should be 10m from the Flood Defence Wall riverward unless the Mean Low Water is encountered before, then Boundary of Works lines turns to be Mean Low Water.

Key to symbols

- Flood Defence (Wall or Embankment)
- MLW — Mean Low Water (MLW)
- MHW — Mean High Water (MHW)
- E1 — Reach extent
- Boundary of works
- Limits of deviation
- Satellite site compounds
- Site access from public road
- Temporary Works
- Permanent Works

Key plan



Reference drawings

1:5000 — 0 — 250m — 500m

Rev	Date	Drawn	Description	Ch'k'd	App'd
P01	20.10.15	IAS	For Planning Approval / Tender	NJC	PJP

Mott MacDonald

Mott MacDonald House
8-10 Sydenham Road
Croydon, CR0 2EE
United Kingdom

T +44 (0)20 8774 2000
F +44 (0)20 8681 5706
W www.mottmac.com

Client

Environment Agency

Title

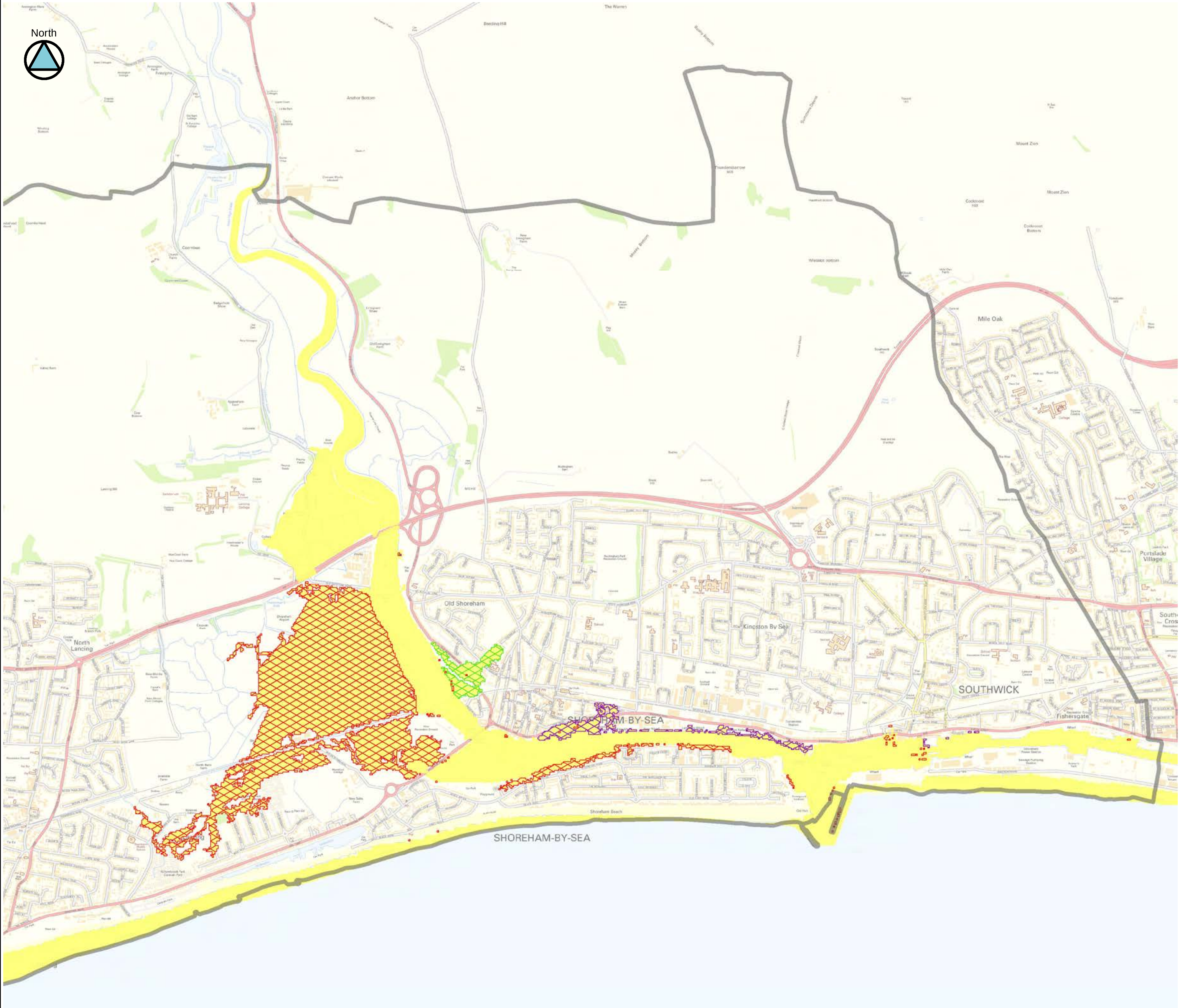
Shoreham Adur Tidal Walls

Planning Drawing - Overview

Designed	F Loy	FL	Eng check	D Collery	DC
Drawn	I A Sussex	IAS	Coordination	N J Clarke	NJC
Dwg check	P M Starnes	PMS	Approved	P J Phipps	PJP

Scale at A1	Status	Rev	Security
1:5000	D2	P01	STD

Drawing Number
MMD-339663-C-DR-01-XX-1030



Scale 1:25,000

Worthing Borough Council

ADUR DISTRICT COUNCIL

LEGEND

- Adur and Worthing Boundary
- 1 in 20 year Present Day

Defence improvements

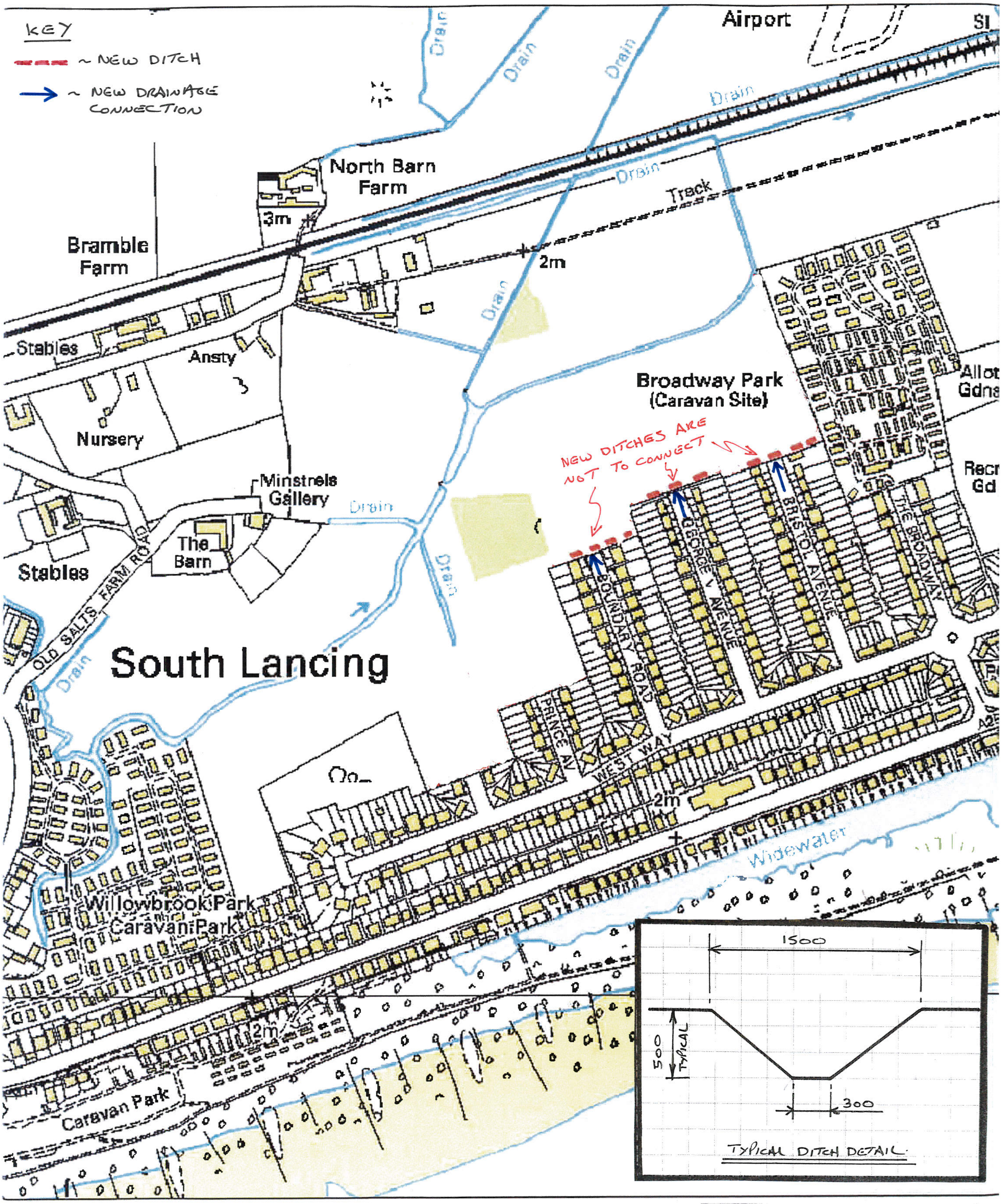
- Adur Tidal Walls (ATW)
- Ropetackle defences
- Shoreham Harbour walls

Note: The cross-hatched polygons show those areas that will no longer be inundated in the 1 in 20 year event if the defence improvements along the River Adur are undertaken.
Three scenarios are shown:
1. Improvement to the Adur Tidal walls
2. Improvement to the Adur Tidal walls and Ropetackle defences
3. Improvement to the Adur Tidal walls and Shoreham Harbour walls
These extents are based on the modelling undertaken as part of the West Bank Tidal Walls (Arun to Adur Model update) 2011.

JBA consulting

Map 17
Impact of proposed defence improvements on the 1 in 20 year (Flood Zone 3b) extent.

Appendix M – West Beach Estate Drainage



NOTE

DEPTH AND LOCATION OF NEW SURFACE WATER CONNECTIONS ARE NOT KNOWN. TO BE ADVISED.

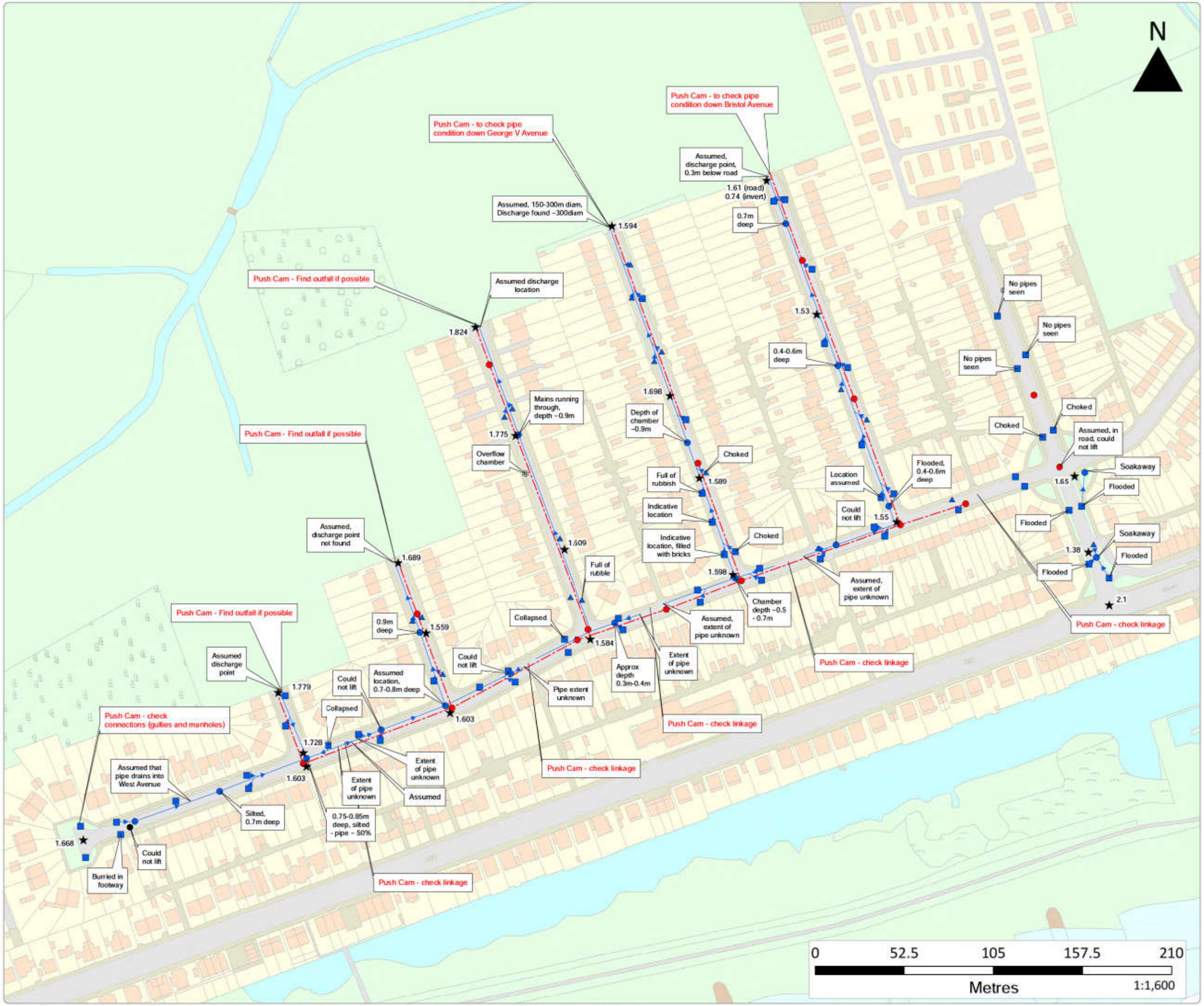
TITLE:	WEST BEACH DITCH
PROJECT:	NEW SALTS FARM
SCALE: / DATE: / DRAWN: / CHK'D:	
JOB NO.	11649
DRG NO.	SK100
REV.	A

Tully De'Ath consultants

Engineering at its Best

SHERIDAN HOUSE, HARTFIELD ROAD, FOREST ROW, SUSSEX RH18 5EA
TELEPHONE 01342 828 000 FACSIMILE 01342 828 001
EMAIL info@tullydeath.com WEB SITE www.tullydeath.com

Lancing SWMP



Legend

★ Temporary Bench Mark Level (m)

Assets (Points)

- Catchpit
- Foul Chamber
- Gully
- Kerb Inlet
- Surface Chamber
- Unknown Chamber

Assets (Lines)

- Assumed Foul Pipe
- Assumed Surface Water Pipe

Notes:
- Foul chambers not lifted
- Orient Road & Mobile home park not surveyed
- Adur & Worthing CC knowledge that Orient Road & lower end of Broadway are drained via soakaways

Drainage Plan (Zone D)

CH2MHILL.

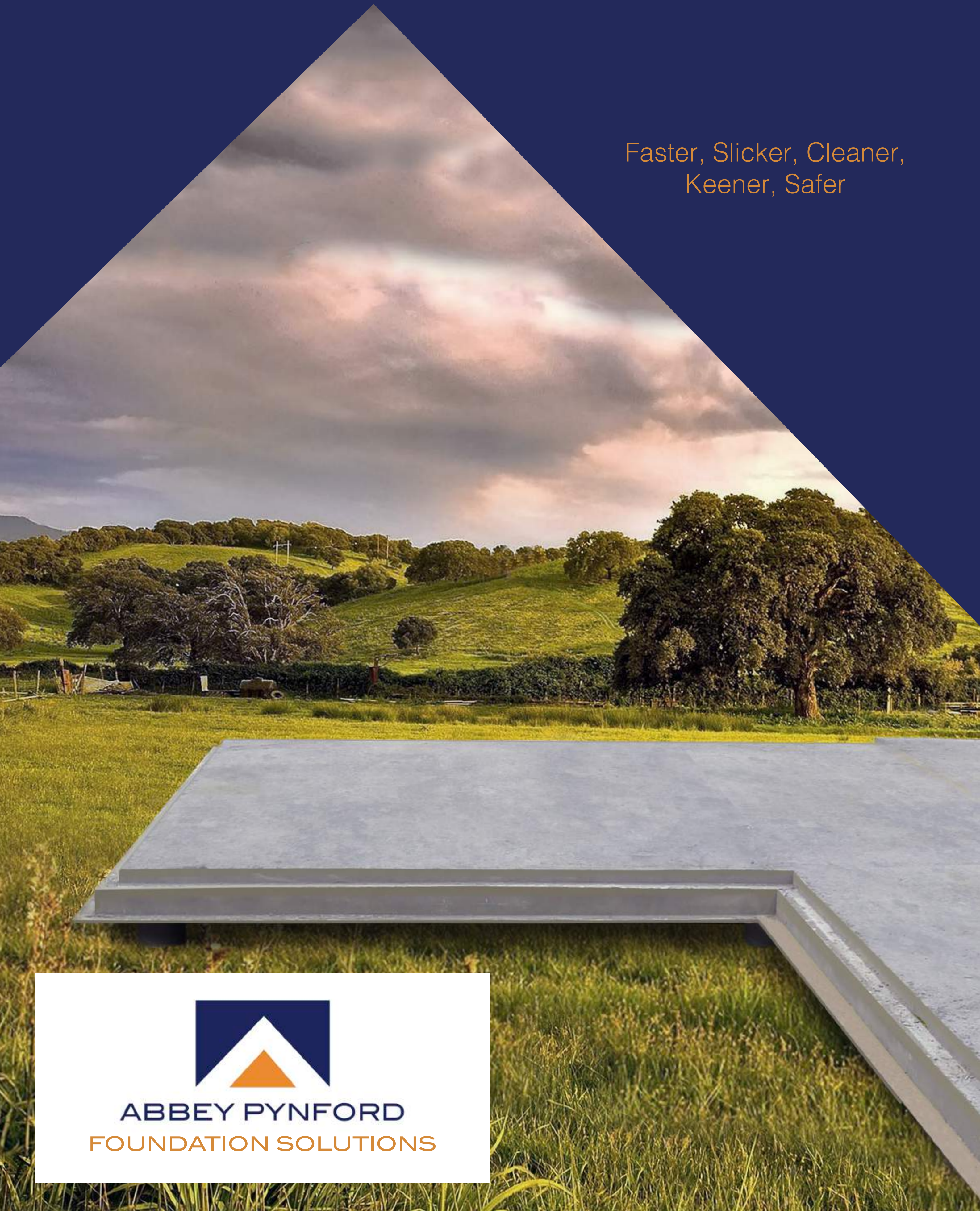
Created by: Helen Winter (20/04/2015)
Checked by: Ali Cotton (20/04/2015)

Appendix N – Foundation and Ground Floor Construction

HOUSEDECK®

Specialist Foundation and Ground Engineering Solutions

Faster, Slicker, Cleaner, Keener, Safer



ABBEY PYNFORD
FOUNDATION SOLUTIONS

Introduction to Housedeck®

Our Housedeck® Foundation system benefits over the traditional pile and beam approach.

Housedeck® replaces the following elements:

Piling mat (95% of projects)

Setting out piles

Pile trimming

Excavations for ground beams

Ground beam construction

Sub-structure brickwork

Blinding within footprint

Pre-cast floor

Resources to manage all of the above

Benefits

Faster construction time, between 1/2 and 2/3 of the traditional construction methods

More competitive

Adaptable due to ability to avoid most sub-surface obstructions

Minimal or no spoil away

Low carbon footprint

Best system to be used adjacent to trees

Service design and construction to DPM (after landscaping)

Ideal system on contaminated and **Brownfield sites**

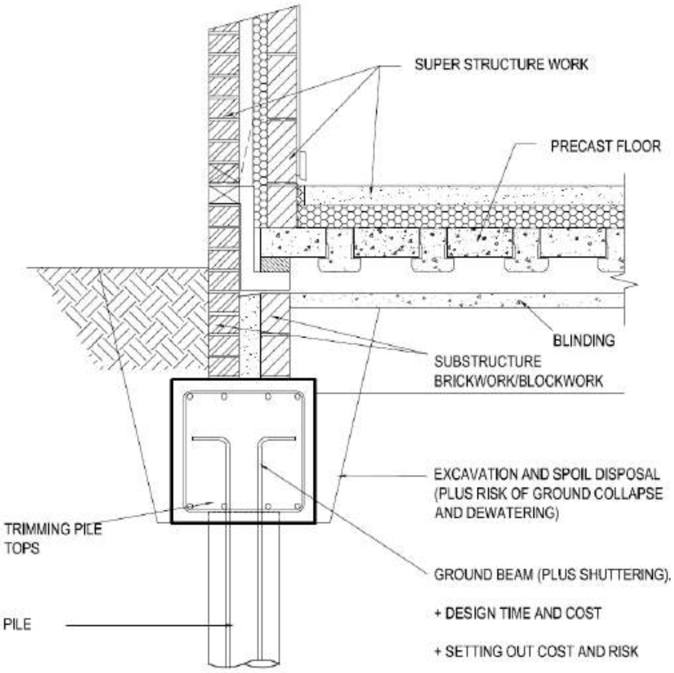
Responsive pro-active **support** on site

LABC and **NHBC** pre-appoved

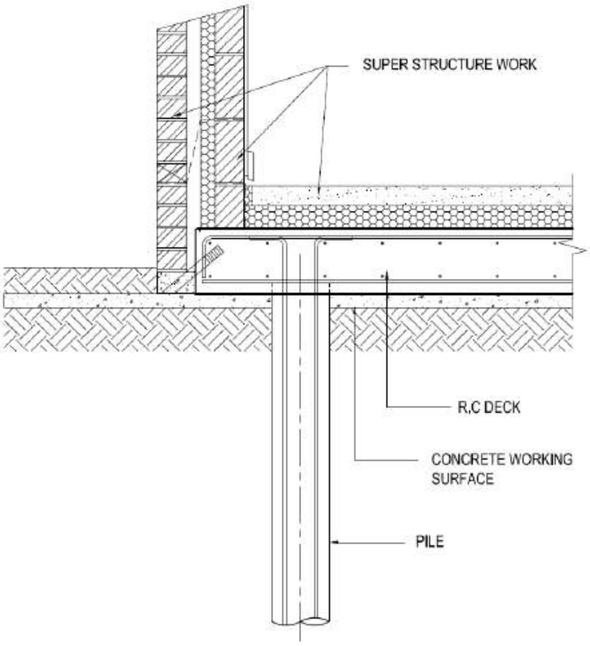
Housedeck is suited to both large multi-plots and one off sites

Pile and Beam

Traditional approach to pile and beam



Abbey Pynford's Housedeck® approach to pile and beam

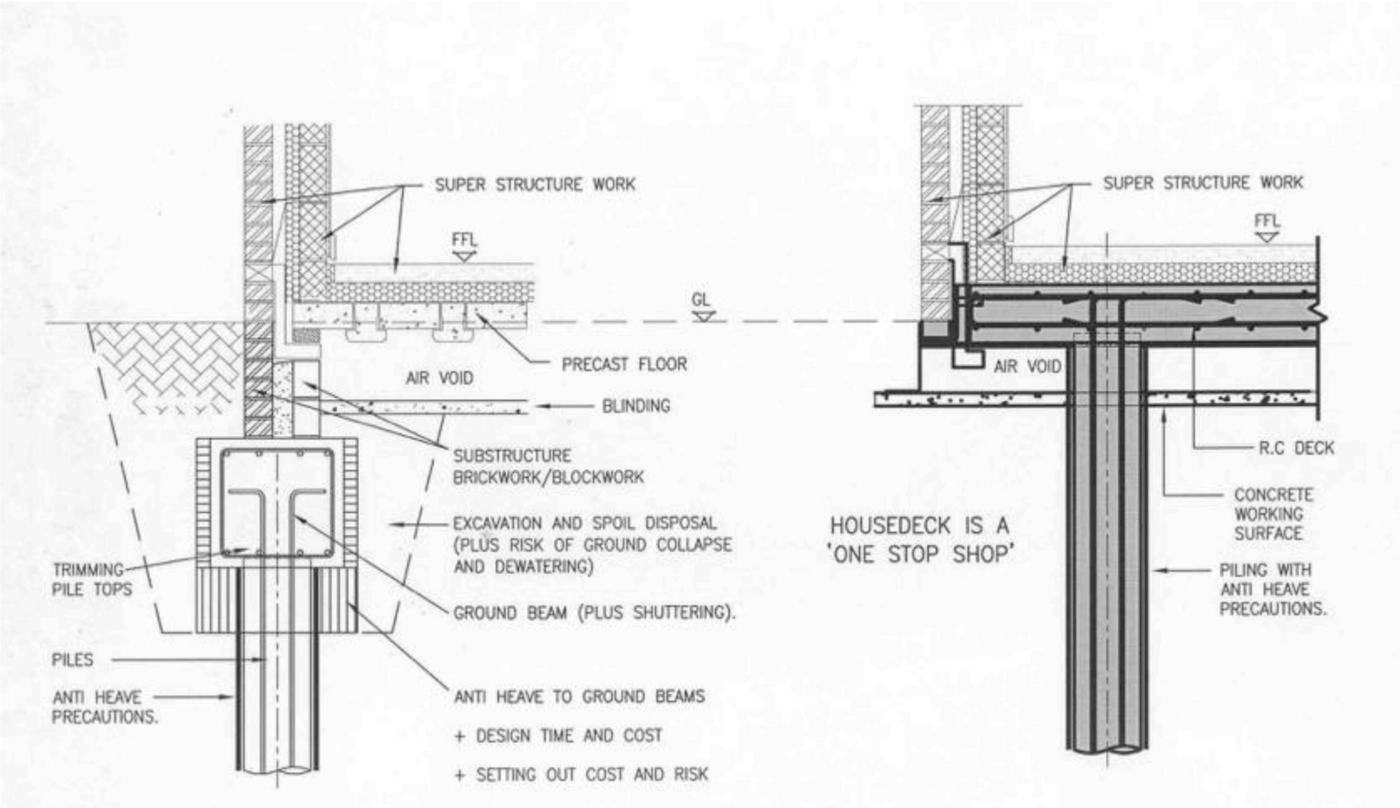


Enlarged edge detail - section

Voided

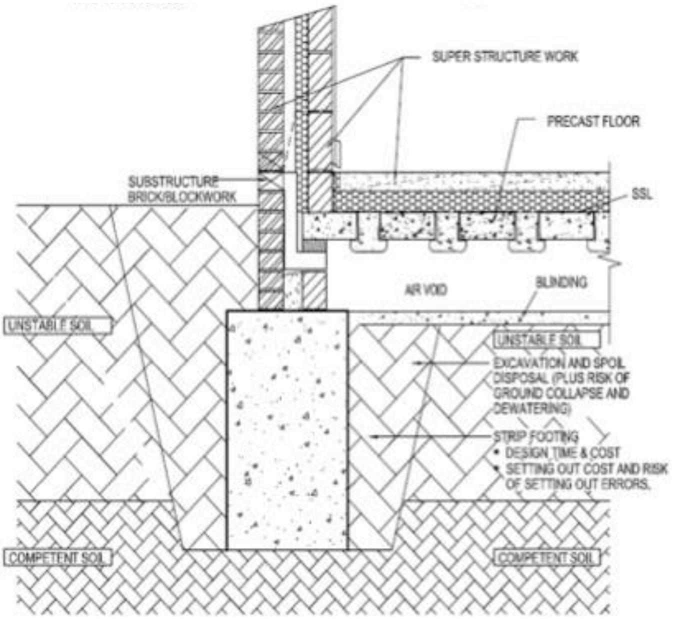
Traditional approach to driven

Abbey Pynford's Housedeck® approach with under slab void

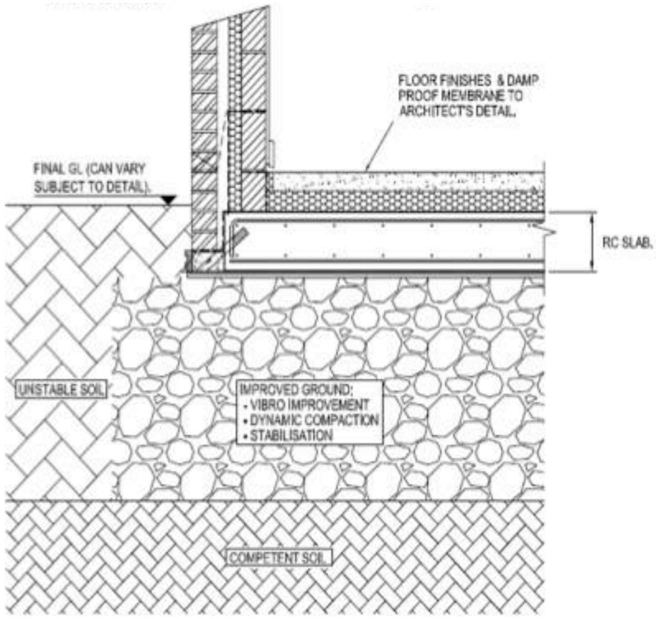


Ground Improvement

Traditional approach to excavated footings



Abbey Pynford's Housedeck® approach to improved ground



Our commitment to you:

You will receive the same attention and quality of service whether you are a small developer or corporate builder

We will provide you with a fully documented offer within two weeks after receiving all required information

Our dedicated in-house design team, using the latest software finite element analysis, ensures that each project is value engineered

We will always operate in the best practice, complying with Health, Safety and Environmental legislation

We promise to serve in your best interests and if we believe that one of your Housedeck systems is not the most appropriate scheme for your needs, we will advise you accordingly

Selection of clients:

Able Construction
Balfour Beatty
Barratt Homes
Carillion Region Building
Clancy Consulting
Corus Group
Crest Nicholson
Dacorum Borough Council
David Wilson Homes
Fairview Homes
Geoffry Osbourne
Jarvis Homes
Laing Homes
Leadbitter
Linden Homes
Mansell Construction
McCarthy & Stone
Northampton Country Council
Oakey Executive Homes
Oakwood Building Limited
Octavia Housing
Persimmon Homes
Redrow Homes
Taylor Wimpey Developments
Taylor Woodrow
V. E. Parrott Limited
Wates Construction
Watford Borough Council
Westbury Homes
Willmott Dixon Construction and Housing

Installation Methods

We offer two basic systems; Standard and Voided. Both can be constructed using Large (40t) midi (15t) or mini (1-5t) piling rigs.

Standard Ground Bearing Slab System

Concrete working surface

Used to support our piling rigs, act as a working surface for steel and brickwork support units and often saves the need for a traditional piling mat.

Piling

We offer a wide range of piling systems , however, wherever possible we elect to use a rig where the concrete working surface will replace the need for a piling mat. We have a range of rig types that are able to offer this on most Housedeck projects.

Brickwork support system (BSUs) and reinforcing

Are fixed after drainage. The BSUs are unique to us, they are fixed in a fraction of the time of traditional shutters and allow an immediate start for follow on trades.

Completion

This will often be achieved within 2/3 days of the time of a traditional foundation, it negates the need for substructure brickwork and pre cast floor slab.

Construction of Anti-Heave Incorporating Voided System

Photo shows a light weight piling rig running on a 50mm concrete working surface instead of a traditional piling mat, saving on the cost of a dig and imported material.

Deck support units are placed on the concrete working surface. Decking is then placed on our proprietary deck support units.

Subsequently, our proprietary brick support units (shown) are fixed to the tanalised plywood, followed by steel fixing, final levelling and concrete pour.

Fix void guard to prevent soil migration into void. Clear site for follow on trades (brick-work). Unique use of stainless steel brickwork support allows almost immediate start for follow-on trades.

Piling Methods
We have the widest range of Piled Support Methods:

Helical
(Helical displacement piles)

DCIP
(Driven cast in place)

Driven
(Driven concrete or steel tubes)

VCIP
(Vibro cast in place)

CHD
(Continual helical displacement)

CFA
(Continuous flight auger)

Bored
(Open bored and cased)

SFA
(Segmented flight auger)

Ground Improvement Methods
We have the widest range of Ground Improvement Methods:

Vibro replacement
(Introduces new materials)

Vibro floatation
(Densifies existing ground)

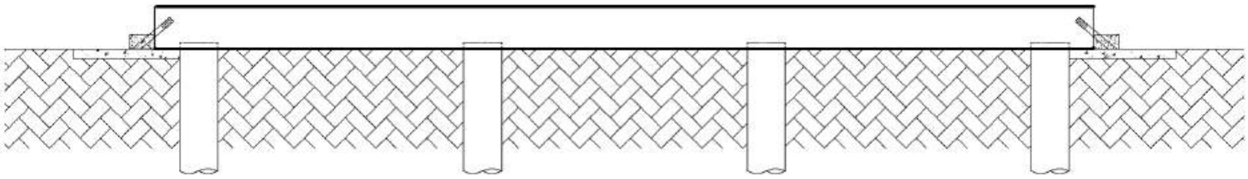
Dynamic compaction
(By drop weight or dynamic rolling)

Soil stabilisation
(Ex situ introduction of hydration powders)

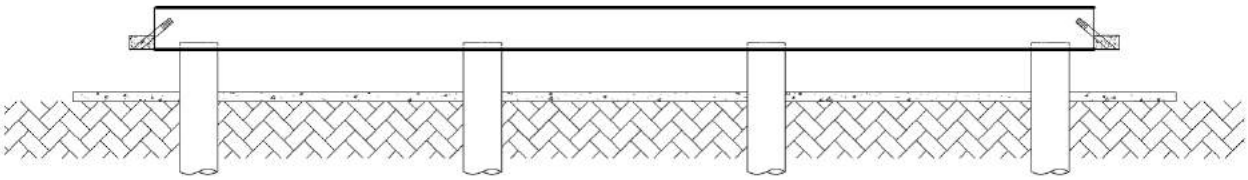
Soil mixing
(In situ introduction of hydration powders)

Engineered fill
(Often imported, may be existing treated by one of the above)

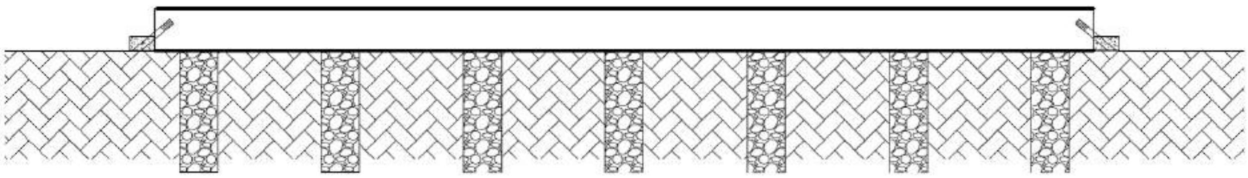
Range of systems for all ground conditions



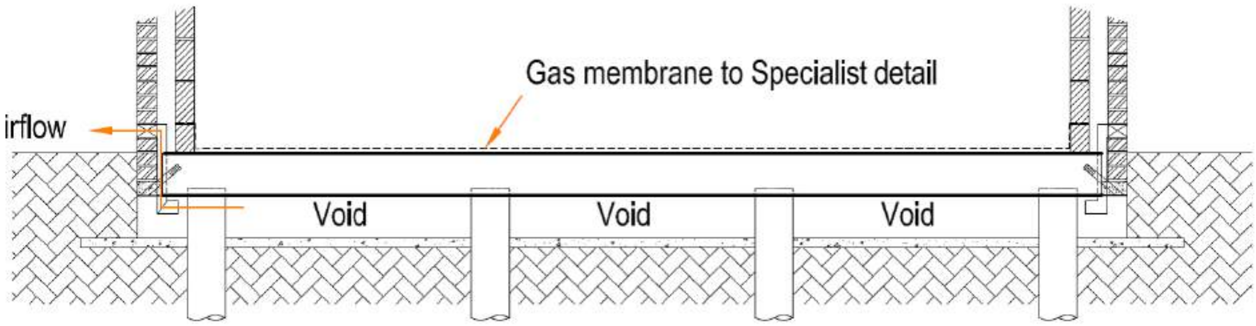
OPTION 1
Standard ground bearing slab



OPTION 2
Anti heave incorporating void
(for highly shrinkable clays)



OPTION 3
Vibro replacement stone columns/
Ground improvement techniques

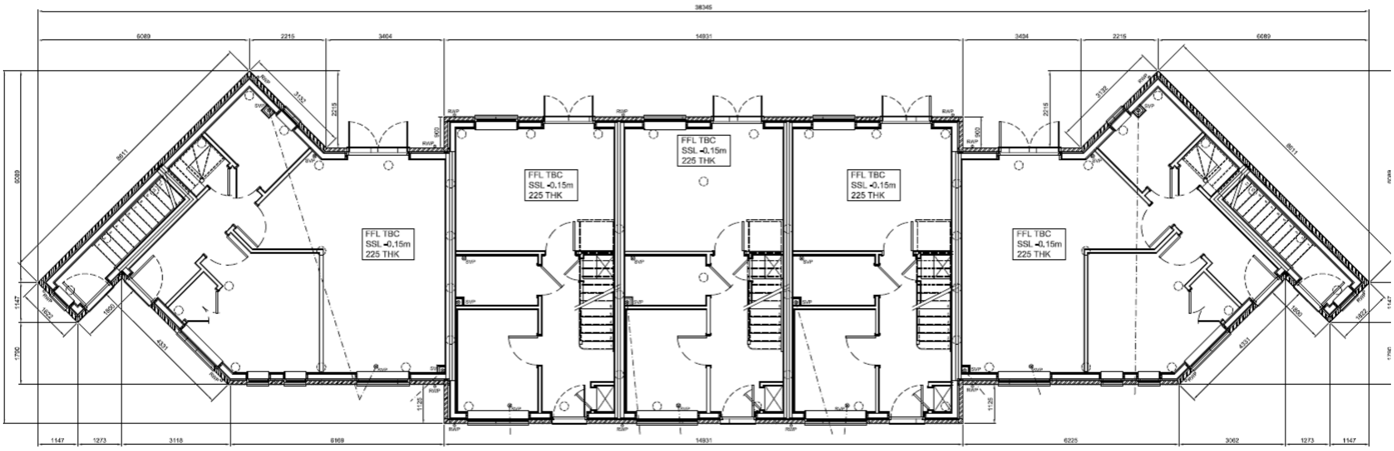


OPTION 4
Gas membrane (for contaminated
and radon sites)

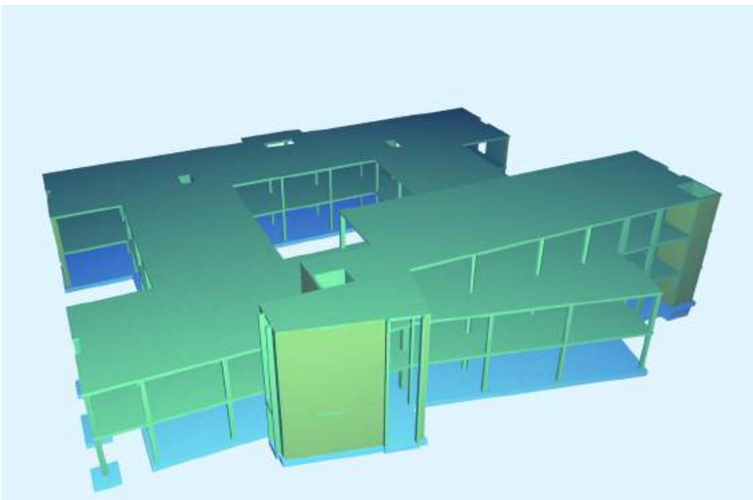
Design

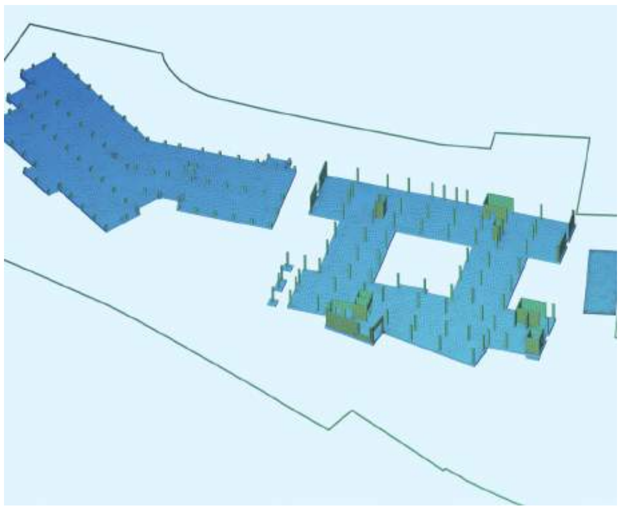
Housedeck® and Comdeck® are complete design and build packages. Abbey Pynford’s in-house Design Department provides detailed pile layout drawings, slab design, bending schedules and calculations for submission to Local Authority Building Controle (LABC) or NHBC. The design software Sofistik is one of the most advanced flat slab software packages in Europe. This software allows piles to be relocated on screen and immediately produces pile loads and reinforcing steel requirements thus allowing a prompt response in addressing the presence of obstructions such as tree roots and man made obstructions.

SOFiSTiK
FINITE ELEMENT SOFTWARE



3D Finite Element Software - provides in-house capability for modelling structures in 3 dimensions, using finite element methods in an AutoCAD environment





Services

Our service always includes:

- Design of deck
- Design of piling or ground treatment
- Setting out
- When no piling mat used – construction of the concrete working surface
- Pile trimming or post ground treatment protection
- Drainage to edge of footprint
- Proprietary decking system (replaces floor slab and substructure brickwork)
- Site management and customer liason
- QMS system
- Pro-active service

Our service can also include:

- Site investigation
- Design of engineered fill
- All drainage installation
- Associated pre-foundation groundworks, site strip etc.
- Fixed price
- Warranty
- Retaining walls
- Lift pits
- Bolt boxes
- Starter bars for RC frames
- Other associated structural features

Introduction to Comdeck®
Industrial and Commercial Foundations

Comdeck® is a natural extension of our proven Housedeck® system, but designed to support heavier loads as are typically imposed by industrial and commercial structures. This proprietary piled raft style foundation system uses the same principles as the Housedeck® system – the significant differences being that pile sizes tend to be larger and slab thickness is typically (although not always) increased from 225mm to 300+mm.

The benefits of Comdeck® are normally the same as those for Housedeck® although speed is often the most attractive benefit.

Chipping Hill Primary School

The school commissioned by Essex county council is desperately required to meet the rising demand in the area and replace the existing outdated and failing facilities. By re-engineering the original scheme which required anti heave precautions, piles and ground beams was converted to the Comdeck slab system supported off stabilised ground. This treated ground enabled Abbey Pynford to eliminate the piles and slim down the overall construction build-up, this drastically reduced off site spoil disposal.

Area: 1500 sq. m
Programme: 8 weeks

Harris Academy

The programme was to install Vibro & CFA piles with ground beams, pilecaps, lift pits, service pits, drainage, underslab services, ground preparation, underslab insulation and cast concrete ground floor. Despite an adverse change in ground conditions, an additional site investigation, redesign and associated change to scope of works, the contractual period was not exceeded.

Area: 6650 sq. m
Programme: 14 weeks

Portway Primary School

Abbey Pynford proposed to remove the requirement for piles by treating the granular made ground by using a variable amplitude roller. This considerably reduced the risk on site by removing the requirement to install bored piles through the asbestos contaminated Made Ground. Due to Abbey Pynford's in house geotechnical expertise the raft thickness was reduced to 300mm offering considerable savings.

Area: 406 sq. m
Programme: 6 weeks

ABBEY PYNFORD

Abbey Pynford work with clients from design through to project competion in the following areas:

- House Foundations - Housedeck®
- Industrial/Commercial Educational Foundations - Comdeck®
- Piling - open site, restricted site, augered, bored and driven
- Mini-piling
- Commercial and Domestic Underpinning - widest range of underpinning systems available
- Industrial Machinery Foundations
- Space creation in the refurbishments market (high level Pynford beams)
- Lifting and moving structures and buildings
- Basements

ABBEY PYNFORD

Abbey Pynford Limited
IMEX, 1st Floor West Wing
575-599 Maxted Road
Hemel Hempstead
Hertfordshire
HP2 7DX

Tel: 01442 212112
Email: info@abbeypynford.co.uk

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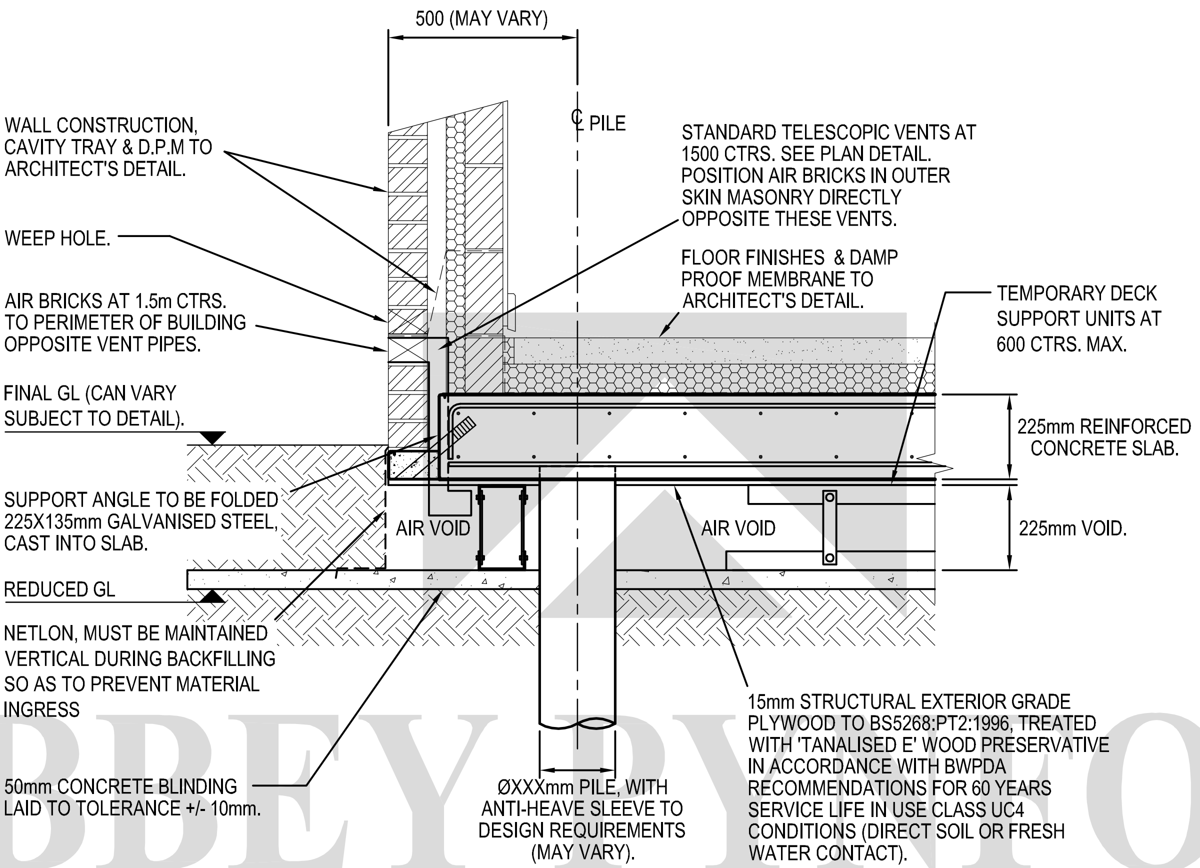
www.smasltd.com
as recognised by SSIP SAFETY SCHEMES IN PROCUREMENT

SAFEMARK
HEALTH AND SAFETY APPROVED BY NHBC
certification mark

DO NOT SCALE: Contractor to check all dimensions and report any omissions or errors



TYPICAL SECTIONAL ELEVATION FOR 225 THICK VOIDED SLAB WITH BLOCKWORK INNER SKIN

SCALE 1:20

Rev	Description	By	Chk	App	Date

ABBEY PYNFORD
Second Floor, Hille House,
132 St. Albans Road,
Watford, Herts. WD24 4AQ
Tel : 0870 085 8400 Fax ; 0870 085 8401
E mail : info@abbeypynford.co.uk
Web : www.abbeypynford.co.uk

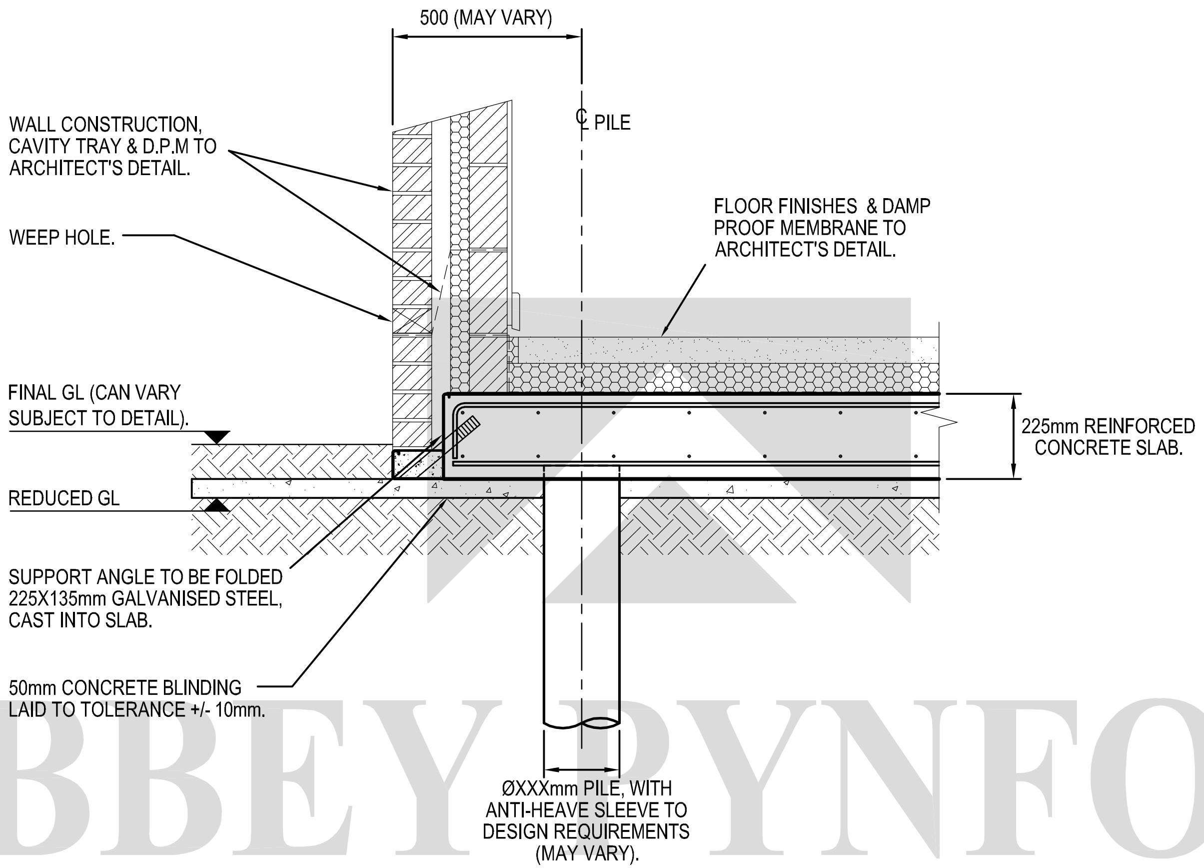
Scale at A4	Drawn By	Date	Checked By	Date	Approved By	Date
1:20	HD	11-2012	PCN	11-2012		

Project No.	Office	Type	Drawing No.	Revision
	W	HD	01	-

Drawing Title:
TYPICAL SECTIONAL ELEVATION FOR 225 THICK VOIDED SLAB WITH BLOCKWORK INNER SKIN WITH VENT

DO NOT SCALE: Contractor to check all dimensions and report any omissions or errors

DETAIL FOR INDICATIVE PURPOSES ONLY AND MUST BE TO ARCHITECT'S DESIGN AND APPROVAL.



TYPICAL SECTIONAL ELEVATION FOR 225 THICK SLAB WITH BLOCKWORK INNER SKIN

SCALE 1:20

Rev	Description	By	Chk	App	Date

ABBEY PYNFORD
Second Floor, Hille House,
132 St. Albans Road,
Watford, Herts. WD24 4AQ
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Scale at A4	Drawn By	Date	Checked By	Date	Approved By	Date
1:20	HD	11-2012	PCN	11-2012		

Project No.	Office	Type	Drawing No.	Revision
	W	HD	20	-

Drawing Title:
TYPICAL SECTIONAL ELEVATION FOR 225 THICK SLAB WITH BLOCKWORK INNER SKIN

Appendix O – Road Improvement Systems

Details and Examples of the Road Improvement System for New Salts Farm

Project Number - 11649
New Salts Farm Road Land – Shoreham

To create a stable road for the New Salts Farm development, it is proposed to use a 3 dimensional geo-grid, filled with open graded stone to form a road foundation. This system is specifically developed for working in very poor ground conditions, as the vehicle loads are transferred horizontally rather than vertically. Examples of the Geoweb road improvement system and product literature are included within this document. All the examples have very poor ground conditions (probably more onerous than New Salts Farm) of which, two specifically refer to constructing roads in saturated soils.

It is proposed that all the roads and parking areas on New Salts Farm will be of a permeable road construction, with the potential for base infiltration. The Geoweb system works well for permeable roads as the cells will be filled with large open graded stone and the side walls of the geo-grids are slotted to allow for horizontal migration of water. Consequently, Geoweb is a standard detail for constructing roads in no-dig root protection zones.

A number of CBR tests have been taken on Phase 1 and a value of 3% has been recommended, which takes account of the potential for high ground water. The Geoweb system is designed to work on ground which is significantly softer than this and we have successfully used it on a private estate road in Tunbridge Wells. This construction is very similar to that which is proposed on New Salts Farm, in that it is permeable with base infiltration on potentially soft ground. Attenuation is also provided within the voids of the sub-base material. The road was constructed during a very wet winter (2012) where the ground had become saturated with no recordable CBR value at formation level. The road had to be constructed to enable the adjacent houses to be built. The Geoweb system was introduced into the road make-up which enabled the construction of the road to be completed. The attached photos were taken in April 2016, where the road shows no signs of distress.

A road design (attached) by Soiltec has been undertaken based on a CBR value of 3%, with construction traffic loading. It should be noted that the construction traffic will be more onerous in terms of loading design than when the road is completed. During the construction phase a macadam base course is generally used over the Geoweb system as a temporary running surface. This is then either removed or holes punctured through it (to allow percolation) at the end of the construction phase. The final surface courses are then added.

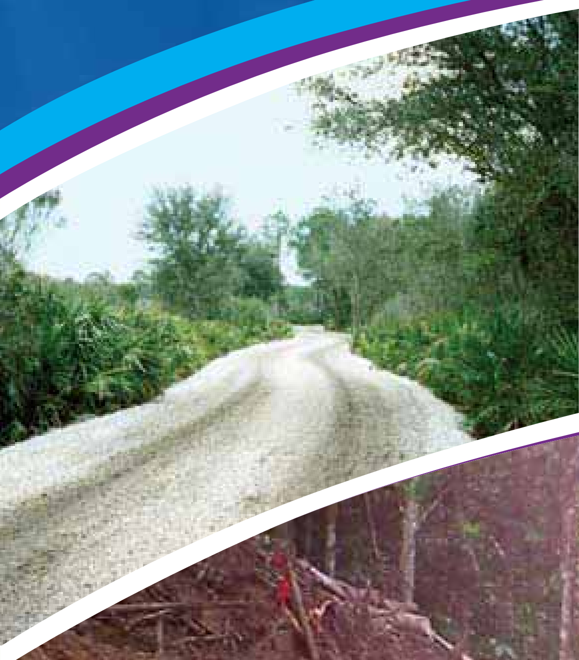
Ground water monitoring wells have been installed across New Salts Farm, of which, 3 are located within Phase 1. They have established that the ground water levels within Phase 1 fluctuates with the tide, with approximately a 2-hour time lag between high tide levels and high ground water levels. During high tide events ground water levels typically get to within 0.4m below ground, dropping to 1.0m below ground level at low tides. However, there are areas on the site where ground water levels do, on occasion, reach ground level. It is intended that the road level will be raised slightly above ground level so that the formation level will generally be above the high tide water level. However, should the ground water level rise further, the introduction of the Geoweb system will provide additional support.

The Geoweb system is specifically designed for roads in very poor ground conditions and is regularly used in Germany and America. Whilst the recorded CBR values are not particularly poor, the fluctuating ground water levels could potentially impact on the construction of the road. The introduction of the Geoweb system will provide good structural stability during the construction phase as well as providing long term stability of the road.

Presto®

GEOSYSTEMS

GLOBAL LEADER • GLOBAL PARTNER



creating

sustainable

environments™

GEOWEB®

load support

APPLICATION OVERVIEW

our commitment:

providing the highest quality

products/solutions

eco-economic solutions for load support

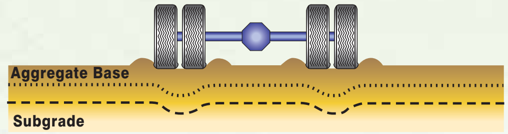
GEOWEB®

the GEOWEB® *system*

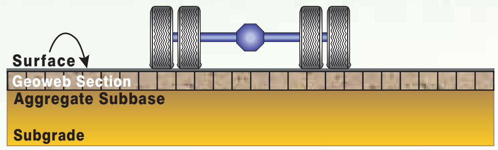
LOW-COST LOAD SUPPORT SOLUTIONS

The Presto Geoweb® load support system is a highly effective, economical solution to unacceptable road, parking, and yard surface problems that result from subgrade material failure or base material instability. Under concentrated or distributed loads, the three-dimensional cellular structure confines infill material and controls shearing, lateral and vertical movement of the infill material.

As a base stabilization system under pavement, the Geoweb material significantly improves pavement life cycle costs. When confined, **base material requirements can be reduced by 50% or more** by substantially reducing the loading on sub-surface soils. As a result, reduced excavation and granular infill needs reduce overall installation cost.



UNCONFINED GRANULAR PAVEMENT SYSTEM



THE GEOWEB GRANULAR PAVEMENT SYSTEM

As a surface stabilization system, the Geoweb structure distributes surface pressures for dynamic and static loading, controlling rutting and **reducing long-term maintenance requirements and costs**. Using permeable infill with a high porosity, the system offers environmental and stormwater management benefits.

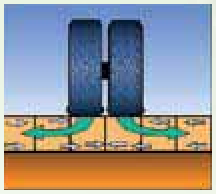
With topsoil/aggregate infill, the Geoweb material can create a vegetated surface that supports occasional loads.

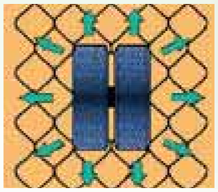
GEOWEB® *system benefits*

- Produces a stiff base with high flexural strength; acts like a semi-rigid slab by distributing loads laterally.
- Minimizes impact of differential and overall settlement even on low-strength subgrades.
- Increases effective structural number, reducing fill depth requirement by 50%.
- May allow use of poor-quality granular fills in place of more costly imported materials.
- With permeable infill, reduces stormwater runoff and effectively performs as an on-site stormwater retention/detention "basin" storage, reducing need and costs for stormwater ponds.

TYPICAL APPLICATIONS:

- Permanent and temporary site access roads
- Permeable, load-supporting surfaces
- Intermodal/port facilities
- Transportation/storage yards
- Roadway shoulders (vegetated or porous aggregate)
- Stabilized base for asphalt/modular block pavements
- Stabilized drainage layer
- Railroad track ballast / subballast structures
- Trails and walkways
- Boat ramps and low water crossings
- Pile cap structures
- Foundation mattresses & pipeline protection
- Trench inverts
- Driveable vegetated surfaces





RESULTS SUPPORTED BY RESEARCH

Test results from numerous research initiatives confirm the benefits of confined aggregate within the Geoweb cellular confinement system vs. unconfined aggregate.

- Reduces thickness and weight of structural support elements by 50 percent or more.
- Allows subgrade materials to withstand more than 10 times the number of cyclic-load applications before accumulating the same amount of permanent deflection.
- Provides over 30% stress reduction when supporting aggregate under pavement.
- Distributes load between pilings, reducing intersoil stress by 40%.

key application areas

The Geoweb® system creates a stabilized structural support system, providing considerable benefits to unstable soils in key areas:

1. Base Stabilization under Paved Surfaces/Subsurfaces
2. Surface Stabilization for Unpaved Permeable Surfaces
3. Load Distribution System over Weak Soils
4. Flexible Concrete Mat

1 *base stabilization*
UNDER PAVED SURFACES/SUBSURFACE

As base support, the Geoweb® load support system creates a stabilized layer under asphalt, concrete or modular block pavement that holds up under heavy, repeated traffic. The system acts like a semi-rigid slab, distributing loads laterally and reducing subgrade contact pressures.

Selection of infill materials for base stabilization is determined by anticipated load characteristics and overall performance requirements. The system is especially effective in soft-soil areas where substantial pavement problems and regular maintenance costs exist or are anticipated as a result.

STABILIZING BASE MATERIALS WITHIN THE THREE-DIMENSIONAL GEOWEB® SYSTEM:

- Requires 50% or less base material when material is confined to achieve the same load support requirements.
- Minimizes load-related deformation and settlement, and reduces pavement degradation and cracking typically associated with soft subgrades.
- Allows the use of lower quality sand and aggregate materials, even over soft subgrades.
- Proven solution for challenging soft-soil stability problems.

2 *surface stabilization*

FOR UNPAVED AND PERMEABLE SURFACES

With permeable infill, the Geoweb® surface stabilization system provides a cost-effective alternative to hard surface pavements with many environmental benefits. By confining aggregate infill, the system improves the load distribution characteristics of unpaved roads and pavement areas, reducing long-term maintenance requirements and costs.

Grass pavement systems offer structural support and the desirable aesthetics of green space, ideal for infrequent traffic requirements.

STABILIZING INFILL MATERIALS WITHIN THE GEOWEB SYSTEM:

- Distributes pressures from dynamic and static loadings throughout the system, reducing lateral and vertical displacement of the infill and undesirable surface rutting.
- With aggregate infill, performs double duty as a load support system and an on-site water detention/retention storage "basin"; may eliminate requirements and costs for on-site stormwater containment systems.
- Reduces stormwater surface runoff, maximizes groundwater replenishment.
- Creates a cooler surface, reducing the heat island effect associated with hard surface pavements.
- Contributes to green building LEED® credits for stormwater management and reducing heat island effect

3 *load distribution system*

OVER WEAK SOILS

The Geoweb® system creates a stabilized base layer, significantly reducing excavation and base material requirements, especially over low-strength subgrades. The system minimizes load deformation and settlement and is especially effective when constructing pavements in coastal or soft-soil areas where infill material requirements and costs are high.

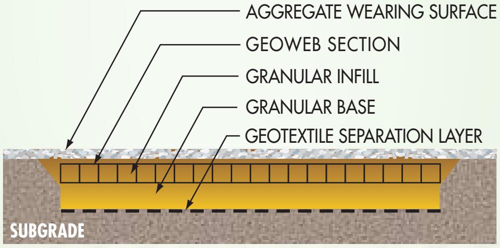
4 *flexible concrete mat*

Flexible concrete-filled Geoweb® mats are quickly created for applications such as boat ramps, low water crossings, or as flexible cover mats for utility protection.

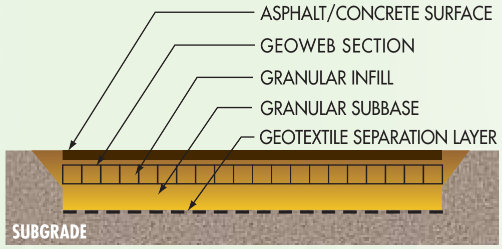
key components

The complete Geoweb® load support system application may include some or all of the following:

- Geoweb sections
- Cell infill materials
- Geotextile separation layer
- ATRA® Key connection device
- ATRA® Anchors
- Geocomposite drainage materials
- Integral polymeric tendons
- Fasteners



SURFACE STABILIZATION SYSTEM



BASE STABILIZATION SYSTEM

size options

Geoweb® sections are available in various cell sizes, cell depths and section lengths to address specific project needs. Load support system details are influenced by the characteristics of subsoil strength, applied load, available granular infill and surface type. Generally, the heavier the applied load and/or the poorer the quality of subsoils, the greater the required cell depth.

integral system components

The following components may be integrated to facilitate and expedite construction or to meet engineering requirements:

ATRA® ANCHORS

In load support applications, anchors are typically not part of the permanent design requirements but rather used to aid construction. Used with 1/2 inch rebar stakes or 10-12 mm dia. rods, ATRA® Anchors are easier to drive than J-hook stakes, improving installation productivity. (1)

When specific conditions dictate permanent anchoring an engineering array of surface anchors may be used.

ATRA® KEY CONNECTION DEVICE

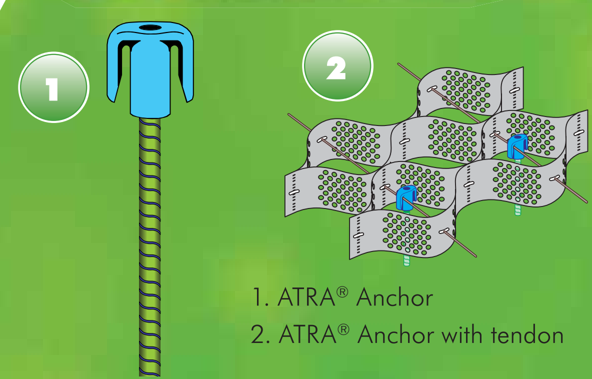
For quick and easy connection of Geoweb® sections, the exclusive ATRA® key connection device reduces contractor installation costs and provides a three-times-stronger connection of Geoweb® sections.

TENDONS

Tendons may be required for providing additional hold-down and stability in the following applications:

- Traffic loadings on a grade
- Wet or saturated soil conditions on trails or access roads through wetlands
- Boat ramps or low-water-crossing applications

Tendons and an ATRA® Anchor array provide additional anchoring to resist uplift forces. (2)



1. ATRA® Anchor
2. ATRA® Anchor with tendon

comprehensive tools and services

Presto Geosystems and our distributors/representatives offer the most-complete services in the industry to support project design and installation requirements.

TOOLS:

- Technical resources binder/CD
- Engineering analysis/technical overviews
- Online SPECMaker® specification development tool
- Project case studies
- Detailed construction instructions

SERVICES:

Project Evaluation Service: We provide engineering analysis of specific project needs and provide recommended preliminary designs for each project.

Construction Services: Qualified on-site field support specialists can be available for construction training, and start-up installation supervision.

PRESTO GEOSYSTEMS' COMMITMENT — *To provide the highest quality products and solutions.*

Presto Geosystems and our worldwide network of knowledgeable distributors/representatives are committed to helping you apply the most cost-efficient solutions to your load support requirements. Our solutions-focused approach to solving problems adds value to every project. Rely on the leaders in the industry when you need a solution that is right for your application.

LEADING-EDGE INNOVATION

Presto is the original developer of the cellular confinement technology and leads the industry in research and development resulting in meaningful product improvements, innovative features, advanced engineering methodologies, proven field results and ultimately long-term solutions to challenging problems.

UNSURPASSED QUALITY

Presto's commitment to quality begins with manufacturing and continues through final installation.

- Quality management system certified to ISO 9001:2000 CE Certification.
- Sections manufactured from high-quality polyethylene provide consistent and maximum seam weld strength.
- Materials engineered to established geosynthetic industry guidelines.
- Sections backed by a 10-year limited warranty.

Presto®

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Geosynthetic spins 'web' of success

Twelve years after installation, polyethylene cellular confinement system holds Wisconsin highway's base course in place as rest of road is prepared for reconstruction

edited by Larry Flynn

The polyethylene geosynthetic material is installed in 1984. Aluminum stretcher frames, used to expand the material, are removed after the material is infilled.

Wisconsin winters are notorious for the damage they can cause to roads. An example of winter's influence was seen on Highway E, a winding, two-lane asphalt road that serves commuters between Little Chute and Oneida. Once called the worst stretch of road in Outagamie County, it was plagued by cracking and heaving during the winter months. Similar problems occur elsewhere in the region where sub-base sand silt pockets enclosed in the state's thick clays become highly saturated and freeze.

The speed limit on a half-mile stretch of Highway E had to be reduced from its normal 55 mph to 15 mph during the winter months. "Heaving could start as early as November and be a problem through March," said Mike Marsden, Outagamie County Highway Commissioner. "We could tell when the frost was out of the ground in the spring because the road would flatten out. We'd put up flashing barriers and advance warning signs. It was really difficult to plow snow in the area."

Little did Marsden know that he'd find a solution to the heaving problem in his own county. The corporate offices of Presto Products Company and the Geosystems group are located in Appleton, Wis. Presto Products Company helped pioneer cellular confinement technology in cooperation with the Army Corps of Engineers in the late '70s. The company's Presto Geoweb Cellular Con-

Defining a geosynthetic: Type and functions

The first use of fabrics in reinforcing roads was attempted by the South Carolina Highway Department in 1926. A heavy cotton fabric was placed on a primed earth base, hot asphalt was applied to the fabric, and a thin layer of sand was put on the asphalt. The department published the results of this work in 1935, describing eight separate field experiments. Until the fabric deteriorated, the results showed that the roads were in good condition and that the fabric reduced cracking, raveling and localized road failures. This project was certainly the forerunner of the separation and reinforcement functions of geosynthetic materials as we know them today.

In all, geosynthetics, perform five major functions: separation, reinforcement, filtration, drainage and moisture barrier. There are six families of geosynthetics: geotextiles, geogrids, geonets, geomembranes, geocomposites and "geo-others."

- **Geotextiles** form the largest group of geosynthetics. They are indeed textiles in the traditional sense, but consist of synthetic fibers rather than natural ones, such as cotton, wool or silk. Geotextiles are porous to water flow across their manufactured plane and also within their plane, but to a widely varying degree.
- **Geogrids** represent a small but rapidly growing segment of the geosynthetics area. Rather than being a woven, nonwoven or knit textile (or textile-like) fabric, geogrids are plastics formed into a very open, gridlike configuration, i.e., they have large apertures. Often they are stretched in one or two directions for improved physical properties. By themselves, there are at least 25 application areas, and they function in two ways: reinforcement and separation.
- **Geonets** constitute another specialized segment of the geosynthetics area. They are usually formed by a continuous extrusion of polymeric ribs at acute angles to one another. When the ribs are opened, relatively large apertures are formed in a netlike configuration. Their design function is completely within the drainage area where they have been used to convey fluids of all types.
- **Geomembranes** represent the second largest group of geosynthetics, and in dollar volume their sales are essentially equal to that of geotextiles. They are impervious thin sheets of rubber or plastic material used primarily for linings and covers of liquid- or solid-storage facilities. Thus, their primary function always is as a liquid or vapor barrier. The range of applications, however, is very wide.
- **Geocomposites** consist of geotextile and geogrid; or geogrid and geomembrane; or geotextile, geogrid, and geomembrane; or any one of these three materials with another material (e.g., deformed plastic sheets, steel cables, or steel anchors). Major functions of this creative effort encompass the entire range of five functions listed for geosynthetics discussed.
- Geo-Others is a general area of geosynthetics that has exhibited such innovation that many systems defy categorization. For want of a better phrase, geo-others describes items, such as threaded soil masses, polymeric anchors, and encapsulated soil cells. As with geocomposites, their primary function is produce-dependent and can be any of the five major functions of geosynthetics. •

The above information war excerpted from the book Designing With Geosynthetics, *Second Edition, by Robert hf. Koemer, Ph.D., P.E. Copyright 1990 by Prentice Hall, Englewood Cliffs, N.J.*

finement System is designed to strengthen structural fill by increasing its shear strength and stiffness.

The system originally was developed for building roads across insufficiently supported grounds, such as beach sands. The system was used to build sand roads for rubber-tired vehicles during the Persion Gulf War.

"Our first project with Outagamie County was at the county landfill site," said Gary Bach, product manager. "We used the system to construct an access road into one of the landfill cells. That application was a success and led us to the Highway E project." To combat Highway E's washboarding pavement, the county worked with the company in 1984 to produce a cross-section design of the road. Presto staff were on site during installation.

The system is an expandable honeycomb-like structure made of high-density polyethylene. The system is designed to produce a stiff base with high flexural strength. According to the company, under load, the system generates powerful lateral confinement forces and high soil-to-cell wall friction. It is to provide a bridging action and improve the long-term load deformation performance of common granular fill materials.

On Highway E, an 8-in. deep system was installed in the problem area. The asphalt pavement was removed and stored for recycling and final topping after reconstruction. The silty clay subbase was cut down 18 in. below the water table level and covered with a geotextile. Next, a 6- to 8-in. layer of 3½-in. to 4½-in. clean crushed stone was added. The system was expanded, positioned and secured at the edges with granular fill. It was then infilled with sand and topped with a 15-in. base course of crushed stone. The completed area was then compacted with a vibratory roller and was immediately ready for traffic.

"Because of our soil conditions, we always use a 15-in. base course on all of our roads, Marsden said. "We probably could have gotten by with less, but we decided not to. If we hadn't used the confinement system, we probably would have reworked the subgrade and added 2 ft of base course. Even though we had fill material available just 6 miles away, it was less expensive to complete the renovation project with the Geoweb material than without it."

Unpaved, the road performed well throughout the following winter, and was surfaced with the recycled asphalt in the summer of 1985.

The system was installed in Highway E in 1984. Now, more than 10 years later, the road still is level and holding up well under all weather conditions. The highway is scheduled to be rebuilt in 1997. The county will widen the highway, fill some valleys, improve sight distance and flatten curves.

"We're not going to touch the section of the road that has the confinement system in it," Marsden said. "The road will be widened, but we won't alter the alignment or the system. We're very pleased with the way it has solved the problems for us on Highway E." •

Reprinted from March 1996 **Roads & Bridges** Magazine

Geoweb carries the load

Cellular confinement system stabilizes poor soil base

"One area that many counties struggle with in road construction is poor soils," says Tom Byle, maintenance and local road construction engineer for the Kent County Road Commission, Kent County, Mich. "Many country roads in America were built over muck, swamps or other similar, inadequate soils. Most counties cannot justify the expense of the standard peat-removal treatment, digging the peat out and replacing it with sand. Many counties end up leaving the road gravel or, if the road is blacktop, shimming the settlements with blacktop until its thickness is literally measured in feet."

Paved surfaces that are built over weak soils can experience subsoil movement and base deterioration, which causes deflections, rutting and cracking in the asphalt surface. One way to combat this problem is to stabilize the weak base and increase the load capacity of those soils. The Geosystems® Group of Presto Products Co., Appleton, Wis., provides a product, the Geoweb® Cellular Confinement System, which is designed to help stabilize road bases for paving.

Cellular confinement comprises base

The Geoweb cellular confinement system is an expandable, honeycomb-like structure made of high-density polyethylene. The three-dimensional network of interconnected, perforated cells are filled with select infill materials, such as aggregates, topsoil, crushed limestone, concrete or a combination of these materials. For structural infills, native soils can be used as long as the fraction of fines, such as silt and clay particles, does not exceed 10 percent and the plasticity index of the material is less than 6 percent. The maximum size of aggregate that can be used to fill the cells depends on the cell depth and radius, neither of which should the aggregate exceed. The chosen aggregate must also be able to stay inside the cell when acted

These two photos show the before and after appearance of 108th Street in Caledonia Township, Mich. The wet, swampy soil (above) gave way to a 200-foot (61-m) mud wave while crews were trying to cut the subgrade. Once the base had been stabilized (at right), the road was paved with asphalt and, four years later, remains smooth and free from deflections, cracks or rutting.

upon by the forces of gravity and fluids.

The Geoweb system confines the base material within its cell walls, decreasing the rate of the infill material's lateral movement and creating passive resistance between adjacent-filled cells. By adding cohesion to cohesionless material, the Geoweb system strengthens the structural fill used in load support applications. The product helps create a stiff base and acts as a semi-rigid slab by distributing loads laterally and cutting down on subgrade contact pressures.

According to Presto Products Co., by using confined versus unconfined aggregate, the material thickness can be reduced by 50 percent. In addition, when aggregate is confined within cellular walls, the subgrade materials may withstand more than 10 times the number of cyclic load applications before accumulating the amount of deflection that an unconfined aggregate experiences, according to the manufacturer. "The Geoweb helps maintain integrity of the pavement base material to the point where it degrades much slower — at the rate of 12 to 15 times slower," says Dan Senf, Presto Products marketing manager. "It can extend the life of the pavement from four to six years to 50 to 60 years."

The Geoweb sections come in various sizes, cell sizes and cell depths. A standard section measures 8 feet (2.4 m) wide by 20 feet (6.1 m) long, with special section lengths ranging from 2 feet (0.6 m) to 30 feet (9.1 m). Larger cell sections are 8 feet (2.4 m) wide by 40 feet (12.2 m) long, with special section lengths ranging from 4 feet (1.2 m) to 60 feet (18.2 m). The standard cell size is 9.6 inches (244 mm) by 8 inches (200 mm), with larger cells measuring 19.2 inches (488 mm) by 16 inches (400 mm). The cell depths for both cell sizes are 3 inches (75 mm), 4 inches (100 mm), 6 inches (150 mm) and 8 inches (200 mm). Cell depth and size is determined by the specific application and details of the base that is being stabilized.

Each section is connected to another section to prevent movement during the infilling operation. Heavy-duty metal staples from a pneumatic stapler are used to connect units. The Geoweb cellular confinement system is anchored into the ground by stakes that bear against the top of the cell wall or against tendons passing through the cell.

The sections also come in two cell types: perforated and nonperforated. The perforations along the cell walls allow water to move from cell to cell, reducing unwanted ponding and providing lateral drainage. They also help in preventing a shear plane from developing between the infill and cell walls.

Geoweb provides stable bases in Michigan

"There are specific benefits for using the Geoweb system with asphalt pavements," says Geoweb distributor John Price, owner of Price and Co., Grand Rapids, Mich. "First of all, it supplies a maximum load distribution, or stress relief, from the asphalt surface to the subgrade with the least amount of thickness. Second, when used on soft subgrades, or with utilities, the Geoweb provides a stiffer base for the pavement."

When used with geotextiles, the cellular confinement system should turn a swampy strip of land into a stable, paved road. For example, Kent County used this technique on the largest project they have constructed — Reeds Lake Blvd. in East Grand Rapids. "In two areas, totaling 1,900 feet (48 m), this stretch of road is literally floating on 30 to 35 feet (9 to 11 m) of wet, soft muck," explains Byle.

"The road is built over swamp land and floods every spring. The water would rise (in the swamp) when the snow melted and it would go right over the road. We ended up closing the road every year at that time. Since we've done the project, though, we have not had to close the road. And the asphalt that was placed over it looks pretty good. There is no cracking on it."

Byle used geotextile, cellular confinement, plastic pipe and lightweight slag on this project. The biggest challenge for Kent County was constructing the culvert that provides the inlet to Reeds Lake. The culvert, located in an area where the muck was over 30 feet (9 m) deep, required three 36-inch (914-mm) smooth, lined plastic pipes. Removing the old culverts destroyed what there was of the existing road bed. The geotextiles were used to keep the culverts from sinking into the muck by tying them back into the existing road bed. The Geoweb cellular confinement system was used to spread the load and stiffen the road bed over the culverts.

Another project took place in Grand Rapids Township, on Leffingwell Ave., from Knapp Ave. to 3 Mile Road. Byle explains that 450 feet (137 m) of this road was in a muck swamp. The crews stripped off the original asphalt and found that the gravel underneath had been placed directly on the muck and at a thickness of less than 6 inches (150 mm). "Running the dozer literally caused eruptions of soft muck up through the gravel," says Byle. "So, we decided to modify our original design and place the cellular confinement system full width across the road bed to stiffen grade and spread the live load."

Crews lay down geotextile and Geoweb cellular confinement sections. Once the Geoweb is placed, slag aggregate is added to fill and cover the cells.

Kent County then took this same technique and applied it to one of the worst road conditions imaginable — a subgrade that, once cut, produced a 200-foot (61-m) long mud wave. The project, which was constructed on 108th Street in Caledonia Township, involved grading, stabilizing and paving the road. An initial solution was to stabilize the grade with geotextile, add underdrain, and cover with lightweight oversize slag. As the dozer operator spread the slag, however, the wave shifted and the underdrain came up through the slag behind the dozer.

Kent County decided to again use the Geoweb cellular confinement system — several 8-foot (2.4-m) by 20-foot (6.1-m) sections of it. The cells were filled and then covered with 2.25 feet (0.69 m) of #4 slag aggregate. Another layer of synthetic geotextile was laid, then covered with 6 inches (150 mm) of 22A slag aggregate. Asphalt was then placed on top of everything. When the project was completed, 2.5 feet (0.8 m) of materials had been added between the top of the subgrade and the bottom of the first lift of asphalt.

"The roadway shows no sign of distress at all," says Byle. "One fact to keep in mind with this type of construction, though, is that settlement is not eliminated. Settlement will occur; but hopefully it will be kept uniform — or, in other words, controlled settlement."

Price adds another suggestion: "The gravel being used needs to go between the bottom of the asphalt and the top of the Geoweb, so that when the asphalt gets hot, it doesn't melt over the Geoweb and get cut through — like a cookie cutter."

Making sure the application of the materials is done correctly is important. To assist with the planning of the project, Presto's Geosystems provides the SPECMaker™ Specification Development Tool. The SPECMaker software system develops complete material and construction specs for the Geoweb cellular confinement system. The program writes a project description based on what materials are available and what design has been provided. The software allows the user to specify the type of application — such as load support on a roadway — performance criteria, cell depth, cell type, anchoring system and infill material selection. ▮

GEOWEB®

BASE STABILIZATION SOLUTION FOR SOFT SUBGRADES

New Mexico

GEOWEB® SYSTEM STRENGTHENS HIGHWAY SUBGRADE

THE PROJECT

New Mexico's State Highway and Transportation Department (NMSH&TD) made news with innovative financing and a record setting pace in the construction of 118 miles (190 km) of four lane highway along a route in the northwestern part of the state formerly known as Highway 44. With a construction timetable of just two years, and with the construction season limited by cold Rocky Mountain winter weather, design engineers needed to solve construction challenges without delay.

THE GEOWEB SYSTEM MET SITE CHALLENGES

Presto Geosystems' **GEOWEB®** Cellular Confinement System was the perfect answer to a unique soft subgrade problem that threatened to stop paving operations with just a half mile of highway to complete and cold winter temperatures only months away.

While most of the highway, now known as U.S. 550, runs through remote and open country, the highway passes through the small town of Cuba. With time pressures, extremely soft subgrade conditions, relatively shallow underground utility lines running under the high-way, and a need to tie in to the existing elevations of sidewalks and parking lots of businesses adjacent to the highway, design engineers were faced with a challenge.

Dumping sand infill in the GEOWEB cellular confinement system.

The finished roadway is paved with asphalt.

GEOWEB®

BASE STABILIZATION SOLUTION FOR SOFT SUBGRADES

New Mexico

Expanding the GEOWEB sections over a geotextile.

Connecting adjacent GEOWEB sections prior to infill.

Infilling the GEOWEB sections with locally-available sand.

"The GEOWEB® System addressed the problem in far less time than any other alternative we had available. It went in quickly, without any complications for the construction crew. The GEOWEB® section will be included in a twenty year monitoring program we have scheduled for the entire U.S. 550 project."

- NMSH&TD Project Engineer Allan Whitesel

THE GEOWEB SOLUTION FOR STIFFENING SOFT SUBGRADES

Unable to proceed on a timely basis with any of the three conventional alternatives (*excavating deep deposits of saturated soils and replacing with more stable materials, thickening the base and subbase structural section to a higher elevation in order to bridge the soft subgrade, or strengthening the limited structural sections with conventional chemical stabilizers or other geotextile/geogrid type products*), they turned to the Presto **GEOWEB®** System. The **GEOWEB®** System is based on cellular confinement (geocell) technology with a proven record of providing an easily deployed stiffened flexural beam for bridging extremely soft subgrade conditions.

USING LOW-COST ONSITE INFILL SAVES COST

Because **GEOWEB®** cells are functional with either clean sand or aggregate infill materials, designers selected a locally available low cost source of free-draining sand for placement within the six inch deep cell structure. The system was deployed over a geotextile to protect the sand infill from contamination.

Working half the width of the highway at a time, crews rapidly installed the **GEOWEB®** for the final half mile. The project required 189,000 sq ft of product. With the firm **GEOWEB®** working platform in place, base construction and asphalt paving were quickly completed, facilitating a timely grand opening for the 118 mile highway improvement project.

PRESTO GEOSYSTEMS

Call 800-548-3424 for more information.

www.prestogeo.com

Ropers Gate, Tunbridge Wells – Example of permeable paved road with base infiltration, surface water storage within the sub-base and the use of the Geo-web system

LOAD SUPPORT SYSTEM GENUINE GEOWEB®

Project description

Project: New Salt farm

Projectnumber: E-2016-077

Date: 29.04.2016

Project description:

A new access road needs to be constructed on soft soil. A CBR value of 3.0% is assumed for the existing subgrade. Geoweb® geocells offer a cost-effective solution to improve the bearing capacity and in-service performance of those structures due to its unique load transfer mechanism. At loading, hoop stresses within the cell walls and passive earth resistance in the adjacent cells are mobilised, which restricts the horizontal deformation of the infill material. As a result the stiffness of the fill material is increased and due to this the vertical stresses and the settlements are reduced.

Given Documents/Parameters:

Loading parameters (axle load; load area; traffic passes):	given by client (RFPE)
Subgrade parameters (CBR; stiffness; angle of friction):	given by client (RFPE)
Layer parameters (angle of friction, stiffness):	assumed by editor

Soil Parameters:

Soil mechanical parameters of the subgrade material were assumed by the client. Preliminary design is therefore carried out for a CBR value 3 percent, which leads to a net bearing capacity of approx. 220,29 kN/m² (depending on the correlation between undrained shear strength and CBR). Soil mechanical parameters of single pavement layers were not provided by the client and were assumed on basis of experience of Soiltec GmbH. Soil parmeters are assumed to be similar in the entire construction area. If in-situ soil parameters are different than assumed the pavement design can differ/should be modified.

Load Parameters:

According to the given information the area is loaded by trucks and pilling rick.
A tyre load of 50 kN is assumed for the design. The tyre load is distributed over a contact area of 0,08 m², which leads to a static contact pressure of 625 kN/m². The number of passes is considered in the serviceability analysis by an increased static reference load, which is calculated due to a load increase factor. Design is carried out for 100000 axle crossings. If load parameters are different than assumed the pavement design can differ/should be modified.

Geoweb® Parameters:

 The recommendations in this Evaluation are based on the specific characteristics, structural values and specifications of Geoweb® manufactured by Presto Products.

Limited State Analysis:

The verification of the bearing capacity is based on the German version of the EuroCode 7, DIN EN 1997-1:2004 + AC:2009, in conjunction with the National Annex DIN EN 1997-1/NA:2010-12 and the supplementary provisions of German Standard DIN 1054. The calculation is based on the partial safety concept. In the partial safety concept the characteristic actions (loads) are increased by multiplying them with prescribed partial factors to produce design actions (loads). The characteristic resistances or soil properties are decreased by dividing them with prescribed partial factors to produce design resistances or soil properties. If the design resistance or soil properties are equal or higher than the design actions an adequate margin of safety against collapse can be assumed. By defining the degree of utilization μ as the quotient of design actions to design resistance or soil properties, a value of μ less than one means a safe design.
For the verification of the safety of the Geoweb® access road, limited state analyses were conducted according to DIN 4017 (proof of sufficient safety against shear failure in limit state GEO-2). Limit state GEO-2 implies the design approach 2* of DIN EN 1997-1:2004 + AC:2009. Herein the actions are increased and the resistances are reduced.
Shearing occurs if the applied pressure on the subgrade soil is higher than the bearing capacity of the subgrade material. The bearing capacity of the subgrade soil is determined on basis of the undrained shear strength (qult. = 5.14 x cu), the existing pressure on the subgrade is calculated according to the state of the art by the load distribution through the Geoweb® stabilized pavement construction, whereas the load distribution angle was determined depending on the infill material of the cells by several large scale and in-situ tests which have been carried out.

Servicability Analysis:

An analytical model on basis of state of the art was used to estimate the settlement of the pavement construction. In the analytical model, the settlements within the Geoweb® layer and beneath the Geoweb® layer are calculated separately and summarized to the overall settlements afterwards. The improvement effect due to the Geoweb® layer is taken into account by an equivalent load area beneath the Geoweb® layer. Settlements are calculated on basis of German Standard DIN 4019. It is assumed that the modified model is adequate for the estimation of settlements.
The pavement design is carried out for maximum allowable settlements of 15 mm.

LOAD SUPPORT SYSTEM
GENUINE GEOWEB®

Project: New Salt farm

Projectnumber: E-2016-077

Date: 29.04.2016

Input parameters

loading conditions:		
Loading type:	SLW 30 - Wheel load 50 kN	DIN EN 1072
Tire load (static) [kN]:	50,00	wheel load from trucks
Axle crossings N [-]:	100.000	
Loading area [m²]	0,080	
Contact pressure (static) [kN/m²]	625,00	
Contact pressure (dynamic) [kN/m²]:	1044,43	
Load increasing factor [-]:	1,67	
Geoweb and infill material:		
Description:	G3V306	
Cell height h [m]:	0,150	
Cell diameter d [mm]:	300,00	
Infill material description:	Gravel 0-32mm	
angle of internal friction [°]	35,00	
infill unit weight [kN/m³]	18,00	
friction coefficient between infill and geocell material [-]:	0,80	
subgrade soil parameters:		
Description:	Soft Subgrade	
Excavation depth before Geocell installation [m]:	0,00	
Unit weight of subgrade material [kN/m³]:	0,00	
angle of internal friction [°]:	0,00	
Undrained cohesion of subgrade [kN/m²]:	0	
CBR-value of subgrade [%]:	3,0	assumed
uniaxial compression stiffness [kN/m²]:	0	
calculated uniaxial compression stiffness [kN/m²]:	6000	based on correlations between CBR and stiffness
Load case according to partial safety concept:		
Load case:	LF 1	*LF1 - Permanent constructions*
Partial safety factor reaction forces [-]:	1,50	*LF2 - temporary constructions*
Partial safety factor restoring forces [-]:	1,40	BS - without factors
settlement reduction factor [-]:	0,20	
allowable settlements [mm]	15,00	according to client

Pavement design with Geoweb® **:

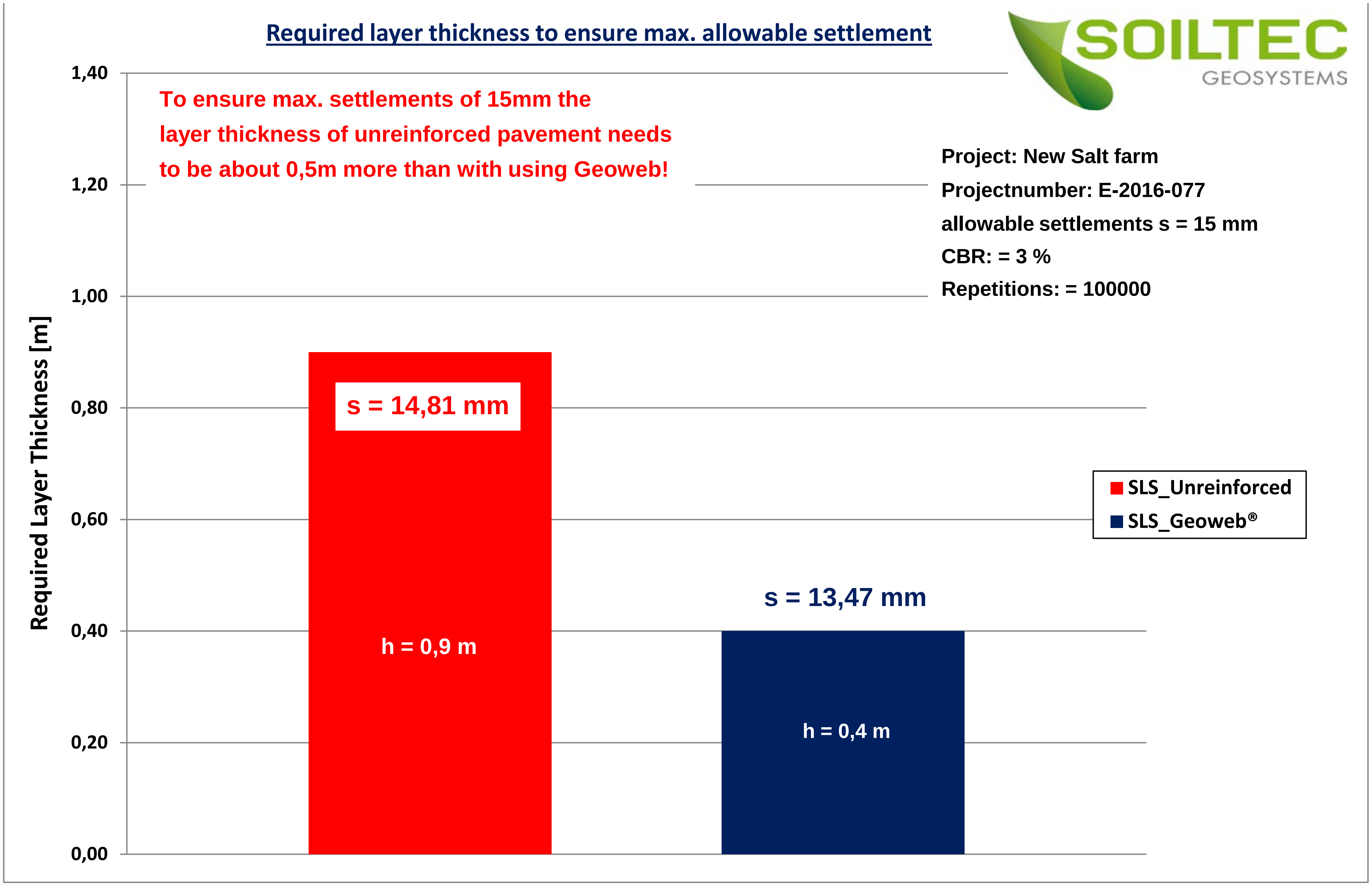
soil layers	angle of friction [°]	unit weight [kN/m³]	layer thickness [m]	stiffness E_s [kN/m²]
Asphalt Layer	45,000	19,000	0,100	120.000
G3V306+Gravel 0/32mm	45,000	18,000	0,150	120.000
Overtopping of Geoweb	35,000	18,000	0,150	120.000
nonwoven geotextile 70 kN/m				

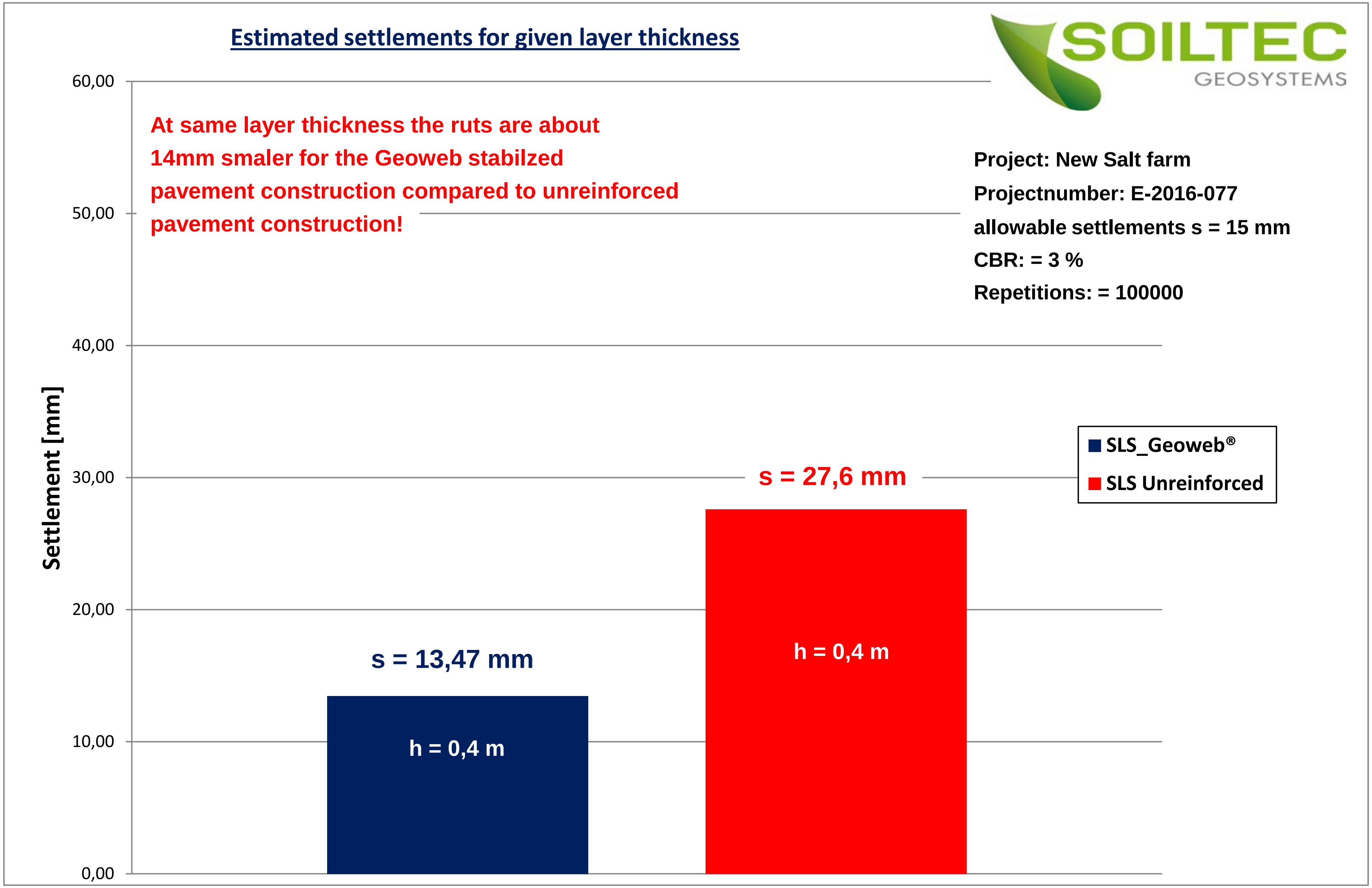
** designed pavement layout considering the specific subgrade strenght and traffic passes for this project.

Pavement design without Geoweb® if available: Required layer thickness to have same settlements as with Geoweb**

soil layers	angle of friction [°]	unit weight [kN/m³]	layer thickness [m]	stiffness E_s [kN/m²]
Asphalt Layer	45,000	19,000	0,100	120.000
Complete Gravel Layer	35,000	18,000	0,800	80.000

** designed pavement layout considering the specific subgrade strenght and traffic passes for this project.


Required layer thickness to ensure max. allowable settlement
SOILTEC
GEOSYSTEMS
To ensure max. settlements of 15mm the
layer thickness of unreinforced pavement needs
to be about 0,5m more than with using Geoweb!
Project: New Salt farm
Projectnumber: E-2016-077
allowable settlements s = 15 mm
CBR: = 3 %
Repetitions: = 100000
Required Layer Thickness [m]
1,40
1,20
1,00
0,80
0,60
0,40
0,20
0,00
s = 14,81 mm
h = 0,9 m
s = 13,47 mm
h = 0,4 m
SLS_Unreinforced
SLS_Geoweb®


Estimated settlements for given layer thickness
SOILTEC
GEOSYSTEMS
At same layer thickness the ruts are about
14mm smaler for the Geoweb stabilzed
pavement construction compared to unreinforced
pavement construction!
Project: New Salt farm
Projectnumber: E-2016-077
allowable settlements s = 15 mm
CBR: = 3 %
Repetitions: = 100000
Settlement [mm]
60,00
50,00
40,00
30,00
20,00
10,00
0,00
SLS_Geoweb®
SLS Unreinforced
s = 27,6 mm
s = 13,47 mm
h = 0,4 m
h = 0,4 m

Becky Warrener

From: Andrew Picton
Sent: 05 October 2016 16:09
To: 'Ken Argent'
Cc: Kevin Macknay; Julian Turner; Sarah Poulter; Jackson, Adrian; 'Dinny Shaw'
Subject: RE: New Salts Farm - Road Construction & Levels

Ken,

Apologies for the delay in responding.

It is the intension that the services (where ever possible) will be kept out of the carriageway. Generally, they will be located beneath the footway or within a service strip running parallel to the road. Where perpendicular service crossing occur, ducting will be provided. The footways will be of more standard from of impermeable construction which will facilitate an easier form of reinstatement.

We have spoken to the suppliers of the Geo-Grid system and it is possible to dig a trench through this type of road although it will clearly require additional supervision to ensure that a new strip of geo-grid is correctly installed when the road is reinstated.

The roads within the site are to remain private so we do have a little more flexibility on the location of the services and the manner in which the repair/installation of the services are carried out.

With a permeable road construction there will be no road gullies or surface water drainage system within the road which will also help to avoid digging up the road.

Regards

From: Ken Argent [mailto:ken.argent@adur-worthing.gov.uk]
Sent: 29 June 2016 08:55
To: Andrew Picton <ajp@tullydeath.com>
Cc: Kevin Macknay <kevin.macknay@westsussex.gov.uk>; Julian Turner <jct@tullydeath.com>; Sarah Poulter <sarah.poulter@hyde-housing.co.uk>; Jackson, Adrian <adrian.jackson@environment-agency.gov.uk>
Subject: Re: New Salts Farm - Road Construction & Levels

Andrew.

i promised to give this product a bit more though after our last meeting, i have copied the EA into my comments which follow:

The system clearly is suitable for weak ground, it also clearly acts as a permeable layer, but the literature does not state that the subgrade can be permanently saturated, as a result high ground water periods during the wet months, unless you utilise the ATRA anchor and tendon.(page 5)

But

Whilst you will strive to install all the necessary services at the time of construction, we all know what utility providers are like, they seem to be attracted to new pristine surfaces, - what effect will trench or localized excavations have on the geogrid integrity, once the tendons and anchors are damaged or the geogrid is cut out?.

Ken Argent | Engineer | Engineering Team | Adur & Worthing Councils
01903221374

On 25 April 2016 at 15:34, Andrew Picton <ajp@tullydeath.com> wrote:

Kevin, Ken

Further to our meeting last week I have attached some product literature for the geogrid system we are looking to use on this site. There are a number of suppliers of the Geoweb system but they are all fundamentally the same. We have successfully used this system on a number of private estate roads where we were required to provide a no-dig road, built directly onto the top soil, or where the ground conditions were very poor (CBR values were non-existent). Generally we have used this system on a permeable road, with base infiltration and attenuation provided within the structure of the road, which is similar to what is proposed for this development.

As typical with estate road design, the construction phase is the more onerous loading situation. Usually a sacrificial impermeable DBM layer sits on top of the Geogrid system during the construction phase which is either left in place with holes punctured into it to allow percolation, or is removed when the permeable surfacing is installed.

We are working up some site specific details and will send them across to you shortly.

I have also attached a copy of the New Salts Farm topographical survey for you information. It was mentioned at our meeting that Phase 1 was located in a low part of the site. Upon closer inspection of the levels, the majority of Phase 1 is relatively high (typically 1.8m AOD) with levels falling to the north. The ditch adjacent to the northern boundary of Phase 1 is the local low point where levels beyond the ditch rise (to the north) up to 1.6.-1.8m AOD before falling again towards the northern boundary.

Kind regards

Andrew Picton | Associate Director

Phone 01342 828000 | Mobile 07739 265802 | tullydeath.com
Tully De'Ath Consultants, Sheridan House, Hartfield Road, Forest Row, East Sussex, RH18 5EA

Appendix P – Green/Blue Roof Details

Examples and Details of Green/Blue Roofs for New Salts Farm

Project Number – 11649
New Salts Farm Road Land - Shoreham

All the houses on the New Salts Farm Development will incorporate Green Roofs with water attenuation (Blue Roof) below. It is well documented on the benefits of Green Roofs in terms of reducing the volume of surface water run-off. The introduction of the Blue Roof system below the green roof will enable further controlled outflows from the roof.

Typically, the green/biodiverse roof area has an undulating, 50-150mm depth of growing medium, which could include a sedum mix, or a local wildflower seed mix and/or selected plug plants; sand, pebble, stone, log piles and animal, insect, bird habitats. The Blue element consists of thin plastic storage crates with an integral flow control device. The Blue Roof system does not store any permanent volumes of water for more than a 12/24-hour period. It will constantly flow/empty and will only restrict the flow when the appropriate design storm event hits.

The size of the crates are designed to attenuate surface water run-off (with a restricted outflow) for all events up to the 1 in 100-year storm with an additional 30% allowance for climate change.

The restricted outflow will discharge to RWP's, external to the building.

The typical roof details provided refer to the ABG system however there are other similar Green/Blue Roof systems available, such as Polypipe and Alumsac. ABG provide warranties and are accepted by the NHBC (see attached documentation).

The maintenance of this type of roof will be dependent on the system used. For the ABG system, a 6-monthly maintenance inspection is required (see attached ABG Green Roof Maintenance Document). The Blue Roof chambers need to be visually checked and the RWP outlet separately cleaned as appropriate. If required, the Blue Roof chamber's internal filter geotextile board can be changed. The Polypipe systems claim to be maintenance free although as a minimum it is recommended that the outlets and flow control devices are inspected at least once a year.

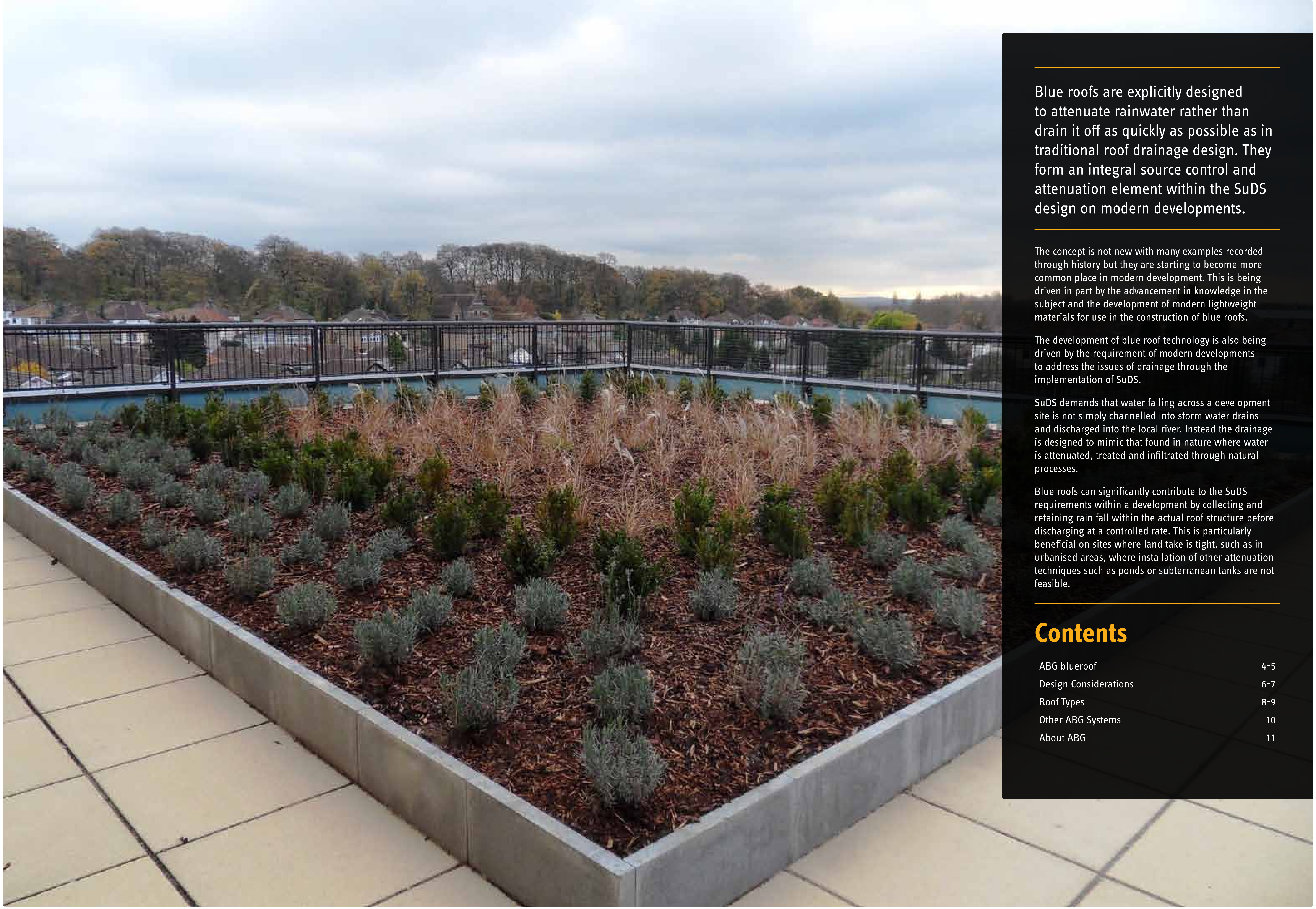
The current intension is that Hyde, as Estate Manager, will manage and maintain the Blue Roof system.

ADDRESS	TEL	FAX	EMAIL
Sheridan House, Hartfield Road, Forest Row, East Sussex, RH18 5EA	01342 828000	01342 828001	info@tullydeath.com

blueroof

A guide to the selection and specification of ABG blue roof system for the attenuation of storm water.

abg | creative geosynthetic engineering



Blue roofs are explicitly designed to attenuate rainwater rather than drain it off as quickly as possible as in traditional roof drainage design. They form an integral source control and attenuation element within the SuDS design on modern developments.

The concept is not new with many examples recorded through history but they are starting to become more common place in modern development. This is being driven in part by the advancement in knowledge in the subject and the development of modern lightweight materials for use in the construction of blue roofs.

The development of blue roof technology is also being driven by the requirement of modern developments to address the issues of drainage through the implementation of SuDS.

SuDS demands that water falling across a development site is not simply channelled into storm water drains and discharged into the local river. Instead the drainage is designed to mimic that found in nature where water is attenuated, treated and infiltrated through natural processes.

Blue roofs can significantly contribute to the SuDS requirements within a development by collecting and retaining rain fall within the actual roof structure before discharging at a controlled rate. This is particularly beneficial on sites where land take is tight, such as in urbanised areas, where installation of other attenuation techniques such as ponds or subterranean tanks are not feasible.

Contents

ABG blueroof

ABG **blueroof** provides attenuation capacity within the green roof or podium deck construction of a development. Utilising this space in this way means that the attenuation capacity required to meet SuDS best practice can be met without the requirement for land consuming ponds and retention basins or the challenges of constructing large subterranean geocellular storage tanks.

blueroof comprises a combined drainage and attenuation void within the roof structure and a roof outlet system designed to release the attenuated water at a controlled discharge rate as permitted in the planning consent of the site.

Designing a green roof in this way allows storage capacities suitable for up to a one in a hundred year storm event, plus an allowance (typically 30%) for the effects of climate change, to be achieved.

This stored water, as with a 'traditional' storage system, can be released at a controlled rate or even used as grey water or irrigation for the vegetation across the development.

The ABG **blueroof** System consists of two key components:

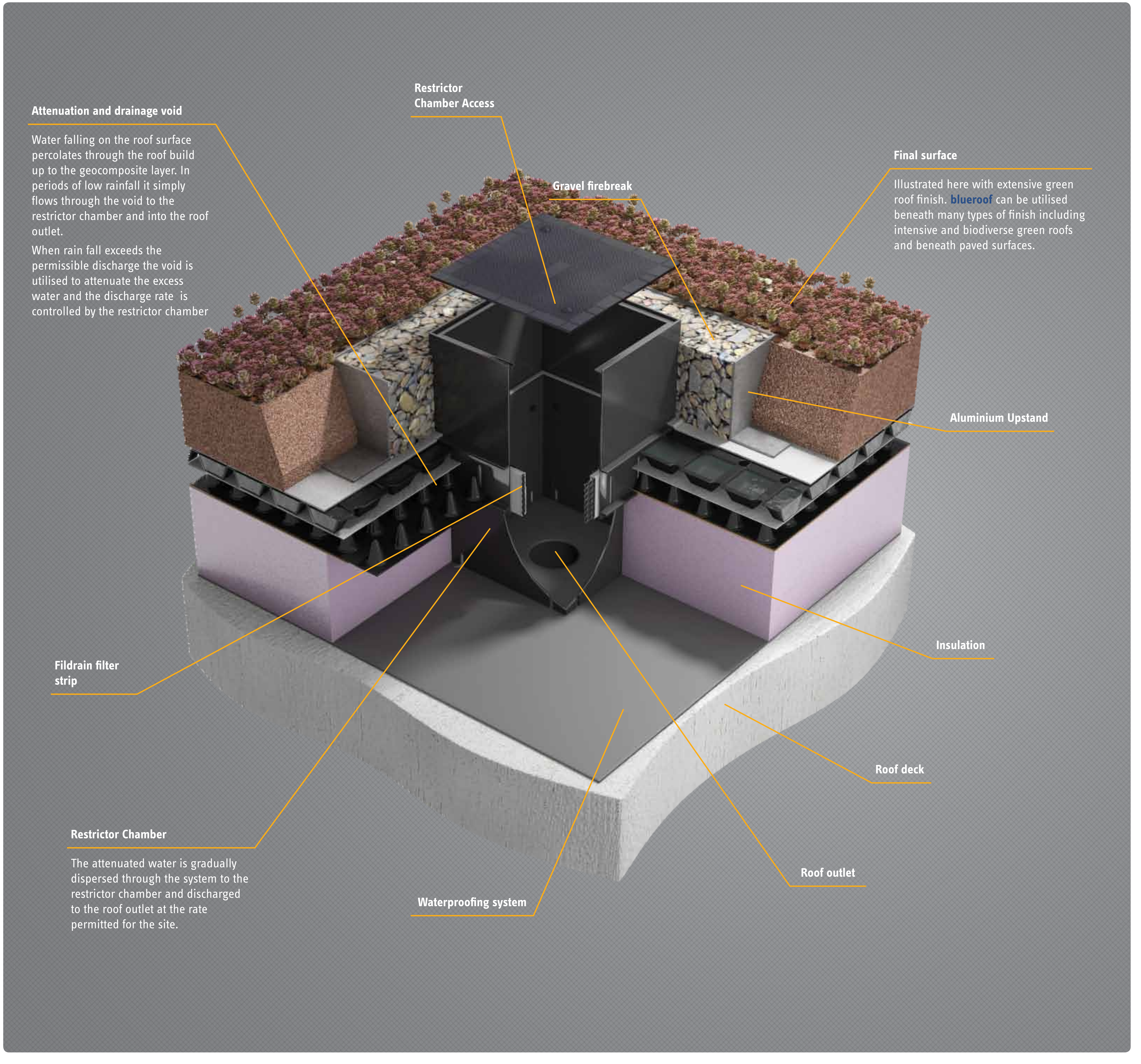
- A drainage geocomposite system with integral filter geotextiles and a series of restrictor chambers. Excess water not absorbed by the vegetation, filters through the green roof and builds up in to the drainage void formed by the geocomposite layers below.
- This water is gradually dispersed through the system to the restrictor chamber and discharged to the roof outlet at the rate permitted for the site.

The storm water attenuation requirements are met within the roof construction, therefore the need for underground storage can be eliminated. The benefits to the overall project include the removal of the excavated material, disruption on site, and the time and cost of installing an underground tank.

Placing the storage within the footprint of the building also has advantages in heavily urbanised developments where external space is at a premium and on site working space and materials storage is limited. This reduction in material movements also helps reduce the carbon footprint of the project.

blueroof is suitable for:

- Supermarkets
- Distribution centres
- Schools and colleges
- Shopping centres
- Underground car parks
- Housing
- Flats
- Office blocks



Design Considerations

Design Factors

As part of the design process ABG will develop response calculations to model the behavior of the roof during storm events. The information required is usually contained within the surface water run-off assessment for the specific site.

The modelling looks at a number of key factors including

- Required rate of discharge.
- Attenuation volume requirement.
- Time to completely discharge attenuated water from the roof structure.
- Roof type.

Rainfall depths for the specific site are calculated according to location, duration and return period (the number of times in set period a storm of that magnitude is likely to occur; 1 in 30 years and 1 in 100 years storms are usually considered). An allowance is also made for future climate change.

Rainfall and run-off should be considered simultaneously to give an actual representation of the **blueroof** behaviour under storm conditions.

Design Capacity

Should attenuation reach its maximum level the restrictor chamber has a built in safety mechanism designed to release excess water into the drainage system. Design capacity will always come with a factor of safety allowing for additional capacity.

In reality, provided the **blueroof** is designed and maintained properly, its designed storage capacity will never be exceeded.

Outlet Design

Traditional roof design tends to have a conservative approach when designing the rainwater outlets with usually more outlets installed than actually required. When designing a blue roof the restrictor chambers are an integral component in controlling the discharge of water from the roof and as such the number required is calculated exactly. Typically this may mean that less outlets are required, less outlets means less penetrations, less detailing and greatly reduces the potential of leaks occurring. The reduction in outlets also has a positive impact on both the construction time, costs and service risers running through the building meaning the construction saves both time and money.

Water Quality

Using the **blueroof** system has a positive impact on the quality of the water discharged. Before the water reaches the roof outlet it has already passed through several processes that remove particulates and pollutants including vegetation and growing medium (if the roof is green) and more importantly through at least two, in a basic system, layers of non-woven, needle punched geotextile whose filtration properties are well documented. The water is treated to such a degree that it reaches the level required in treatment train stage one allowing the water to be released from the roof directly into the river system

In a truly holistic design consideration should be given to using the attenuated water for secondary uses such as the irrigation of gardens and washing paths etc. The water could also be considered for grey water reuse applications although it may need to undergo a further treatment stage in order to do so.

Structural Considerations

The introduction of a **blueroof** may have loading implications for the structure of the building. It is vital to consult a structural engineer at an early stage especially when designing for a SuDS solution where water will be stored within the roof structure. This will enable you to determine any constraints you may be under, although this is not as onerous as may be expected.

Traditional structural loadings in roof design take into account the dead weight of the roof structure, the materials used to construct it, plus an allowance for load applied by snow falling on the roof.

blueroof stores collected water across the entire area of the roof at a shallow depth, typically less than 100mm. At full capacity this would exert a maximum additional load of $1.0kN/m^2$.

In reality it is exceedingly unlikely that the roof will ever reach full capacity as it will start to drain as a soon as it starts to rain and will continually drain throughout the storm event at the rate determined by the restrictor outlet.

When taking into account that there is no screed required to achieve a fall on the roof and construction tolerances the additional design allowed for load is, in fact, usually negligible.

ABG Technical Department are able to advise on the loadings the roof will generate when fully charged.

Waterproofing Design

A key element of any roofing system is the waterproofing. **blueroof** is compatible with all modern waterproofing materials (ABG recommend monolithic bonded systems). The selection of which waterproofing type is down to the type of roof construction and, to a degree, personal preference. As a concept **blueroof** is compatible with both warm and inverted roof constructions.

Once installed it is recommended that the waterproofing layer be electronically tested for integrity before being covered installation of the blueroof components commences.

Care should be taken during installation of subsequent layers however once the insulation is installed the waterproofing system is covered and protected from damage from further works during normal operations.

As with other roof types the waterproofing should be detailed to a height of 150mm higher than the final fill level.

ABG work with leading manufacturers and installers of waterproofing systems and can offer project specific advice and guidance to ensure the optimum solution is selected.

Access and Maintenance

The British Standards Institution state that all new builds must provide access to the roof area to enable a minimum of two inspections per year. In achieving this compliance to working at height regulations must be considered. If a building is of a height which can cause an injury from a fall, including roofs under 2m, then edge protection is required.

The level of maintenance required is dependent on the final finish. Paved podium decks and extensive green roofs are relatively low maintenance where as intensive green roofs require maintenance like any garden.

Specific attenuation should be given to the **blueroof** elements such as the outlets which should be checked a minimum of twice annually.

As with any green roof the design should allow for the safe removal of materials from the roof

Thermal Performance

blueroof needs to meet the building regulations required to achieve the thermal performance. At the moment, as with green roofs, the **blueroof** build up cannot be considered as part of the roof build up when calculating thermal performance so insulation specification must be done as per a traditional roof design.

It is recommended that the insulation material be extruded polystyrene (XPS) and not expanded polystyrene (EPS). EPS in contact with water degrades which will result in the roof losing thermal performance ultimately leading to the requirement for an expensive reroofing operation.

However, research shows that the introduction of layers of drainage, growing media and vegetation have an impact on the thermal performance and can offer additional benefits on the development including cost benefits and reducing the carbon footprint.

Geography

Geographical location and orientation are an important part of designing a **blueroof**. Which area of the country, the amount of average rainfall in that area and the prevalent wind direction all affect the design and must be considered.

When using a vegetated finish the geographic location impacts the species selection with many species suitable for green roofs being specific to a region.

Final Finishing

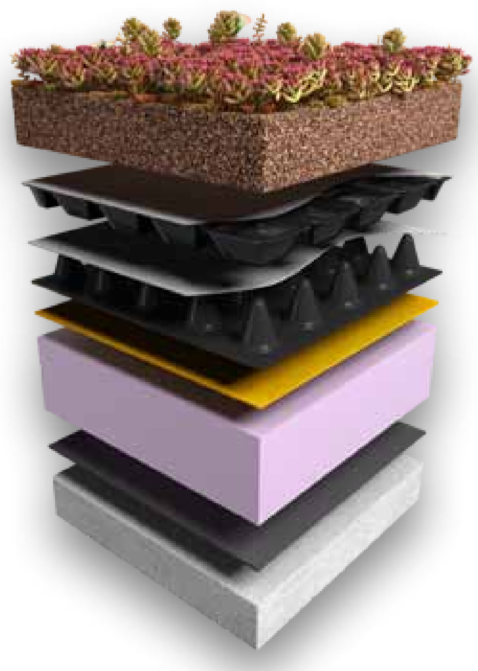
blueroof can be designed beneath all green roof types including extensive, intensive and biodiverse (brown). It is also suitable for use beneath paved or trafficked areas such as frequently used on podium decks. Suitable surfaces include permeable block paving, rubberised asphalt, ballasted etc. **blueroof** is also suitable for use with photovoltaic cells (PV).

The options are endless and comes down to the clients requirement for the final finish of the roof.

ABG Technical department are able to advise and assist with project specific design guidance to help meet the clients requirements.

Inverted Roof

In inverted roofs two layers of composite are used above the XPS insulation layer overlaid with a slimline separation membrane. In conjunction with a restrictor outlet chamber the two layers of composite provide a combined drainage and attenuation function across the roof area.



Podium Deck

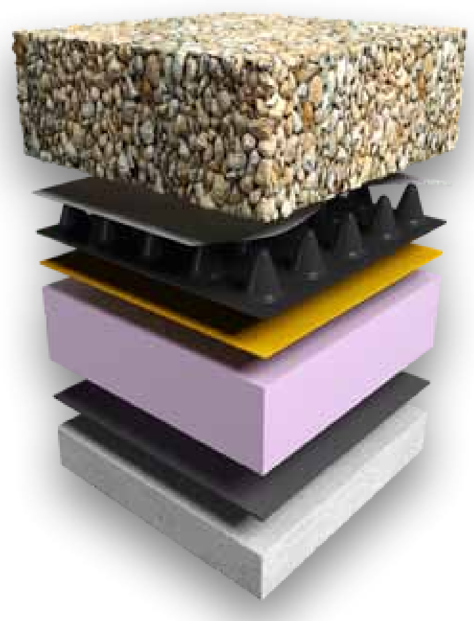
In podium deck construction typically the system utilises two layers of Deckdrain within the system. The upper layer forms a free flowing layer addressing drainage requirements during low flow whilst the layer beneath providing attenuation capacity during and after storm events.

Warm Roof

In warm roof construction the composites behave in much the same way as within the inverted roof construction with the whilst providing protection to the waterproofing system laid over the insulation.

Ballasted Roof

In ballasted roof construction the void within the ballast provides additional attenuation capacity therefore negating the requirement for a second layer of composite. The composite provides the main attenuation void across the roof area.



Roof Types

Like green roofs blue roofs are suitable for use on a wide range of substrates and are compatible with most modern waterproofing systems.

They are also flexible when it comes the roof build up being equally as effective on an uninsulated podium construction as they are within a warm or inverted roof construction

The illustrations on the left represent some of the more common roof build-ups with which the blueroof system can be used but they do not define its entire extent. The highly modular nature of the blueroof components mean that a system can be designed for most flat roofs and podium decks.

specific advice and guidance on individual project requirements is available through the ABG Technical Team who are happy to review your scheme and determine suitability.

SuDSpave

The SuDSpave system comprises complementary components that create an integrated porous paving system to effectively manage the safe collection, treatment, management and dispersal of surface water.

SuDSpave is configurable to individual project requirements and offers a range of surface solutions to meet the aesthetic and performance requirements. In addition, a range of geogrids and geocells can minimise construction depth whilst meeting the structural requirements. In addition high performance geotextiles help treat collected water to meet quality expectations. Finally geocomposites can allow the formation of a free storage void across the paved area to attenuate surface water during storm events.

Drainage

ABG drainage geocomposites offer high performance cost effective alternatives to traditional stone groundwater drainage solutions and have been used extensively in a wide range of civil engineering, environmental and building drainage applications.

Drainage geocomposites offer very high flow capacity, many times that of traditional crushed stone (specific data is available), this is achieved through the unique open structure created by the cuspated core construction which allows unhindered water flow through the system.

Geotextiles ensure that fines do not enter the flow void minimising the occurrence of blockages and allowing continuity of flow through the whole life of the installation

Webwall Retaining Walls

Webwall is a geosynthetic system designed for the construction of flexible retaining walls. It uses a geocellular mattress which is laid in layers with each expanded and filled with site won materials to form a structure with a vegetated face.

The Webwall system offers a solution in many SuDS applications with its primary use being in the construction of steep embankments on SuDS structures such as swales, channels and attenuation basins and ponds. Constructing steeper embankments minimises the land take of the structure freeing up more land for development.

As with all ABG systems a full design service is offered through our in-house team of chartered civil engineers.

Erosion control systems

ABG offer a broad range of erosion control products that includes biodegradable and non-biodegradable erosion control mats, They can help with the surface protection of many elements within the SuDS scheme including swales, channels, ponds and attenuation basins

Silt laden run-off from exposed soil slopes is a major concern for the Environment Agency who consider it a pollutant. Erosion control systems help to ensure the environment is protected throughout the life of the project from construction to establishment of the vegetation.

As with all ABG products design advice on which materials are appropriate for your specific requirement and their specification is available from our technical department.

About ABG

ABG is a market leader in the design, development, manufacture and technical support of high performance geosynthetic systems for use in a wide range of civil engineering, environmental and sustainable building projects.

Formed in 1988, based in Meltham, in the heart of the Pennines, ABG have developed an excellent reputation for developing quality products and delivering outstanding service. The ability for rapid product development ensures that the most innovative, up to date and cost effective solution can be found for many engineering problems.

ABG's involvement in roof drainage goes back over twenty five years and we have a complete range of products developed specifically for use in this technically demanding application.

Technical support is provided by our trained and experienced staff, many of whom are Chartered Civil Engineers. This extensive support extends to full design, design validation, feasibility studies, cost advice and advice on meeting regulatory requirements.

Part of this technical support includes developing and driving knowledge within our active markets including working with both international and local regulatory bodies on developing guidance and best practice in the use of innovative geosynthetics to solve complex engineering issues.

For further information or to discuss your project specific requirements contact ABG:

t 01484 852096
e blueroof@abgltd.com
www.abgltd.com

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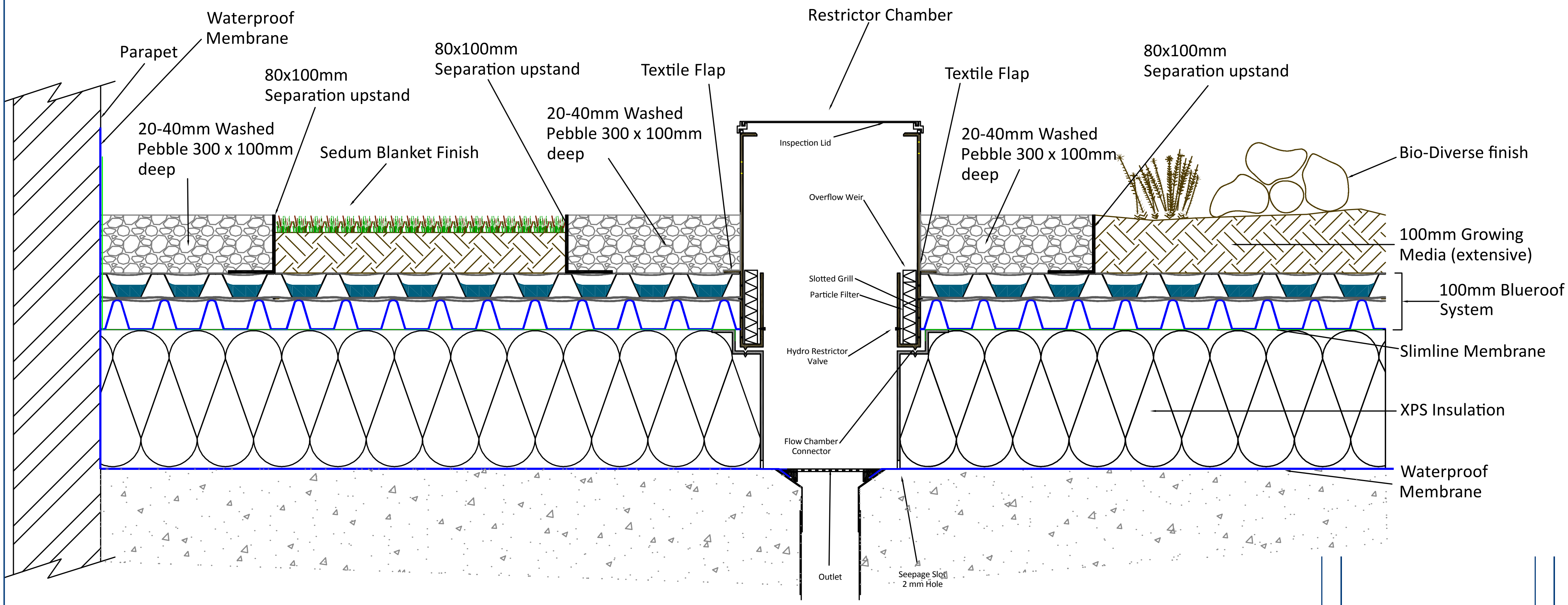

Rev:	Comments:	By:	Date:

Project: BLUE ROOF

Title: BR01

Drawn by: GJB	Date: 29/10/15	
Scale: NTS	Drawing Ref: 01	Rev: XX

abg creative geosynthetic engineering

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Retaining Walls| Erosion Control| Landfill
Tunnels | Green Roofs | Landscaping | Highways

ABG blueroof System 10/20/30 Year Warranty

Warranty Serial Number:			
Contract/Building Name:			
Building Owner:			
Building Address:			
	Roof/Podium Area 1	**Roof/Podium Area 2**	**Roof/Podium Area 3**
Waterproofing Membrane System Type & Install Team Name:			
ABG blueroof System Type & ABG Approved Install Team Name:			
Final Surface FinishType & Install Team Name:			

Date of Substantial Completion and Final Inspection:

Details of the ABG blueroof System:

ABG Ltd ('ABG'), warrants to the owner of the building described above ("Owner"), that subject to the terms, conditions, and limitations stated herein, ABG will warranty the performance of the "ABG **blueroof** System" for the Warranty period commencing with the date of substantial completion of the installation of the ABG **blueroof** System.

This warranty applies to ABG **blueroof** systems for which:

1. All work has been completed by ABG, or it's approved installer, for supply & installation of the ABG **blueroof** System.
2. The underlying waterproofing system has been tested for water-tightness using electronic leak detection, and inspected and certified by the manufacturer or installer.
3. The waterproofing has been maintained in a protected condition between the time that the waterproofing has been installed and the ABG **blueroof** System has been installed.
4. The installation of the ABG **blueroof** System has been completed using an approved ABG installer and ABG supplied materials.
5. The Owner maintains the ABG **blueroof** System in line with the ABG Operating & Maintenance (O&M) procedures provided on practical completion of the installation which will include regular scheduled inspections of the roof or podium area (including visual inspections of the ABG **blueroof** restrictor chamber boxes), and appropriate care and maintenance of the final surface finishes for the duration of the warranty.
6. The Owner of the building provides safe access to all roof or podium areas where the ABG **blueroof** Systems have been installed for the duration of the warranty and where appropriate has maintained safety systems incorporated into the building.
7. If the final surface finishes (including any vegetated/green roof, hard or soft landscaping, pavers, or ballasted finishes) have been supplied & installed by an ABG approved installer, then this warranty will apply to this additional buildup above the ABG **blueroof** System.

Where ABG's approved installer has installed a vegetated/green roof final surface finish, ABG will warrant the vegetated cover performance to achieve and maintain a foliage coverage rate of seventy (70) percent for the duration of this warranty. The appropriate level of care, weeding, fertilization, and irrigation must be provided by the Owner, as required within the ABG O&M procedures.

TERMS, CONDITIONS, LIMITATIONS

1. The Owner shall notify ABG on the first business day immediately following the discovery of a failure in the ABG **blueroof** System, and confirm in writing within 10 working days.
2. If, on inspection by ABG, ABG determines that the identified failure in the ABG **blueroof** System is caused by a defect then ABG shall affect repairs. The decision of ABG with respect to repairs shall be final and binding.
3. This warranty does not extend to conditions caused by, and ABG shall not be responsible for any damage caused by:

(a) Any act of negligence, accident, or misuse including, but not limited to, lack of maintenance, damage by other persons or trades, vandalism, falling objects, civil disobedience, or act of war, or:
(b) Vehicular, Pedestrian travel or recreational use, except in areas specifically designated for these purposes, or:
(c) Damage by a natural disaster including, but not limited to, earthquake, lightning, fire, hail, high winds, hurricane, tornado, flood, erosion, drought, acid rain, thermal shock or other acts of God, or:
(d) Damage caused by animals, birds, or insect or disease infestation, or:

(e) Other building components, including cracking, building movement, settlement, deflection of roof deck, deterioration of walls, movement of metal work, water entry other than the roof, and defects in the materials used as a base under the roof, or:
(f) Service to or maintenance of any roof top equipment or traffic of any nature on the roof except in designated areas, or:
(g) Removal of any portion of the ABG **blueroof** System, including any of the final surface finishes above the ABG **blueroof**, and disturbance of the ABG **blueroof** restrictor chamber boxes , without prior written approval by ABG, or:
(h) Chemical attack, including but not limited to petroleum-based products, solvents, contaminants, chemical waste, exhaust or heat generated by mechanical units, deicing materials, fertilizers, herbicides and pesticides that are not approved by ABG, and alike, onto the final finished surface level, ABG **blueroof** system, and waterproofing system, or:
(i) A proscribed activity, including the failure to comply with Operating & Maintenance Plan, and any construction or installation subsequent to the installation of the ABG **blueroof** system that has not been authorised in writing by ABG. The Owner must promptly notify ABG in writing of any proposed alterations, additions or changes of any kind that will affect the ABG **blueroof** System, or:
(j) Alterations or repairs made on or through the completed ABG **blueroof** System, or objects such as but not limited to fixtures, equipment, or structures are placed on or attached to the completed ABG **blueroof** System or the final surface finishes, without first obtaining written authorisation from ABG, or:
(k) Failure by the Owner or their lessee to use reasonable care in maintaining the roof or podium area as described in the building or sites Operating & Maintenance plan, or:
(l) Poor irrigation water quality, in particular reference to where vegetated/green roof final surface finishes are installed by the ABG nominated installer, or another contractor, or:
(m) Deficient design applied to the ABG **blueroof** System such as contact with incompatible materials and/or substrates, installation next to highly reflective surfaces without an irrigation system, exposure to heat below roof deck, such as from steam or hot water pipes, insufficient drainage design, or:
(n) Any change of use of the roof or podium area, associated loading parameters, or changes in the final surface finishes, not discussed and approved by ABG prior to the commencement of these changes, or:
(o) The Owner or their lessee fails to comply with every term and condition stated herein.
4. During the period of this warranty, ABG, its agents and employees, shall have free access to the roof or podium areas during regular business hours
5. No liability will be accepted for any disruption caused by any repair work.
6. ABG shall have no obligation under this warranty until all invoices for materials and services associated with the ABG **blueroof** System, and where applicable for the final surface finishes, have been paid in full.
7. This warranty will apply only to installations where the final surface finishes have been agreed with ABG prior to installation; the maintenance is provided exclusively by an installer accepted by ABG; and an ABG **blueroof** Maintenance Agreement, compliant with the Operating & Maintenance Plan, is in effect for the duration of the warranty.
8. Where present the appearance of the vegetated/green roof final surface finishes should be expected to change over the years. A process of natural succession will result in the botanical evolution of the vegetated/green roof cover. The future distribution of plants species cannot be accurately predicted. The long-term coverage of the vegetated cover can be guaranteed only in conjunction with an ABG **blueroof** Maintenance Agreement required under this Warranty.
9. The Owner shall notify ABG in writing within 48 hours of discovering that any of the final surface finishes/coverage is insufficient, changed or damaged according to this warranty.
10. ABG's cumulative cost to repair or replace the ABG **blueroof** System shall not exceed the original cost of the ABG **blueroof** System (including only when applicable, costs for the final surface finishes when installed by an ABG approved installer).
11. ABG's failure at any time to enforce any of the terms or conditions stated herein shall not be construed to be a waiver of such provision.
12. All warranties set forth herein relating to the performance of the ABG **blueroof** System, including without limitation, the warranty relating to any the final surface finishes, will be voided if the Owner fails to maintain an ABG **blueroof** Maintenance Agreement with a contractor accepted by ABG for the duration of this warranty. The Owner shall supply copies of the executed ABG **blueroof** Maintenance Agreement on demand as proof that the maintenance programme complies with the Operating & Maintenance Plan and includes regularly scheduled inspections, and appropriate care of both the ABG **blueroof** System and the final surface finishes.
13. This warranty is extended solely and exclusively to the Owner of the Building at the time the ABG **blueroof** System is installed. It does not extend nor is it otherwise assignable or transferable to any other party unless approved in advance and in writing by ABG.

NO REPRESENTATIVE OF ABG HAS AUTHORITY TO MAKE ANY REPRESENTATIONS OR PROMISES EXCEPT AS STATED HEREIN.

This Warranty is effective from: …………………………… For: ………………….years.

Signed for and on behalf of ABG Ltd: …………………………………………………… Name: ……………………………………………………….

Position: …………………………………………………

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ABG **blueroof** – NHBC Requirements

The NHBC have approved various site-specific, ABG **blueroof** installations, with a stormwater attenuation/control system at roof and/or podium levels. [Note 1]

Key elements of the ABG **blueroof** system and that have been previously, NHBC approved:

- A zero-falls system.
- A hot melt (fully bonded, monolithic), or cold liquid applied, weathering membrane system.
  - Typically an inverted construction, but can also be a warm roof system.
  - A BBA approved system, and installed by an approved contractor by the manufacturer.
- An extruded (XPS) polystyrene insulation build-up (where an inverted build-up).
  - A BBA approved solution, and installed by an approved contractor by the manufacturer.
- Flow control chambers (minimum of 2 no. per roof area; final number TBC by ABG and Project Engineer)
  - With integral overflow capability.
  - Ability to cater for multiple design storm events (as required by the planning parameters).
- Installed by Geogreen team (part of ABG Ltd, and hence an approved installer of the blue roof system).
- Multiple layers of filter geotextiles within the blue roof construction, fulfilling SuDS requirements.

ABG will provide:

- Component details for the blue roof system (CE datasheets & product information). [Note 2]
- Section details for the specific project, blue roof construction. [Note 3]
- Blue roof calculations, showing storage rates and run-off rates for the project engineer's approval. [Note 4]
- Dual warranty with the flat roofing manufacturer/contractor, based on ongoing maintenance programmes & contracts being in place (as required for final surface finishes e.g. green roofs). [Note 5]
- Confirmation that ABG is happy to work with the BBA approved flat roofing manufacturer.

The BBA approved flat roofing manufacturer (and their nominated/approved contractor) will provide:

- Leak test certification and visual inspection of the roof deck and installed membrane.
  - Any remedial works that may be required & then reported back to the NHBC.
- Confirmation that the membrane system is suitable for zero falls, and any structural requirements (see last point below).
- Confirmation that they are happy to work with ABG's **blueroof** system.
- Confirmation that the insulation is suitable for zero falls, and any structural requirements (see point last below).
- Based on the project structural engineer's design requirements, will also provide compliance with:
  - Deflection analysis based on the installation of a blue roof system & any surface finishes.
  - Any further requirements for loadings, movement joints, position of outlets, drainage, and roof construction strengthening.

<u>Notes:</u>
ABG will provide details of:

1. ABG & NHBC site-specific approved project list.
2. ABG CE marking & product datasheets.
3. ABG **blueroof** section.
4. ABG **blueroof** calculation.
5. ABG **blueroof** warranty document.

Green roof case study

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Center Parcs Holiday Village, Woburn Forest, Bedfordshire

Image reproduced courtesy of Center Parcs

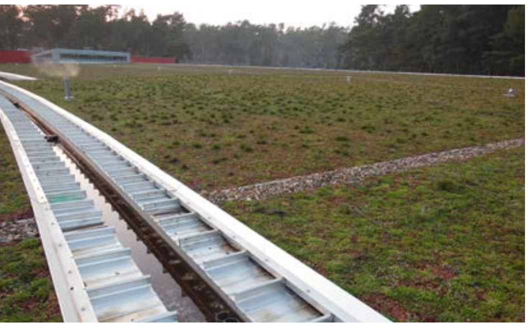
Center Parcs, Woburn Forest, Bedfordshire is the latest addition to Center Parcs UK resorts. Like many of their resorts it is set in woodlands drawing inspiration in its design from the surrounding environment. The development opened in June 2014 and represents a total investment of £250m.

Creating and managing biodiversity is at the heart of the philosophy of Center Parcs. It is considered fundamental to the guest experience and is a key element of the Center Parc villages that the design both protects and enhances the environment in which the village is set.

ABG have played a part in the development installing extensive green roofs across three structures within the complex including The Pancake House, Aqua Sana Spa and The Venue (a dedicated event space, with theatre style seating for up to 800 delegates, or 680 delegates for a gala dinner) comprising eight individual and flexible event suites, and a business centre, Jardin des Sports, and

Project information

Main contractor	Bowmer and Kirkland
Client	Center Parcs
Architect	Holder Mathias
Installation sub-contractor	Geogreen Solutions (ABG)
Project size	9,200m^2 across three roof areas
Roof substrate	Kalzip metal, profiled deck except The Pancake House which is single-ply warm roof.
Products	Roofdrain Green roof growing medium Planting On-going maintenance



service yard canopy roof.

Working for the main contractor, Bowmer and Kirkland, Geogreen Solutions began the installation during October 2013. The works included laying of the Roofdrain drainage and storage layer, spreading of the green roof growing medium before hydro-seeding or planting plug-plants across the roof area.

Roofdrain is a high performance drainage composite which forms a highly efficient drainage void above the waterproofing whilst also providing water storage to irrigate the planting dueing periods of dry weather. It comprises a cuspated HDPE drainage core onto which a geotextile filter fabric is bonded. The filter fabric allows water to percolate through the roof build up and into the void formed by the core whilst stopping fines entering the drainage system.

ABG's green roof growing medium is specially developed for use on roof structures. It comprises a blend of crushed brick and organic waste materials and is designed to provide the nutrients required to ensure the establishment of healthy vegetation across the green roof structure.

Maintenance

Following installation, Geogreen were retained on a maintenance contract requiring four visits in the first year following installation, and then an on-going contract requiring two visits per year to ensure the roof remains healthy and free from weeds.

About ABG

ABG are a market leader in the development of high performance geosynthetic systems for use in the built environment. Established for over 25 years ago and based in the UK, in the heart of the Yorkshire Pennines, ABG have built a reputation for delivering innovative system led solutions combined with technical support and outstanding customer service. Contact ABG today to discuss your project specific requirements and discover how ABG knowledge and products can help on your project.

ABG Design, Supply & Install
- Geogreen Green Roof Projects

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Appendix Q – Lancing Brooks Modelling

New Salts Farm Road, Shoreham Modelling Report

Final Report

March 2017

Hyde Housing

The Hyde Group

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125-135 Preston Road,

Brighton

BN1 6AF

JBA Project Manager

Jack Southon,
JBA Consulting,
35 Perrymount Road,
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West Sussex,
RH16 3BX

Revision History

Revision Ref / Date Issued	Amendments	Issued to
Version 1.0 (February 2017)	Draft	Andrew Picton
Version 2.0 (March 2017)	Final - update Appendix	Andrew Picton

Contract

This report describes work commissioned by Andrew Picton from Tully De'ath Consultants on behalf of Hyde Housing Association Ltd, by an email dated 02/11/2017. Aaron Barber, Ffion Wilson and Jack Southon of JBA Consulting carried out this work.

Prepared by .... Aaron Barber
Technical Assistant

Ffion Wilson BSc MSc PIEMA
Analyst

Reviewed by .... Jack Southon BSc MSc FRGS MCIWEM C.WEM CSci CEnv
Project Manager

Purpose

This document has been prepared as a Final Report for Hyde Housing Association Ltd. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to Hyde Housing Association Ltd.

Acknowledgements

We would like to acknowledge and thank Andrew Picton and Julian Turner from Tully De'ath Consultants as well as Environment Agency for their provision of the relevant information required for this assessment

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A printed copy of the main text in this document will result in a carbon footprint of 49g if 100% post-consumer recycled paper is used and 63g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Contents

List of Figures

1 Introduction

JBA Consulting was appointed by Andrew Picton from Tully De'ath to prepare a Flood Risk Modelling Report to support the assessment of fluvial flood risk to New Salts Farm Road, Shoreham.

The above referenced site was identified as a potential strategic development site for residential purposes within Adur District Council's emerging 2016 Local Plan. Hyde Housing Association Ltd have proposed to develop the agricultural area between New Salts Farm and the Hasler Estate to provide a new residential area in five phases.

A hydraulic model was required to refine the understanding of current and future flood risk to the site and identify the effect of possible mitigation measures that could be implemented within the site's existing drainage system known as the Lancing Brooks. The hydraulic modelling will be used to inform the Flood Risk Assessment (FRA) for the site (to be prepared by others).

2 Hydraulic modelling

An existing 1D ISIS model of the area had been developed to inform the 2015 Surface Water Management Plan. This model was based on cross section topographical survey and run for steady state flow estimates that did not appear to be based on any recognised method of flow estimation. The model was not suitable for the purposes of supporting a site-specific Flood Risk Assessment. Therefore, a new model and new hydrology was required. To this end a new model was development in InfoWorks ICM based on open LiDAR data and river sections were taken from the 1D Lancing Brook ISIS Model. Direct rainfall inflows for the hydraulic model have been derived based on the Environment Agency's flood estimation guidelines (Version 5, 2015). An FEH Calculation Record can be found in Appendix A.

2.1 Catchment boundary

The catchment boundary was based on two neighbouring FEH catchments which are located within in the proposed development site. These are identified as the south-western and south-eastern FEH catchments in Figure 2-1 below. The entire Lancing Brooks catchment was not used as it was considered too large an area and may not represent the sites characteristics as accurately..

From review of channel gradients and flow direction in the existing ISIS model it was identified that a number of channels drained away from the site to the north. Additionally, the northern FEH catchment, illustrated in Figure 2-1, drains to the north-east as illustrated by the coloured nodes. Therefore, the northern FEH Catchment was not considered as representative of the development site characteristics as the south-eastern and south-western FEH Catchments.

The Lancing SWMP indicates that surface water runoff from the A27 drains via a series of outfalls into the Lancing Brooks. However, this will flow south and eastwards from the A27 into the Northern catchment area shown in Figure 2-1. As the outflow for this FEH catchment is approximately 1km to the north-east of the site, the outflow from the A27 was not considered in the model.

The catchment area used was sensitivity tested - details of this testing and further details of the direct rainfall methodology can be found in the accompanying FEH Calculation record.

Figure 2-1: FEH catchments in proximity to the site draining to their respective colour nodes

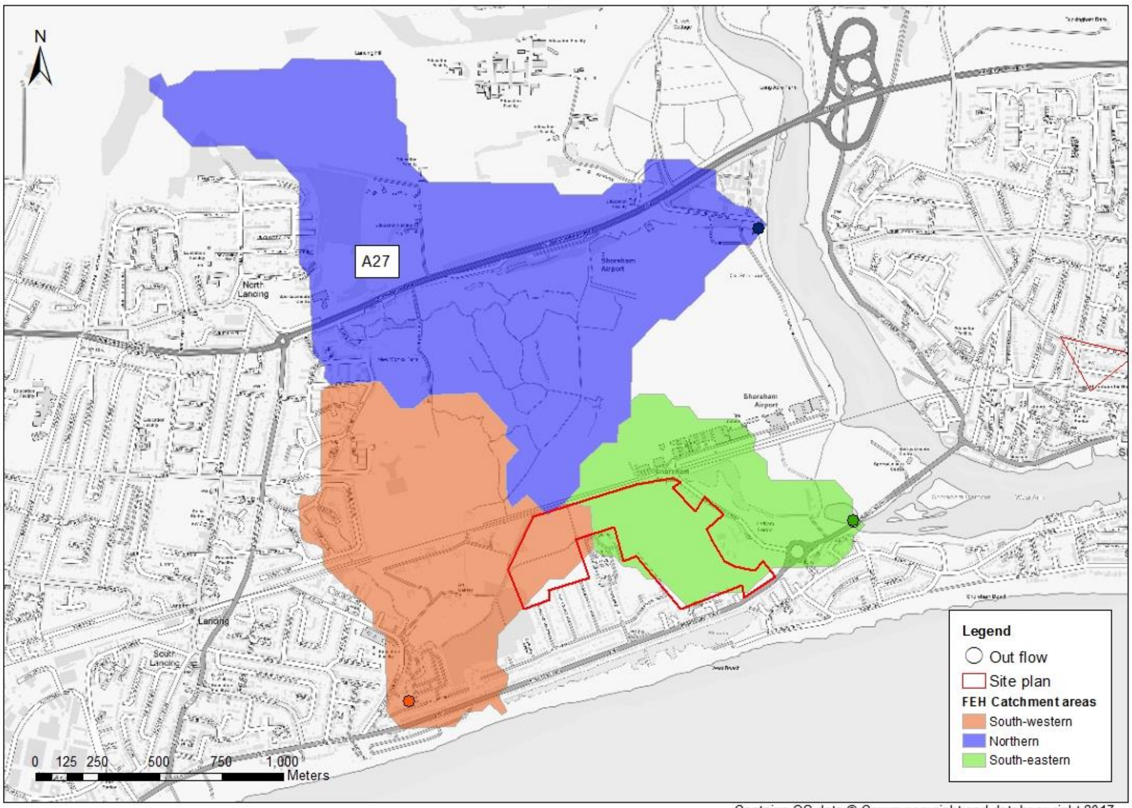
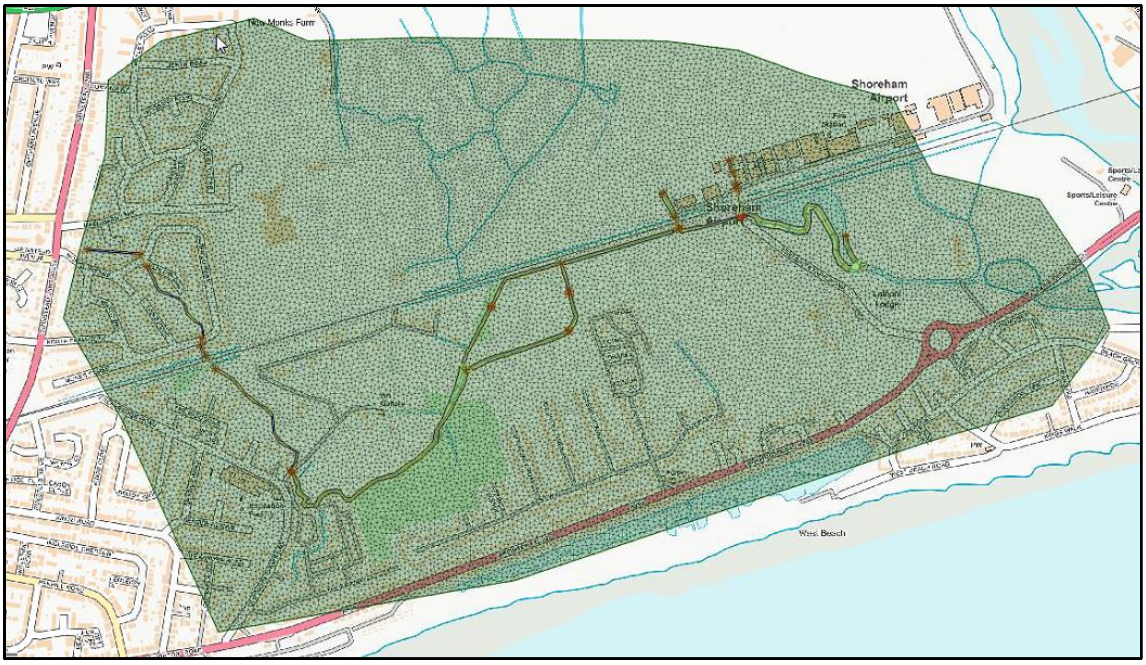


Figure 2-2 below shows the extent of the model 2D domain and 1D channel elements.

Figure 2-2: Extent of the modelled 2D domain and 1D channel elements



2.2 Return periods

The model was run for a range of baseline conditions for the following return periods:

- 100-year (1% AEP)
- 1000-year (0.1% AEP)
- 100- year (1% AEP) + 20% climate change

- 100-year (1% AEP) + 40% climate change

In February 2016 new climate change guidance was published by the Environment Agency (Flood risk assessments: climate change allowances) to support the assessment of flood risk in line with National Planning Policy Framework (NPPF) which sets out how the planning system should help minimise vulnerability and provide resilience to the impacts of climate change.

The climate change allowances are predictions of anticipated change for peak river flow by river basin district and peak rainfall intensity.

They are based on climate change projections and different scenarios of carbon dioxide (CO2) emissions to the atmosphere. There are different allowances for different epochs or periods of time over the next century.

To inform the Flood Risk Assessment we have, in line with Table 2 of the updated climate change guidance, applied the a 20% and 40% uplift to account for the "Central" and "Upper End" respectively to represent the anticipated changes in extreme rainfall intensity in small catchments for the 2080s epoch (2070 to 2115)

https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

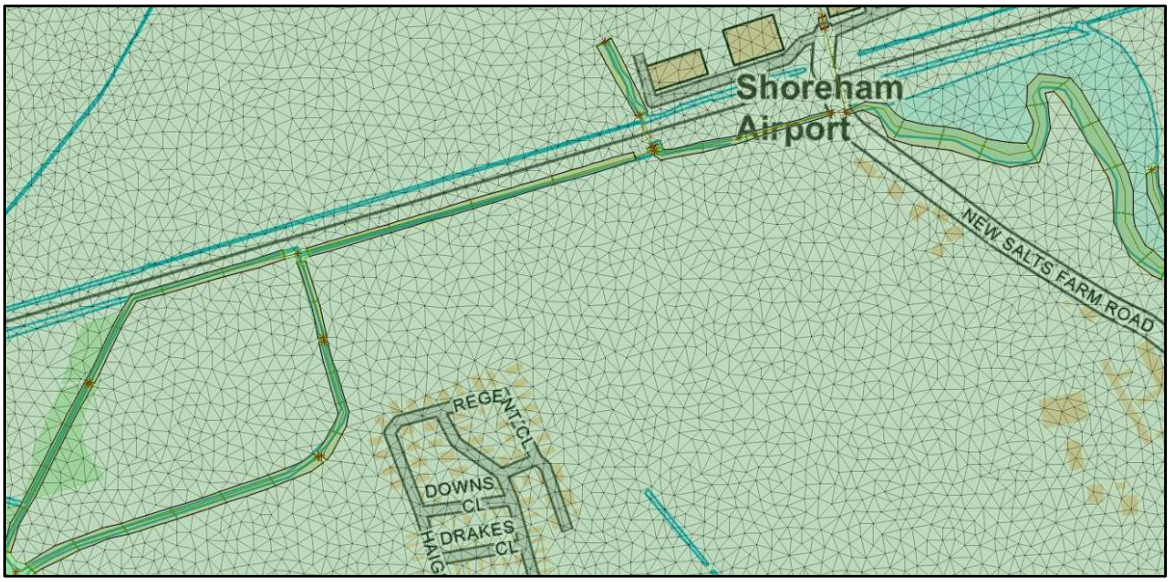
2.3 Scenario testing

The model was then used to test various scenarios. These scenarios were:

- Baseline: This is considered to represent the existing topographic situation.

Figure 2-3 below shows a section of the baseline model between New Salts Farm Road and Windsor Way.

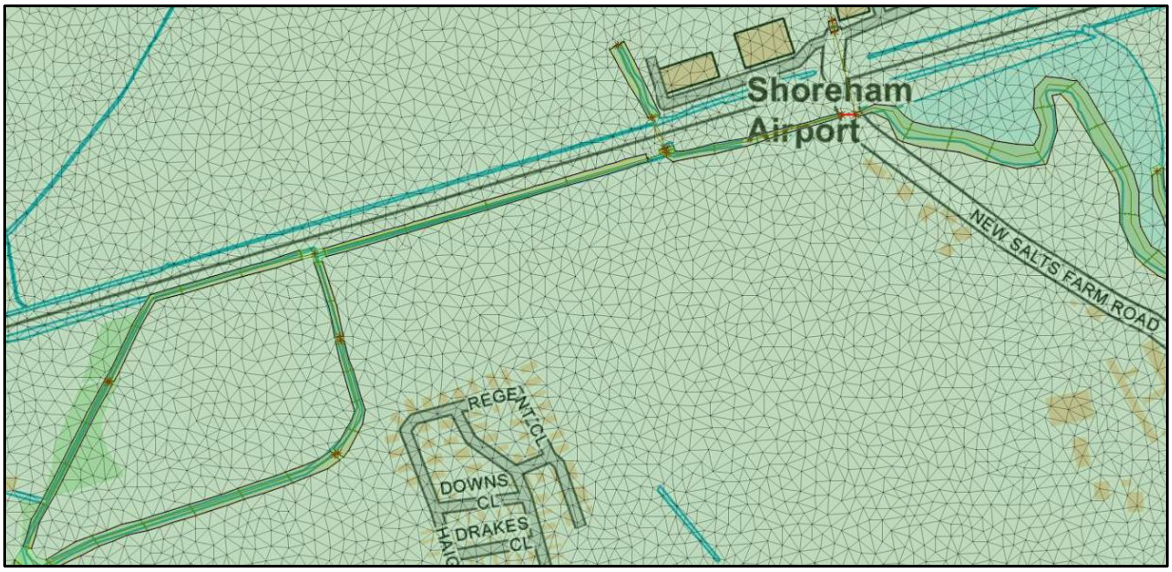
Figure 2-3: Section of the baseline model between New Salts Farm Road and Windsor Way



- NSFR V1: Increase the size of the bridge under New Salts Farm Road from a 950mm circular culvert to a 3700mm(w) by 1000mm(h) rectangular culvert.
- NSFR V2: Increase the size of the bridge under New Salts Farm Road from a 950mm circular culvert to a 4000mm(w) by 1500mm(h) rectangular culvert.

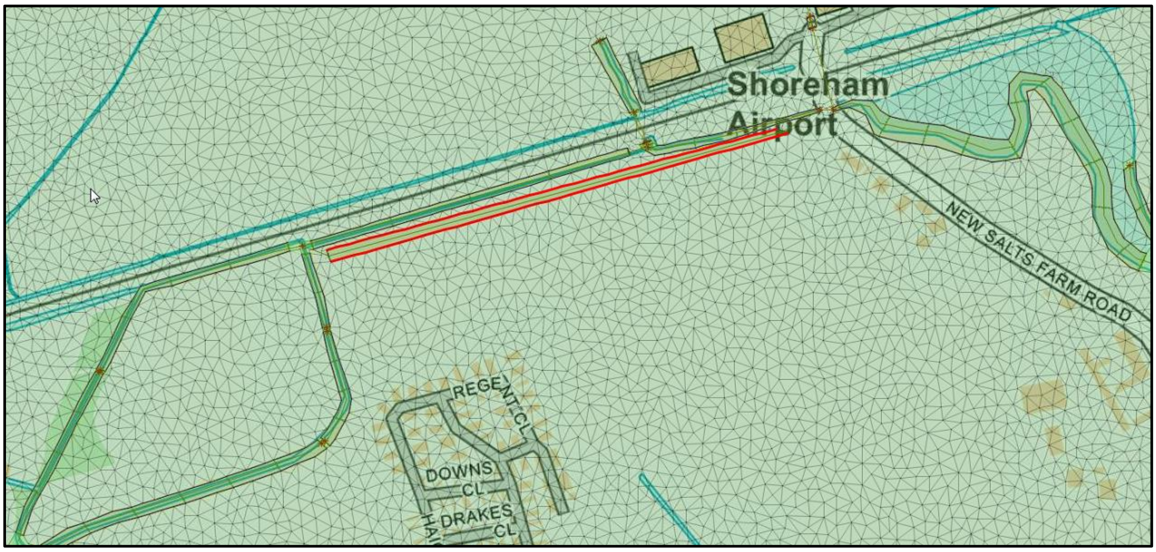
Figure 2-4 below highlights (in red) the bridge under New Salts Farm Road that is amended in scenarios NSFR V1 and NSFR V2.

Figure 2-4: Location of the bridge under New Salts Farm Road



- Swale: Using a ‘corridor’ of land to provide a new channel (highlighted in red on Figure 2-5 below) which connects the ditch towards the western end of the site and flows back into the existing channel on the eastern boundary of the site just upstream of the bridge.

Figure 2-5: Swale location



- Tide lock: Re-run the above scenarios assuming the outfall is tide locked for the duration of the event.

3 Results

The results of the model were converted into .shp files and then represented within maps produced in ArcGIS v10.4. Two versions of the maps were produced:

- Version 1: Raw depth data
- Version 2: Hazard and depth maps which excluded flooding with a Hazard less than 0.575m in line with the national updated Flood Map for Surface Water Guidance

Appendices

A FEH Calculation Record

JBA consulting

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Doncaster
Dublin
Edinburgh
Exeter
Glasgow
Haywards Heath
Isle of Man
Limerick
Newcastle upon Tyne
Newport
Peterborough
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Flood estimation report

Introduction

This report template is based on a supporting document to the Environment Agency's flood estimation guidelines (Version 5, 2015). It provides a record of the hydrological context, the method statement, the calculations, and decisions made during flood estimation and the results.

Contents

Revision history

Revision ref / date issued	Amendments	Issued to
Version 1.0 (17/02/2017)	N/A	A Picton
Version 2.0	Internal draft	
Version 3.0 (08/03/2017)	Amended to respond to WSCC comments	A Picton

Approval

	Name and qualifications	Date
Method statement prepared by:	A Barber BSc	17/02/2017
Method statement reviewed by:	J Southon BSc MSc FRGS MCIWEM C.WEM CSci CEnv	17/02/2017
Calculations prepared by:	A Barber BSc	17/02/2017
Calculations reviewed by:	J Southon BSc MSc FRGS MCIWEM C.WEM CSci CEnv	17/02/2017

Abbreviations

AM.................................. Annual Maximum

AREA .... Catchment area (km^2)

BFI .... Base Flow Index

BFIHOST .... Base Flow Index derived using the HOST soil classification

CFMP.............................. Catchment Flood Management Plan

CPRE.............................. Council for the Protection of Rural England

FARL............................... FEH index of flood attenuation due to reservoirs and lakes

FEH................................. Flood Estimation Handbook

FSR................................. Flood Studies Report

HOST .... Hydrology of Soil Types

NRFA .... National River Flow Archive

POT................................. Peaks Over a Threshold

QMED .... Median Annual Flood (with return period 2 years)

ReFH .... Revitalised Flood Hydrograph method

SAAR .... Standard Average Annual Rainfall (mm)

SPR................................. Standard percentage runoff

SPRHOST .... Standard percentage runoff derived using the HOST soil classification

Tp(0) .... Time to peak of the instantaneous unit hydrograph

URBAN .... Flood Studies Report index of fractional urban extent

URBEXT1990 .... FEH index of fractional urban extent

URBEXT2000 .... Revised index of urban extent, measured differently from URBEXT1990

WINFAP-FEH .... Windows Frequency Analysis Package – used for FEH statistical method

1 Method statement

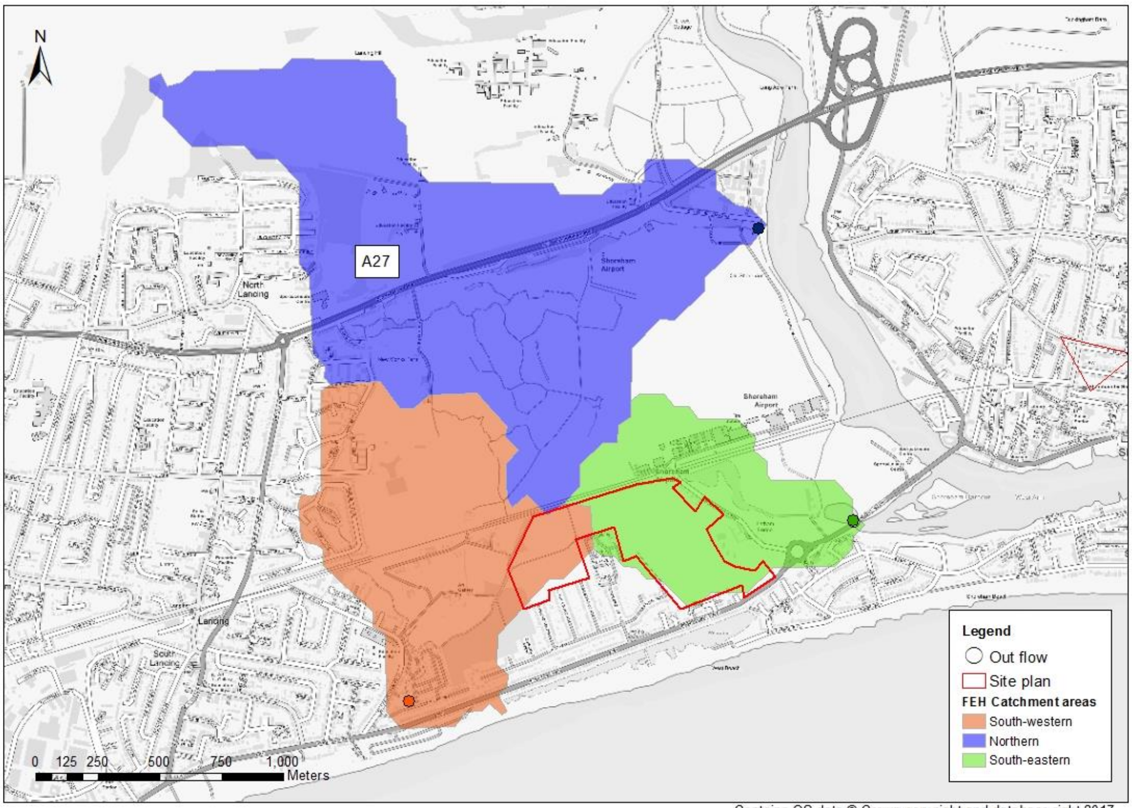
1.1 Requirements for flood estimates

Item	Comments
Overview • Purpose of study • Peak flows or hydrographs? • Range of return periods and locations	The aim of this project is to better understand the flood risk associated with the Lancing Brooks drainage system, between Shoreham and Lancing, West Sussex. The purpose of the hydrological assessment is to derive inflows for the detailed hydraulic model. Full hydrographs will be derived for the following annual exceedance probability (AEP) events: 5%, 1%, and 0.1%. Additional model runs are needed for the 1% AEP event, with flows increased to allow for the possible effects of climate change. The climate change scenarios follow the Environment Agency's guidance: Flood risk assessments: climate change allowances[1] initially released in February 2016. In line with Table 2 of the updated climate change guidance, 20% and 40% uplift to account for the "Central" and "Upper End" respectively was applied to represent the anticipated changes in extreme rainfall intensity in small catchments for the 2080s epoch (2070 to 2115).

[1] Environment Agency (2016). Flood risk assessments: climate change allowances. https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

1.2 The catchment

Item	Comments
Map (Include river network, catchment boundary and gauging stations)	



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Item	Comments
Description Include topography, climate, geology, soils, land use and any unusual features that may affect the flood hydrology.	The Lancing Brooks are a series of ditches that drain the Lancing area, between Lancing and Shoreham, West Sussex. The ditches flow through open scrubland, and is culverted under Brighton/Shoreham Airport and the West Coastway Railway Line. From here, the drains flow south east towards the tidal sluice outflow with the River Adur. The Lancing Brooks are a series of drainage ditches on flat, low lying land near the coast. The site is underlain by superficial alluvium and tidal deposits, sitting above bedrock of clay and chalk. The clay is locally variable in thickness and extent in the Lancing Brooks catchment. The underlying Chalk deposit outcrops at the surface towards the north of the site. Areas of land to the north of the railway line are overlain with several meters of made ground, below the new Golf Course and Brighton and Hove football training ground. The Lancing Brooks catchment is permeable, with a BFIHOST of 0.68 (>0.65 threshold for defining a permeable catchment). Regional groundwater flow occurs from the North in the Chalk downs, flowing south downhill towards the coast. Within the Lancing Brooks, groundwater emergence is variable, depending on the presence of the overlying clay, acting as an aquitard, preventing emergence of groundwater. The superficial alluvium deposits within the Lancing Brook catchment is also fairly permeable, containing local scale aquifers, sometimes perched above clay, restricting infiltration to the deeper chalk aquifer. Groundwater flooding is known to occur, especially in the winter, where the regional groundwater flow in the chalk aquifer is greater, and the water table is higher. In areas where the clay aquitard is breached, the groundwater has potential to rise to the surface in the form of an artesian well. The Lancing Brooks catchment is moderately urbanised. Surface water sewer outfalls are located around the edges of development in Shoreham and Lancing. The existing developments currently have flooding issues related to emergence of groundwater, and surface water flooding.

	There are three identified surface water outfall locations within the Lancing Brooks system, shown in the figure above. The outfalls broadly correspond with the FEH catchment areas. However, there are some drains which cut across topographic catchment boundaries. The outfall locations are tidal on their downstream end which can result in tidal locking, which prevent the Lacing Brook drainage network from fully discharging into the Adur Estuary, and this would allow water levels to increase in the Lancing Brook drainage network. The catchment is poorly defined due to the low relief and the artificial nature of the drainage network, and is ungauged.

1.3 Source of flood peak data

Source Record any changes made	There is no available flood peak data within the Lancing Brook catchment, and no representative donor was identified.

1.4 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data	Details
Check flow gaugings (if planned to review ratings)	N/A		None Available	Ungauged
Historic flood data Include chronology and interpretation of flood history in Annex or separate report.	Yes	Limited	Lancing SWMP, Internet	Past 2012/13 and 2013/14 flooding events in Lancing housing estates, caused by surface water flooding and maintenance issues with surface water sewers. Ground water emergence and effect on soakaway performance.
Flow or river level data for events	N/A		None Available	Ungauged
Rainfall data for events		Yes	Available but not used	TBR data could have been licensed from the Met office or the EA however it was not felt that it would add significant rigor to the flow estimation over and above the use of FEH-DDF values
Potential evaporation data		Yes	Available but not used	PE data could have been licensed from the EA however it was not felt that it would add significant rigor to the flow estimation over and above sensitivity testing of runoff coefficients.
Results from previous studies		Yes	SWMP	A 1D model was produced for the SWMP. The inflows for this study were not supported by recognised flow estimation methods
Other data or information (e.g. groundwater, tides, channel widths, low flow statistics)		Yes		Tide data available but not used. Tide-locked scenarios ran. Soil Moisture Deficit data not acquired for this assessment.

1.5 Hydrological understanding of catchment

Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	The main area of interest is shown in red on the figure above. It is located to the south of the railway line that crosses the brook system and spans across two of the three FEH catchment areas. The likely cause of flooding in this area is excess surface water which is unable to drain away due to tide locking issues and/or as a result of elevated groundwater levels. This results in the drainage ditches exceeding the channel capacity.
Any unusual catchment features to take into account?	Poorly defined topographical catchment due to the dense network of drainage ditches. Tidal Locking. Low lying topography. Some drainage ditches may cut across topographical catchment boundaries. Bidirectional flow is possible in these channels. No pumps are present.

1.6 Initial choice of approach

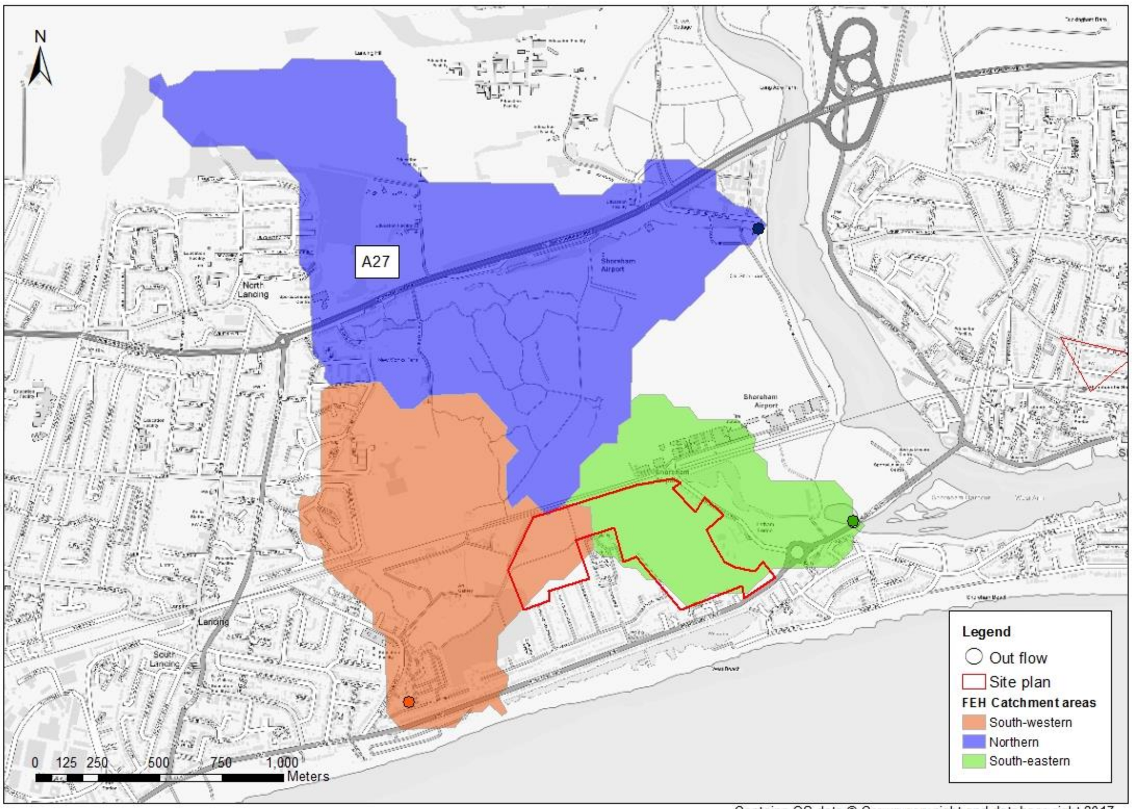
Is FEH appropriate? (it may not be for extremely heavily urbanised or complex catchments) If not, describe other methods to be used.	No - FEH Statistical methods and ReFH methods are not appropriate due to the low topographic gradient which would make it difficult to select inflow points that properly represent the contribution of flow within the catchment.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	Initially it was identified that using Continuous Simulation would be the most appropriate hydrological method for this complex catchment. However, it was found there was insufficient hydrometric data to calibrate the Probability Distributed Model (PDM). Therefore, it was proposed that Direct Rainfall would be applied. Direct Rainfall has the significant advantage of avoiding applying discrete inflows to the model at specified locations (as is usual in hydraulic modelling) which may result in unreliable results because of gentle topography of the coastal plains. Direct rainfall also has the benefit of providing outputs compatible with the requirements for a combined source (integrated model) model In an integrated model, rainfall is routed overland, through the pipe network and into the river systems. With exception of groundwater, this is a good description of the physical process occurring within a catchment. In effect, this is a fully distributes hydraulic model and appropriate for use as a pseudo fluvial model. Similar recent studies on nearby catchments with similar characteristics have shown that the application of the FEH Statistical, ReFH, or FEH Rainfall Runoff methods not to be the robust way forward for this type of catchment. For the direct rainfall hydrological approach to give an accurate representation of the flows, the entire topographic catchment (the area contributing runoff to the site) had to be modelled. Due to the gentle relief in the area the it was concluded that the FEH catchments were not representative. Therefore, a desktop assessment was undertaken to delineate the contributing drainage area for the site. This was undertaken in GIS software and included reviewing topographic LIDAR data for the

	wider area, OS mapping, available topographic and cross section survey, and available surface water sewer records. From this a contributing catchment was defined. This area was extended out by buffer of 100m to account for uncertainty and this area was used for the 2D zone. The FEH CD-ROM contains a database of catchment descriptors, along with parameters of the FEH rainfall Depth-Duration-Frequency (DDF) model. Parameters are provided both for point rainfall, on a 1km2 grid, and for catchment-average rainfall. The parameters produced for a watercourse catchment are an amalgamation of the 1 km2 grid parameters within the catchment. Design rainfall statistics for each of the 1km grid squares across the catchment were extracted from the FEH CD-ROM (v3) in order to assess the variability in rainfall depths and storm duration across the catchment. As there was little variation in these parameters across the model domain the parameters for the South-west catchment DDF parameters were used to create a hyetograph specific to the model domain. The events simulated for this study follow a summer profile. During the summer months' interception from deciduous trees and evapo-transpiration can be important losses, however given the land cover type found in the catchment it was considered that these losses would be negligible during a flood event. Therefore, infiltration was the major loss considered. The approach selected to model rainfall losses via infiltration in rural areas of the catchment was to calculate infiltration in the hydraulic model using fixed infiltration zones. These were applied at a uniform spatial distribution across the 2D zone. The design runoff used SPRHOST value from catchment descriptors to determine runoff. This was sensitivity tested using Other values used for runoff coefficient testing in 10% intervals up to 100% runoff to represent a completely saturated or completely impermeable surface.
Software to be used (with version numbers)	FEH CD-ROM V3.0 InfoWorks ICM v7.0

Hyetograph	**FEH Rainfall Parameters**					
	C	D1	D2	D3	E	F
Catch 1	-0.026	0.407	0.315	0.366	0.309	2.392

2 Locations where flood estimates required

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.



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2.1 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
Catch1	Direct Rainfall	Lancing Brooks	Lancing Brooks	519175	104680	N/A	N/A

N.B. Based on south-west catchment

2.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	SPRHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 2000	FPEXT
Catch1	1	0.34	0.68	28.27	1.54	5.6	725	0.1395	0.7063

Note: Red text denotes catchment descriptors which have been changed from FEH CD-ROM values. See Section 2.3 for an explanation of these changes.

2.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (add maps if needed)	FEH catchment areas were identified. However, there is a low level of confidence in the drainage areas that contribute flow within the catchment due to the low relief and artificial nature of the drainage system. It was not possible to accurately derive updated catchment areas using common methods (such as ArcHydro) therefore direct rainfall was used. The direct rainfall area was determined by reviewing the channel gradients and drainage directions to establish likely drainage catchment area and then increasing this to account for any areas that drain away from the site.
Record how other catchment descriptors were checked and describe any changes. Include before/after table if necessary.	No other catchment descriptors were adjusted.
Source of URBEXT	N/A
Method for updating of URBEXT	N/A

3 Statistical method NOT USED

3.1 Overview of estimation of QMED at each subject site

Site code	QMED from CDs (m^3/s) RURAL	Final method	Data transfer						Final estimate of QMED (m^3/s) URBAN
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted ave. adjustment	
Are the values of QMED spatially consistent?									

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added below.

The QMED adjustment factor A/B for each donor site is given in Table 3.2. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

Important note on urban adjustment

The method used to adjust QMED for urbanisation, for both subject sites and donor sites, is that published in Kjeldsen (2010)[2] in which PRUAF is calculated from BFIHOST. The result will differ from that of WINFAP-FEH v3.0.003 which does not correctly implement the urban adjustment of Kjeldsen (2010). Significant differences will occur only on urban catchments that are highly permeable.

3.2 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	
Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	

[2] Kjeldsen, T. R. (2010). Modelling the impact of urbanization on flood frequency relationships in the UK. Hydrol. Res. **41**. 391-405.

3.3 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing	Method (AM or POT)	Adjust-ment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)

3.4 Derivation of pooling groups

Several subject sites may use the same pooling group.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)

Notes
Pooling groups were derived using the procedures from Science Report SC050050 (2008).

3.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (Error! Reference source not found.)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape after adjustments)	Growth factor for 100-year return period

Notes
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.
Urban adjustments are all carried out using the v3 method: Kjeldsen (2010).
Growth curves were derived using the procedures from Science Report SC050050 (2008).

3.6 Flood estimates from the statistical method

3.6.1 Design Events

Table 3-1: Flow estimates derived using pooled analysis with donor adjustment

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	30	50	75	100	200	1000

Table 3-2: Flow estimates derived using an Enhanced Single Site analysis with donor adjustment

Site code	Flood peak (m^3/s) for the following return periods (in years)										
	2	5	10	20	25	30	50	75	100	200	1000

3.6.2 Climate change events

Table 3-3: Climate change flow estimates derived using pooled analysis with donor adjustment

Site code	Flood peak (m^3/s) for the following climate change scenarios		
	1% (plus 35%)	1% (plus 45%)	1% (plus 105%)

Table 3-4: Climate change flow estimates derived using an enhanced single site analysis

Site code	Flood peak (m^3/s) for the following climate change scenarios		
	1% (plus 35%)	1% (plus 45%)	1% (plus 105%)

4 Revitalised flood hydrograph (ReFH) method - NOT USED

4.1 Parameters for ReFH model (rural catchments)

Site code	**Method** OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	**Tp (hours)** Time to peak	**C_{max} (mm)** Maximum storage capacity	**BL (hours)** Baseflow lag	**BR** Baseflow recharge
Brief description of any flood event analysis carried out (further details should be given in the annex)	N/A				

4.2 Design events for ReFH method

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?				

4.3 Flood estimates from the ReFH method

Site code	Flood peak (m^3/s) for the following return periods (in years)									

5 Discussion and summary of results

5.1 Final choice of method

Choice of method and reasons. Include reference to type of study, nature of catchment and type of data available.	Taking into consideration the flat nature of the catchment, and the predominant sources of water coming from surface water runoff, local runoff and groundwater, the Direct Rainfall method is considered the most suitable. The method chosen to derive the rainfall inputs is the FEH DDF method. Hydrological analysis was undertaken identify critical storm duration. The direct rainfall statistics will be input into InfoWorks ICM. Fixed infiltration losses were selected, a steady runoff coefficient is set. This remains constant throughout the simulation

5.2 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main assumptions in this study are that: • Catchment boundary – it is assumed that the 2D direct rainfall boundary accounts for the area that contributes runoff to the relevant section of the Lancing Brook system that presents flood risk to the site. It is assumed that the LIDAR data available fairly reflects the catchment topography. • Other watercourses – it is assumed that the remainder of the Lancing Brook with a channel gradient draining away from the site does not contribute to flood risk to the site. This includes much of the drainage to the west and north of the airport and all areas north the of A27. • Contributing areas – it is assumed that there is not significant inflow to the top of the Marsh Barn Lane channel from areas to the west of the A2025 (Grinstead Lane). It is assumed that any urban drainage infrastructure in these areas would have a limited design capacity (up to 1:30 AEP capacity) and that any overland flows in excess of the local urban drainage infrastructure would be curtailed by the A2025 (Grinstead Lane) which bisects the area. It is not known if any urban drainage infrastructure is connected upstream. The 2D model boundary around the area is based on the south-west FEH catchment boundary, extrapolated into the south east area for the outfall. • Runoff Coefficient – Sensitivity tests have been undertaken on runoff coefficients. Initially based on SPRHOST value of 28.27, and tested up to 100% runoff in 10% increments thereafter • SPRHOST runoff has been used throughout the model domain including the more urban areas to the south and west of the model domain. Runoff may be greater in these urban areas, but so too will the flows lost to urban drainage infrastructure. It is understood that most of the properties in this area drain to soakaway. It was concluded that these areas only represent a small portion of the contributing area are the increase in runoff as a result of the presence of impermeable surfaces would be similar to the decrease observed by accounting for the presence of urban drainage infrastructure including soakaways. • A range of storm duration scenarios were tested in the Hydraulic model. These tests produced a critical duration for the site of 4.5 hours. As a result of this test it was recommended a design storm duration of 4.5 hours (270

	minutes) was used in the model. A summer profile was used.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed.	DDF rainfall is extrapolated when used beyond the 1:200 AEP event However it is common practice to use up to 1:1000 AEP event.
Give what information you can on uncertainty in the results, e.g. confidence limits from Kjeldsen (2014).	There are a number of catchment characteristics that make flood flow estimation difficult for the catchment: • Permeable nature of the catchment. • Urban areas adjacent to the catchment • Tidally influenced outfall. • Low-lying and flat with extensive floodplains. • Multiple outfalls for drainage system with potential for channels to flow in both directions. • Number of potential sources of flooding under different conditions - fluvial, pluvial, groundwater. The 'Flood estimation guidelines' state that: "It is inevitable that on unusual catchments or for extreme return periods there are few ideal methods. Standard methods are likely to be least applicable to very small or large catchments, complex urban catchments, permeable catchments and extreme events." On this basis, 'standard methods' were rejected for this study in favour of Direct Rainfall. However, Direct Rainfall also has inherent areas of uncertainty. One aspect of uncertainty considered for the Direct Rainfall approach was that the Annual Exceedance Probability (AEP) of the rainfall event is unlikely to equal the AEP of the fluvial flow. The AEP of a fluvial flow is a result of the joint probability of the rainfall event and the antecedent conditions. In order to make a direct rainfall approach equivalent to a fluvial flow the antecedent conditions for design events have been set. An appropriate antecedent condition has been estimated based on Catchment Descriptors and local knowledge.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	This work has been used to inform a site-specific flood risk assessment. Future use of these flows should satisfy themselves that rainfall parameters are appropriate for the subject site and that critical storm duration and profile has been considered prior to re-using the results of these estimates.
Give any other comments on the study, e.g. suggestions for additional work.	Future gauging of the Lancing Brooks would be recommended to improve confidence in flow estimates in the catchment.

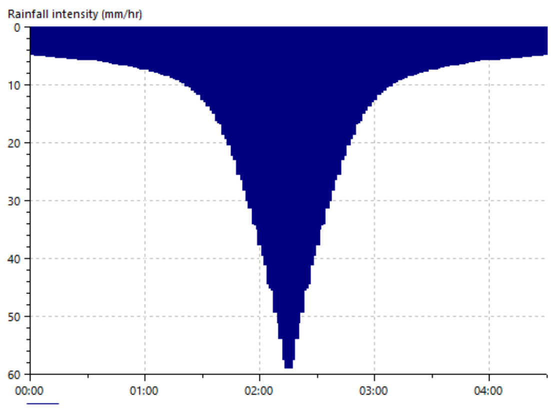
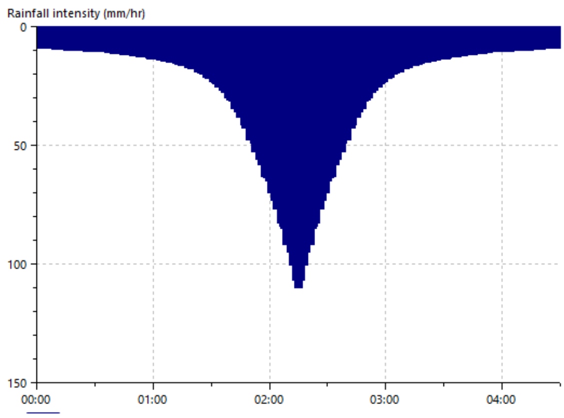
5.3 Checks

Are the results consistent, for example at confluences?	Yes, flood peaks within the model increase with both catchment size and return period.
What do the results imply regarding the return periods of floods during the period of record?	No observed flood peaks to compare against.
What is the range of 100-year growth factors? Is this realistic?	N/A
If 1000-year flows have been derived, what is the range of ratios	Only rainfall values have been produced.

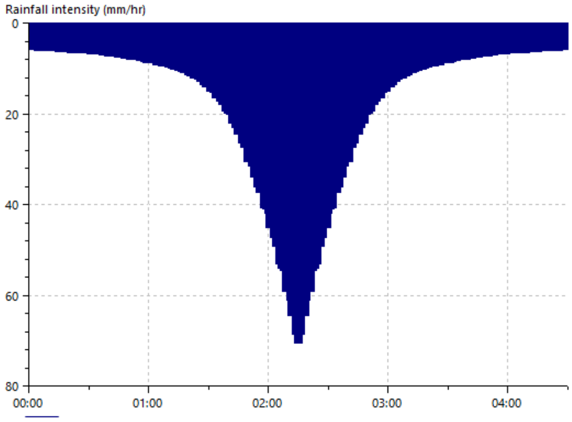
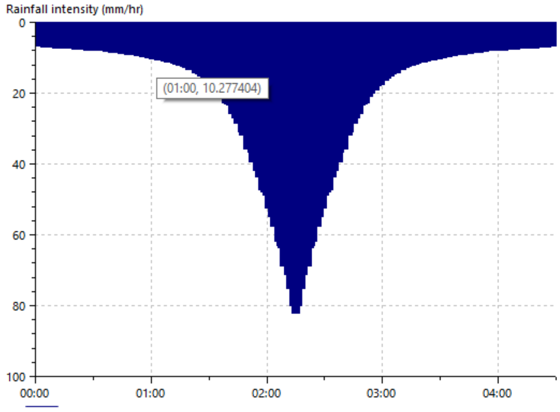
for 1000-year flow over 100-year flow?	
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	A 1D model was produced for the SWMP. The inflows for that study were not supported by recognised flow estimation methods therefore they have not been compared to the values here in.
Are the results compatible with the longer-term flood history?	No quantitative flood history available to compare to.
Describe any other checks on the results	No other checks on specific flows have been undertaken. The flowing sensitivity test have been undertaken: Runoff Coefficient -tested from SPRHOST coefficient of 28.27% standard, with scenarios also tested at increments of 10% up to 100%. Direct rainfall Catchment – tested with larger active domain (see Annex). Storm duration - Tested for a range of storm duration to determine critical duration for the site.

5.4 Final results –

5.4.1 Design Events

Site code	Rainfall intensity (mm/hr) for the following annual exceedance probabilities (%)	
	1	0.1
Catch 1		

5.4.2 Climate change events

Site code	Rainfall intensity (mm/hr) for the following annual exceedance probabilities (%)	
	1+20%	1+40%
Catch 1		

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Hyetographs stored within ICM database and in section 7 below

6 Applying the flows to the hydraulic model

6.1 Simulating design floods in the hydraulic model

How are the inflows applied to the hydraulic model to represent design flood conditions throughout the model domain?	Hyetographs applied to as direct rainfall to 2D domain as shown below. See comments on runoff coefficient above. 
Record the results of any checks between the peak flows given in this report and the peak flows within the hydraulic model.	No peak flows established in this report. Peak rainfall intensities match values applied. Losses are accounted for using SPRHOST coefficients, detailing the percentage of rainfall runoff into channels. SPRHOST coefficient of 28.27% standard, with scenarios also tested at increments of 10% up to 100%.

6.2 Calibration flows for the model

Is there enough certainty in hydrological data or models to calculate flows for calibration events that will reduce uncertainty in the hydraulic model structure and parameters?	No
How are the flows calculated for calibration events?	No calibration data available

Annex – supporting informationDesign Rainfall values

Time	Rainfall intensity (mm/hr) for the following annual exceedance probabilities (%)			
	1.00	1+20%	1+40%	0.10
00::00:00	5.01	5.01	5.01	9.34
00::00:01	5.01	5.01	5.01	9.34
00::00:02	5.06	5.06	5.06	9.42
00::00:03	5.17	5.17	5.17	9.63
00::00:04	5.17	5.17	5.17	9.63
00::00:05	5.17	5.17	5.17	9.63
00::00:06	5.17	5.17	5.17	9.63
00::00:07	5.17	5.17	5.17	9.63
00::00:08	5.31	5.31	5.31	9.89
00::00:09	5.32	5.32	5.32	9.92
00::00:10	5.32	5.32	5.32	9.92
00::00:11	5.32	5.32	5.32	9.92
00::00:12	5.32	5.32	5.32	9.92
00::00:13	5.40	5.40	5.40	10.07
00::00:14	5.48	5.48	5.48	10.21
00::00:15	5.48	5.48	5.48	10.21
00::00:16	5.48	5.48	5.48	10.21
00::00:17	5.48	5.48	5.48	10.21
00::00:18	5.49	5.49	5.49	10.24
00::00:19	5.63	5.63	5.63	10.50
00::00:20	5.63	5.63	5.63	10.50
00::00:21	5.63	5.63	5.63	10.50
00::00:22	5.63	5.63	5.63	10.50
00::00:23	5.63	5.63	5.63	10.50
00::00:24	5.74	5.74	5.74	10.71
00::00:25	5.79	5.79	5.79	10.79
00::00:26	5.79	5.79	5.79	10.79
00::00:27	5.79	5.79	5.79	10.79
00::00:28	5.79	5.79	5.79	10.79
00::00:29	5.84	5.84	5.84	10.88
00::00:30	5.95	5.95	5.95	11.09
00::00:31	5.95	5.95	5.95	11.09
00::00:32	5.95	5.95	5.95	11.09
00::00:33	5.95	5.95	5.95	11.09
00::00:34	5.95	5.95	5.95	11.09
00::00:35	6.09	6.09	6.09	11.35
00::00:36	6.10	6.10	6.10	11.38
00::00:37	6.14	6.14	6.14	11.44
00::00:38	6.26	6.26	6.26	11.67
00::00:39	6.26	6.26	6.26	11.67
00::00:40	6.34	6.34	6.34	11.82
00::00:41	6.42	6.42	6.42	11.96
00::00:42	6.42	6.42	6.42	11.96
00::00:43	6.54	6.54	6.54	12.20
00::00:44	6.57	6.57	6.57	12.25
00::00:45	6.59	6.59	6.59	12.28
00::00:46	6.73	6.73	6.73	12.55
00::00:47	6.73	6.73	6.73	12.55
00::00:48	6.79	6.79	6.79	12.66
00::00:49	6.89	6.89	6.89	12.84
00::00:50	6.89	6.89	6.89	12.84
00::00:51	7.00	7.00	7.00	13.04
00::00:52	7.04	7.04	7.04	13.13
00::00:53	7.04	7.04	7.04	13.13
00::00:54	7.20	7.20	7.20	13.42
00::00:55	7.20	7.20	7.20	13.42
00::00:56	7.29	7.29	7.29	13.60
00::00:57	7.51	7.51	7.51	14.00
00::00:58	7.51	7.51	7.51	14.00
00::00:59	7.61	7.61	7.61	14.18
00::01:00	7.67	7.67	7.67	14.30
00::01:01	7.67	7.67	7.67	14.30
00::01:02	7.95	7.95	7.95	14.82
00::01:03	7.98	7.98	7.98	14.88
00::01:04	8.01	8.01	8.01	14.94
00::01:05	8.14	8.14	8.14	15.17

00::01:06	8.14	8.14	8.14	15.17
00::01:07	8.30	8.30	8.30	15.46
00::01:08	8.45	8.45	8.45	15.75
00::01:09	8.45	8.45	8.45	15.75
00::01:10	8.70	8.70	8.70	16.22
00::01:11	8.77	8.77	8.77	16.34
00::01:12	8.80	8.80	8.80	16.40
00::01:13	9.08	9.08	9.08	16.92
00::01:14	9.08	9.08	9.08	16.92
00::01:15	9.27	9.27	9.27	17.27
00::01:16	9.55	9.55	9.55	17.80
00::01:17	9.55	9.55	9.55	17.80
00::01:18	9.88	9.88	9.88	18.41
00::01:19	10.02	10.02	10.02	18.67
00::01:20	10.02	10.02	10.02	18.67
00::01:21	10.64	10.64	10.64	19.84
00::01:22	10.64	10.64	10.64	19.84
00::01:23	10.83	10.83	10.83	20.19
00::01:24	11.27	11.27	11.27	21.01
00::01:25	11.27	11.27	11.27	21.01
00::01:26	11.83	11.83	11.83	22.06
00::01:27	12.21	12.21	12.21	22.76
00::01:28	12.21	12.21	12.21	22.76
00::01:29	13.05	13.05	13.05	24.33
00::01:30	13.15	13.15	13.15	24.51
00::01:31	13.37	13.37	13.37	24.92
00::01:32	14.24	14.24	14.24	26.55
00::01:33	14.24	14.24	14.24	26.55
00::01:34	14.87	14.87	14.87	27.72
00::01:35	15.50	15.50	15.50	28.88
00::01:36	15.50	15.50	15.50	28.88
00::01:37	16.87	16.87	16.87	31.45
00::01:38	17.22	17.22	17.22	32.09
00::01:39	17.42	17.42	17.42	32.47
00::01:40	19.25	19.25	19.25	35.88
00::01:41	19.25	19.25	19.25	35.89
00::01:42	20.07	20.07	20.07	37.40
00::01:43	21.29	21.29	21.29	39.68
00::01:44	21.29	21.29	21.29	39.68
00::01:45	23.04	23.04	23.04	42.94
00::01:46	23.79	23.79	23.79	44.35
00::01:47	23.79	23.79	23.79	44.35
00::01:48	26.61	26.61	26.61	49.60
00::01:49	26.61	26.61	26.61	49.60
00::01:50	27.45	27.45	27.45	51.17
00::01:51	29.43	29.43	29.43	54.85
00::01:52	29.43	29.43	29.43	54.85
00::01:53	31.30	31.30	31.30	58.35
00::01:54	32.56	32.56	32.56	60.68
00::01:55	32.56	32.56	32.56	60.68
00::01:56	35.37	35.37	35.37	65.93
00::01:57	35.69	35.69	35.69	66.52
00::01:58	36.38	36.38	36.38	67.80
00::01:59	39.13	39.13	39.13	72.94
00::02:00	39.13	39.13	39.13	72.94
00::02:01	41.01	41.01	41.01	76.44
00::02:02	42.89	42.89	42.89	79.94
00::02:03	42.89	42.89	42.89	79.94
00::02:04	46.14	46.14	46.14	86.01
00::02:05	46.96	46.96	46.96	87.52
00::02:06	47.40	47.40	47.40	88.34
00::02:07	51.34	51.34	51.34	95.69
00::02:08	51.34	51.34	51.34	95.69
00::02:09	53.22	53.22	53.22	99.19
00::02:10	56.04	56.04	56.04	104.45
00::02:11	56.04	56.04	56.04	104.44
00::02:12	59.76	59.76	59.76	111.39
00::02:13	61.36	61.36	61.36	114.37
00::02:14	61.36	61.36	61.36	114.36

00::02:15	61.36	61.36	61.36	114.36
00::02:16	61.36	61.36	61.36	114.37
00::02:17	59.76	59.76	59.76	111.39
00::02:18	56.04	56.04	56.04	104.44
00::02:19	56.04	56.04	56.04	104.45
00::02:20	53.22	53.22	53.22	99.19
00::02:21	51.34	51.34	51.34	95.69
00::02:22	51.34	51.34	51.34	95.69
00::02:23	47.40	47.40	47.40	88.34
00::02:24	46.96	46.96	46.96	87.52
00::02:25	46.14	46.14	46.14	86.01
00::02:26	42.89	42.89	42.89	79.94
00::02:27	42.89	42.89	42.89	79.94
00::02:28	41.01	41.01	41.01	76.44
00::02:29	39.13	39.13	39.13	72.94
00::02:30	39.13	39.13	39.13	72.94
00::02:31	36.38	36.38	36.38	67.80
00::02:32	35.69	35.69	35.69	66.52
00::02:33	35.37	35.37	35.37	65.93
00::02:34	32.56	32.56	32.56	60.68
00::02:35	32.56	32.56	32.56	60.68
00::02:36	31.30	31.30	31.30	58.35
00::02:37	29.43	29.43	29.43	54.85
00::02:38	29.43	29.43	29.43	54.85
00::02:39	27.45	27.45	27.45	51.17
00::02:40	26.61	26.61	26.61	49.60
00::02:41	26.61	26.61	26.61	49.60
00::02:42	23.79	23.79	23.79	44.35
00::02:43	23.79	23.79	23.79	44.35
00::02:44	23.04	23.04	23.04	42.94
00::02:45	21.29	21.29	21.29	39.68
00::02:46	21.29	21.29	21.29	39.68
00::02:47	20.07	20.07	20.07	37.40
00::02:48	19.25	19.25	19.25	35.89
00::02:49	19.25	19.25	19.25	35.88
00::02:50	17.42	17.42	17.42	32.47
00::02:51	17.22	17.22	17.22	32.09
00::02:52	16.87	16.87	16.87	31.45
00::02:53	15.50	15.50	15.50	28.88
00::02:54	15.50	15.50	15.50	28.88
00::02:55	14.87	14.87	14.87	27.72
00::02:56	14.24	14.24	14.24	26.55
00::02:57	14.24	14.24	14.24	26.55
00::02:58	13.37	13.37	13.37	24.92
00::02:59	13.15	13.15	13.15	24.51
00::03:00	13.05	13.05	13.05	24.33
00::03:01	12.21	12.21	12.21	22.76
00::03:02	12.21	12.21	12.21	22.76
00::03:03	11.83	11.83	11.83	22.06
00::03:04	11.27	11.27	11.27	21.01
00::03:05	11.27	11.27	11.27	21.01
00::03:06	10.83	10.83	10.83	20.19
00::03:07	10.64	10.64	10.64	19.84
00::03:08	10.64	10.64	10.64	19.84
00::03:09	10.02	10.02	10.02	18.67
00::03:10	10.02	10.02	10.02	18.67
00::03:11	9.88	9.88	9.88	18.41
00::03:12	9.55	9.55	9.55	17.80
00::03:13	9.55	9.55	9.55	17.80
00::03:14	9.27	9.27	9.27	17.27
00::03:15	9.08	9.08	9.08	16.92
00::03:16	9.08	9.08	9.08	16.92
00::03:17	8.80	8.80	8.80	16.40
00::03:18	8.77	8.77	8.77	16.34
00::03:19	8.70	8.70	8.70	16.22
00::03:20	8.45	8.45	8.45	15.75
00::03:21	8.45	8.45	8.45	15.75
00::03:22	8.30	8.30	8.30	15.46
00::03:23	8.14	8.14	8.14	15.17

00::03:24	8.14	8.14	8.14	15.17
00::03:25	8.01	8.01	8.01	14.94
00::03:26	7.98	7.98	7.98	14.88
00::03:27	7.95	7.95	7.95	14.82
00::03:28	7.67	7.67	7.67	14.30
00::03:29	7.67	7.67	7.67	14.30
00::03:30	7.61	7.61	7.61	14.18
00::03:31	7.51	7.51	7.51	14.00
00::03:32	7.51	7.51	7.51	14.00
00::03:33	7.29	7.29	7.29	13.60
00::03:34	7.20	7.20	7.20	13.42
00::03:35	7.20	7.20	7.20	13.42
00::03:36	7.04	7.04	7.04	13.13
00::03:37	7.04	7.04	7.04	13.13
00::03:38	7.00	7.00	7.00	13.04
00::03:39	6.89	6.89	6.89	12.84
00::03:40	6.89	6.89	6.89	12.84
00::03:41	6.79	6.79	6.79	12.66
00::03:42	6.73	6.73	6.73	12.55
00::03:43	6.73	6.73	6.73	12.55
00::03:44	6.59	6.59	6.59	12.28
00::03:45	6.57	6.57	6.57	12.25
00::03:46	6.54	6.54	6.54	12.20
00::03:47	6.42	6.42	6.42	11.96
00::03:48	6.42	6.42	6.42	11.96
00::03:49	6.34	6.34	6.34	11.82
00::03:50	6.26	6.26	6.26	11.67
00::03:51	6.26	6.26	6.26	11.67
00::03:52	6.14	6.14	6.14	11.44
00::03:53	6.10	6.10	6.10	11.38
00::03:54	6.09	6.09	6.09	11.35
00::03:55	5.95	5.95	5.95	11.09
00::03:56	5.95	5.95	5.95	11.09
00::03:57	5.95	5.95	5.95	11.09
00::03:58	5.95	5.95	5.95	11.09
00::03:59	5.95	5.95	5.95	11.09
00::04:00	5.84	5.84	5.84	10.88
00::04:01	5.79	5.79	5.79	10.79
00::04:02	5.79	5.79	5.79	10.79
00::04:03	5.79	5.79	5.79	10.79
00::04:04	5.79	5.79	5.79	10.79
00::04:05	5.74	5.74	5.74	10.71
00::04:06	5.63	5.63	5.63	10.50
00::04:07	5.63	5.63	5.63	10.50
00::04:08	5.63	5.63	5.63	10.50
00::04:09	5.63	5.63	5.63	10.50
00::04:10	5.63	5.63	5.63	10.50
00::04:11	5.49	5.49	5.49	10.24
00::04:12	5.48	5.48	5.48	10.21
00::04:13	5.48	5.48	5.48	10.21
00::04:14	5.48	5.48	5.48	10.21
00::04:15	5.48	5.48	5.48	10.21
00::04:16	5.40	5.40	5.40	10.07
00::04:17	5.32	5.32	5.32	9.92
00::04:18	5.32	5.32	5.32	9.92
00::04:19	5.32	5.32	5.32	9.92
00::04:20	5.32	5.32	5.32	9.92
00::04:21	5.31	5.31	5.31	9.89
00::04:22	5.17	5.17	5.17	9.63
00::04:23	5.17	5.17	5.17	9.63
00::04:24	5.17	5.17	5.17	9.63
00::04:25	5.17	5.17	5.17	9.63
00::04:26	5.17	5.17	5.17	9.63
00::04:27	5.06	5.06	5.06	9.42
00::04:28	5.01	5.01	5.01	9.34
00::04:29	5.01	5.01	5.01	9.34
T	1.00	1+20%	1+40%	0.10
00::00:00	5.01	5.01	5.01	9.34
00::00:01	5.01	5.01	5.01	9.34

00::00:02	5.06	5.06	5.06	9.42
00::00:03	5.17	5.17	5.17	9.63
00::00:04	5.17	5.17	5.17	9.63
00::00:05	5.17	5.17	5.17	9.63
00::00:06	5.17	5.17	5.17	9.63
00::00:07	5.17	5.17	5.17	9.63
00::00:08	5.31	5.31	5.31	9.89
00::00:09	5.32	5.32	5.32	9.92
00::00:10	5.32	5.32	5.32	9.92
00::00:11	5.32	5.32	5.32	9.92
00::00:12	5.32	5.32	5.32	9.92
00::00:13	5.40	5.40	5.40	10.07
00::00:14	5.48	5.48	5.48	10.21
00::00:15	5.48	5.48	5.48	10.21
00::00:16	5.48	5.48	5.48	10.21
00::00:17	5.48	5.48	5.48	10.21
00::00:18	5.49	5.49	5.49	10.24
00::00:19	5.63	5.63	5.63	10.50
00::00:20	5.63	5.63	5.63	10.50
00::00:21	5.63	5.63	5.63	10.50
00::00:22	5.63	5.63	5.63	10.50
00::00:23	5.63	5.63	5.63	10.50
00::00:24	5.74	5.74	5.74	10.71
00::00:25	5.79	5.79	5.79	10.79
00::00:26	5.79	5.79	5.79	10.79
00::00:27	5.79	5.79	5.79	10.79
00::00:28	5.79	5.79	5.79	10.79
00::00:29	5.84	5.84	5.84	10.88
00::00:30	5.95	5.95	5.95	11.09
00::00:31	5.95	5.95	5.95	11.09
00::00:32	5.95	5.95	5.95	11.09
00::00:33	5.95	5.95	5.95	11.09
00::00:34	5.95	5.95	5.95	11.09
00::00:35	6.09	6.09	6.09	11.35
00::00:36	6.10	6.10	6.10	11.38
00::00:37	6.14	6.14	6.14	11.44
00::00:38	6.26	6.26	6.26	11.67
00::00:39	6.26	6.26	6.26	11.67
00::00:40	6.34	6.34	6.34	11.82
00::00:41	6.42	6.42	6.42	11.96
00::00:42	6.42	6.42	6.42	11.96
00::00:43	6.54	6.54	6.54	12.20
00::00:44	6.57	6.57	6.57	12.25
00::00:45	6.59	6.59	6.59	12.28
00::00:46	6.73	6.73	6.73	12.55
00::00:47	6.73	6.73	6.73	12.55
00::00:48	6.79	6.79	6.79	12.66
00::00:49	6.89	6.89	6.89	12.84
00::00:50	6.89	6.89	6.89	12.84
00::00:51	7.00	7.00	7.00	13.04
00::00:52	7.04	7.04	7.04	13.13
00::00:53	7.04	7.04	7.04	13.13
00::00:54	7.20	7.20	7.20	13.42
00::00:55	7.20	7.20	7.20	13.42
00::00:56	7.29	7.29	7.29	13.60
00::00:57	7.51	7.51	7.51	14.00
00::00:58	7.51	7.51	7.51	14.00
00::00:59	7.61	7.61	7.61	14.18
00::01:00	7.67	7.67	7.67	14.30
00::01:01	7.67	7.67	7.67	14.30
00::01:02	7.95	7.95	7.95	14.82
00::01:03	7.98	7.98	7.98	14.88
00::01:04	8.01	8.01	8.01	14.94
00::01:05	8.14	8.14	8.14	15.17
00::01:06	8.14	8.14	8.14	15.17
00::01:07	8.30	8.30	8.30	15.46
00::01:08	8.45	8.45	8.45	15.75
00::01:09	8.45	8.45	8.45	15.75
00::01:10	8.70	8.70	8.70	16.22

00::01:11	8.77	8.77	8.77	16.34
00::01:12	8.80	8.80	8.80	16.40
00::01:13	9.08	9.08	9.08	16.92
00::01:14	9.08	9.08	9.08	16.92
00::01:15	9.27	9.27	9.27	17.27
00::01:16	9.55	9.55	9.55	17.80
00::01:17	9.55	9.55	9.55	17.80
00::01:18	9.88	9.88	9.88	18.41
00::01:19	10.02	10.02	10.02	18.67
00::01:20	10.02	10.02	10.02	18.67
00::01:21	10.64	10.64	10.64	19.84
00::01:22	10.64	10.64	10.64	19.84
00::01:23	10.83	10.83	10.83	20.19
00::01:24	11.27	11.27	11.27	21.01
00::01:25	11.27	11.27	11.27	21.01
00::01:26	11.83	11.83	11.83	22.06
00::01:27	12.21	12.21	12.21	22.76
00::01:28	12.21	12.21	12.21	22.76
00::01:29	13.05	13.05	13.05	24.33
00::01:30	13.15	13.15	13.15	24.51
00::01:31	13.37	13.37	13.37	24.92
00::01:32	14.24	14.24	14.24	26.55
00::01:33	14.24	14.24	14.24	26.55
00::01:34	14.87	14.87	14.87	27.72
00::01:35	15.50	15.50	15.50	28.88
00::01:36	15.50	15.50	15.50	28.88
00::01:37	16.87	16.87	16.87	31.45
00::01:38	17.22	17.22	17.22	32.09
00::01:39	17.42	17.42	17.42	32.47
00::01:40	19.25	19.25	19.25	35.88
00::01:41	19.25	19.25	19.25	35.89
00::01:42	20.07	20.07	20.07	37.40
00::01:43	21.29	21.29	21.29	39.68
00::01:44	21.29	21.29	21.29	39.68
00::01:45	23.04	23.04	23.04	42.94
00::01:46	23.79	23.79	23.79	44.35
00::01:47	23.79	23.79	23.79	44.35
00::01:48	26.61	26.61	26.61	49.60
00::01:49	26.61	26.61	26.61	49.60
00::01:50	27.45	27.45	27.45	51.17
00::01:51	29.43	29.43	29.43	54.85
00::01:52	29.43	29.43	29.43	54.85
00::01:53	31.30	31.30	31.30	58.35
00::01:54	32.56	32.56	32.56	60.68
00::01:55	32.56	32.56	32.56	60.68
00::01:56	35.37	35.37	35.37	65.93
00::01:57	35.69	35.69	35.69	66.52
00::01:58	36.38	36.38	36.38	67.80
00::01:59	39.13	39.13	39.13	72.94
00::02:00	39.13	39.13	39.13	72.94
00::02:01	41.01	41.01	41.01	76.44
00::02:02	42.89	42.89	42.89	79.94
00::02:03	42.89	42.89	42.89	79.94
00::02:04	46.14	46.14	46.14	86.01
00::02:05	46.96	46.96	46.96	87.52
00::02:06	47.40	47.40	47.40	88.34
00::02:07	51.34	51.34	51.34	95.69
00::02:08	51.34	51.34	51.34	95.69
00::02:09	53.22	53.22	53.22	99.19
00::02:10	56.04	56.04	56.04	104.45
00::02:11	56.04	56.04	56.04	104.44
00::02:12	59.76	59.76	59.76	111.39
00::02:13	61.36	61.36	61.36	114.37
00::02:14	61.36	61.36	61.36	114.36
00::02:15	61.36	61.36	61.36	114.36
00::02:16	61.36	61.36	61.36	114.37
00::02:17	59.76	59.76	59.76	111.39
00::02:18	56.04	56.04	56.04	104.44
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00::02:20	53.22	53.22	53.22	99.19
00::02:21	51.34	51.34	51.34	95.69
00::02:22	51.34	51.34	51.34	95.69
00::02:23	47.40	47.40	47.40	88.34
00::02:24	46.96	46.96	46.96	87.52
00::02:25	46.14	46.14	46.14	86.01
00::02:26	42.89	42.89	42.89	79.94
00::02:27	42.89	42.89	42.89	79.94
00::02:28	41.01	41.01	41.01	76.44
00::02:29	39.13	39.13	39.13	72.94
00::02:30	39.13	39.13	39.13	72.94
00::02:31	36.38	36.38	36.38	67.80
00::02:32	35.69	35.69	35.69	66.52
00::02:33	35.37	35.37	35.37	65.93
00::02:34	32.56	32.56	32.56	60.68
00::02:35	32.56	32.56	32.56	60.68
00::02:36	31.30	31.30	31.30	58.35
00::02:37	29.43	29.43	29.43	54.85
00::02:38	29.43	29.43	29.43	54.85
00::02:39	27.45	27.45	27.45	51.17
00::02:40	26.61	26.61	26.61	49.60
00::02:41	26.61	26.61	26.61	49.60
00::02:42	23.79	23.79	23.79	44.35
00::02:43	23.79	23.79	23.79	44.35
00::02:44	23.04	23.04	23.04	42.94
00::02:45	21.29	21.29	21.29	39.68
00::02:46	21.29	21.29	21.29	39.68
00::02:47	20.07	20.07	20.07	37.40
00::02:48	19.25	19.25	19.25	35.89
00::02:49	19.25	19.25	19.25	35.88
00::02:50	17.42	17.42	17.42	32.47
00::02:51	17.22	17.22	17.22	32.09
00::02:52	16.87	16.87	16.87	31.45
00::02:53	15.50	15.50	15.50	28.88
00::02:54	15.50	15.50	15.50	28.88
00::02:55	14.87	14.87	14.87	27.72
00::02:56	14.24	14.24	14.24	26.55
00::02:57	14.24	14.24	14.24	26.55
00::02:58	13.37	13.37	13.37	24.92
00::02:59	13.15	13.15	13.15	24.51
00::03:00	13.05	13.05	13.05	24.33
00::03:01	12.21	12.21	12.21	22.76
00::03:02	12.21	12.21	12.21	22.76
00::03:03	11.83	11.83	11.83	22.06
00::03:04	11.27	11.27	11.27	21.01
00::03:05	11.27	11.27	11.27	21.01
00::03:06	10.83	10.83	10.83	20.19
00::03:07	10.64	10.64	10.64	19.84
00::03:08	10.64	10.64	10.64	19.84
00::03:09	10.02	10.02	10.02	18.67
00::03:10	10.02	10.02	10.02	18.67
00::03:11	9.88	9.88	9.88	18.41
00::03:12	9.55	9.55	9.55	17.80
00::03:13	9.55	9.55	9.55	17.80
00::03:14	9.27	9.27	9.27	17.27
00::03:15	9.08	9.08	9.08	16.92
00::03:16	9.08	9.08	9.08	16.92
00::03:17	8.80	8.80	8.80	16.40
00::03:18	8.77	8.77	8.77	16.34
00::03:19	8.70	8.70	8.70	16.22
00::03:20	8.45	8.45	8.45	15.75
00::03:21	8.45	8.45	8.45	15.75
00::03:22	8.30	8.30	8.30	15.46
00::03:23	8.14	8.14	8.14	15.17
00::03:24	8.14	8.14	8.14	15.17
00::03:25	8.01	8.01	8.01	14.94
00::03:26	7.98	7.98	7.98	14.88
00::03:27	7.95	7.95	7.95	14.82
00::03:28	7.67	7.67	7.67	14.30

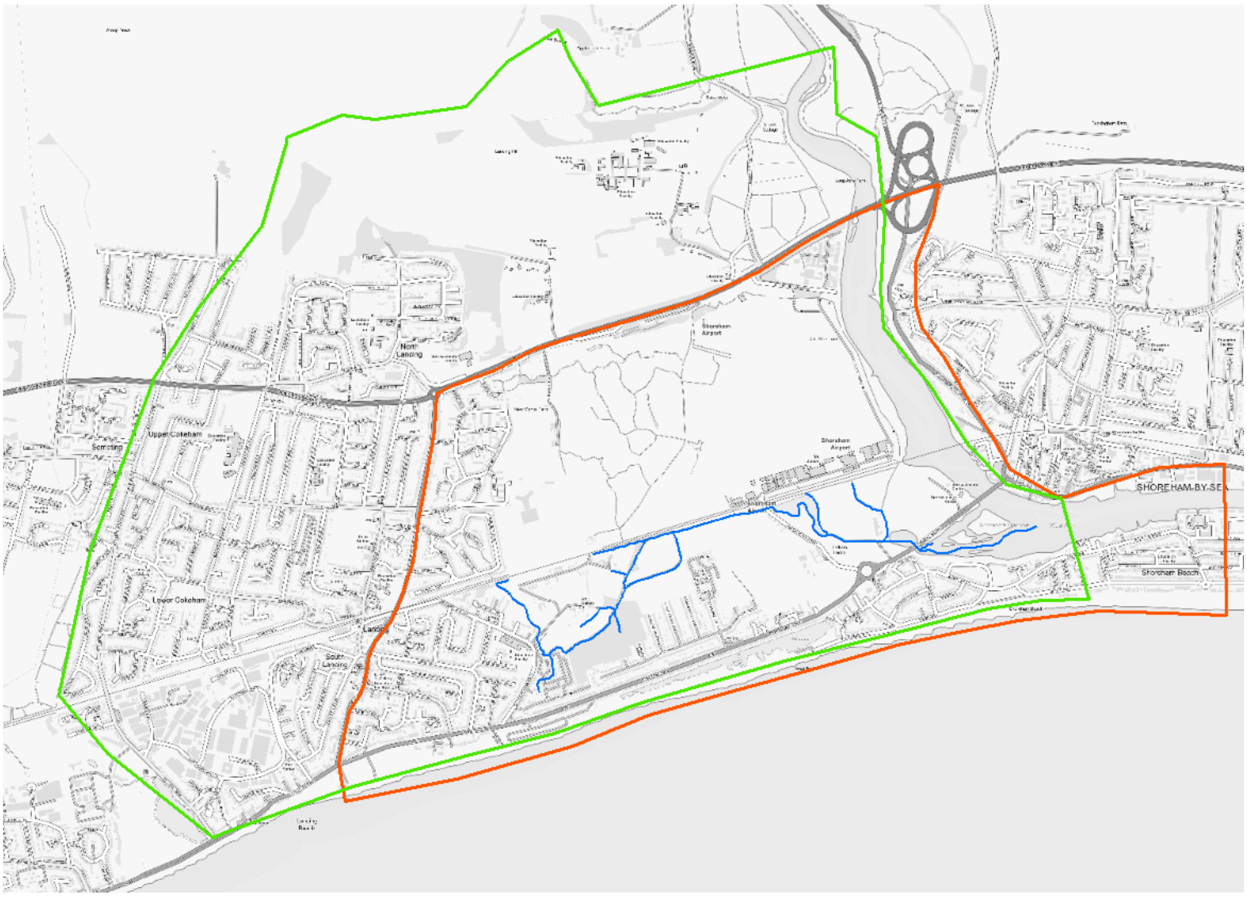
00::03:29	7.67	7.67	7.67	14.30
00::03:30	7.61	7.61	7.61	14.18
00::03:31	7.51	7.51	7.51	14.00
00::03:32	7.51	7.51	7.51	14.00
00::03:33	7.29	7.29	7.29	13.60
00::03:34	7.20	7.20	7.20	13.42
00::03:35	7.20	7.20	7.20	13.42
00::03:36	7.04	7.04	7.04	13.13

B Sensitivity test on contributing area

To test the assumption of contributing area for the direct rainfall approach a larger extent model was developed. The design model domain is shown on the figure below in red. The sensitivity model domain is shown in green. The Design domain was increased in area from 6.7km^2 to approximately 11.6km$^{2.}$ Both the design model domain and the sensitivity model domain were run 2D only (without any embedded 1D channels or culverts) for the 1:100 AEP event with runoff based on SPRHOST throughout the active domain.

This sensitivity test was undertaken to identify flow paths into the contributing area that may not have been accounted for within the original design domain.

Figure B1 – Design domain and sensitivity domain.



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As can be seen in Figure B2 below four flow paths have been identified. Two of these (shown in purple) have been identified to drain outside of the previous domain. Two flow paths (shown in red) have been identified to drain to inside the previous domain.

Of these, the southern flow path flowing Monks Avenue, has potential to contribute a small volume of additional flow to the top of Brook in this location. This brook drains to the south and then passes east across the study site. An enlargement of the flow path can be found in Figure B3. Given the small area concerned and the noted presence of urban drainage infrastructure in this area connected to soakaways (Chapter 5 of Surface Water Management Plan) it is considered that the additional volume of flow this are may contribute to flooding at the site to be negligible. It is recommended that the results of sensitivity test to runoff coefficient should be reviewed to allow model results users to understand the probable outcomes of increase flood volumes.

The second red flow path runs parallel to the south of the A27 carriageway. This flow path runs east and crosses Marsh Barn Lane where it would be picked up by the network of brooks to the west of Shoreham Airport. These brooks continue to drain east and outfall to the estuary. They do not flow though study site, therefore their omission from the model domain is considered immaterial.

Figure B2 – Sensitivity test results.

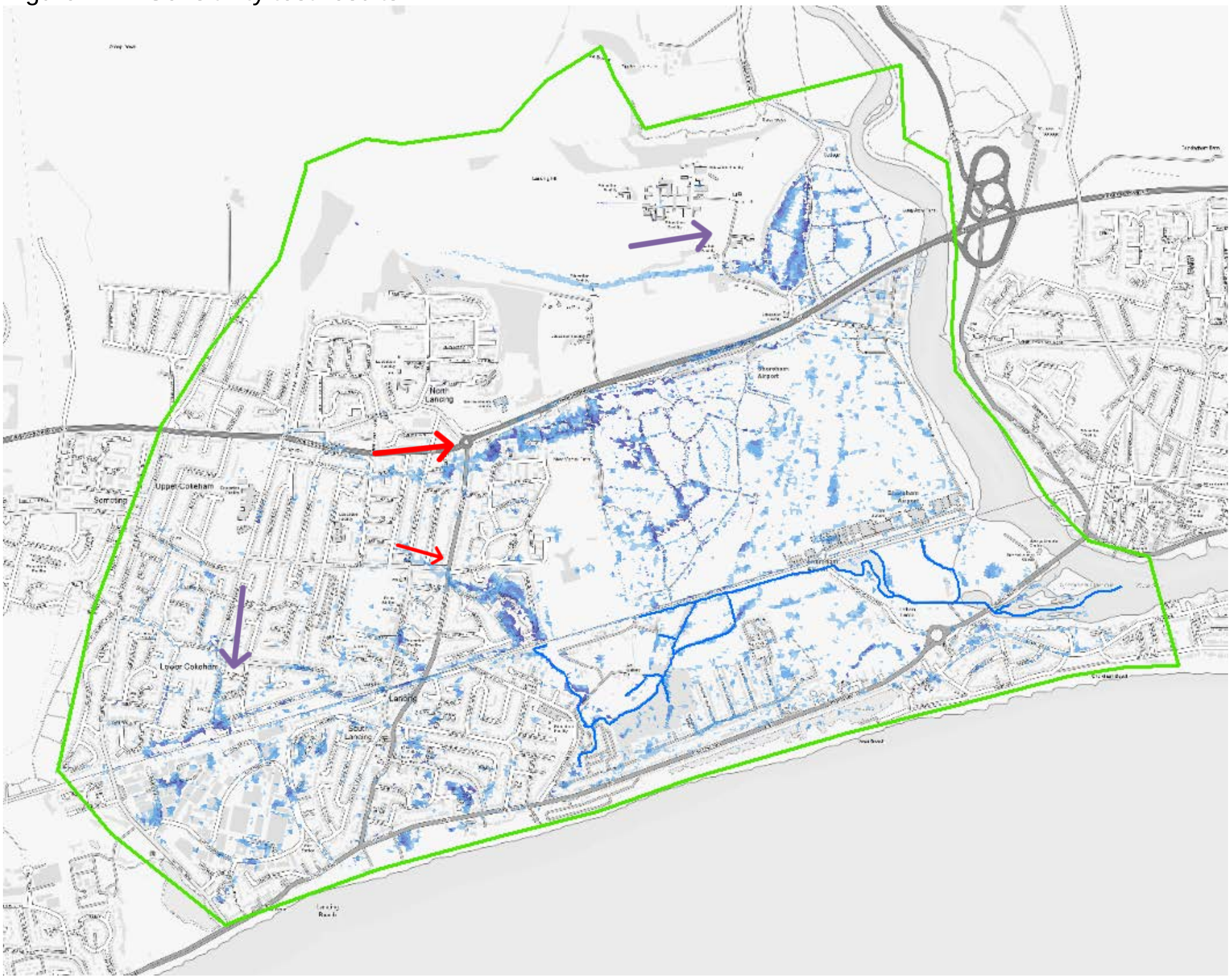

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Figure B3 – Monks Avenue Flow path

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Figure B4 – A27 Flow path

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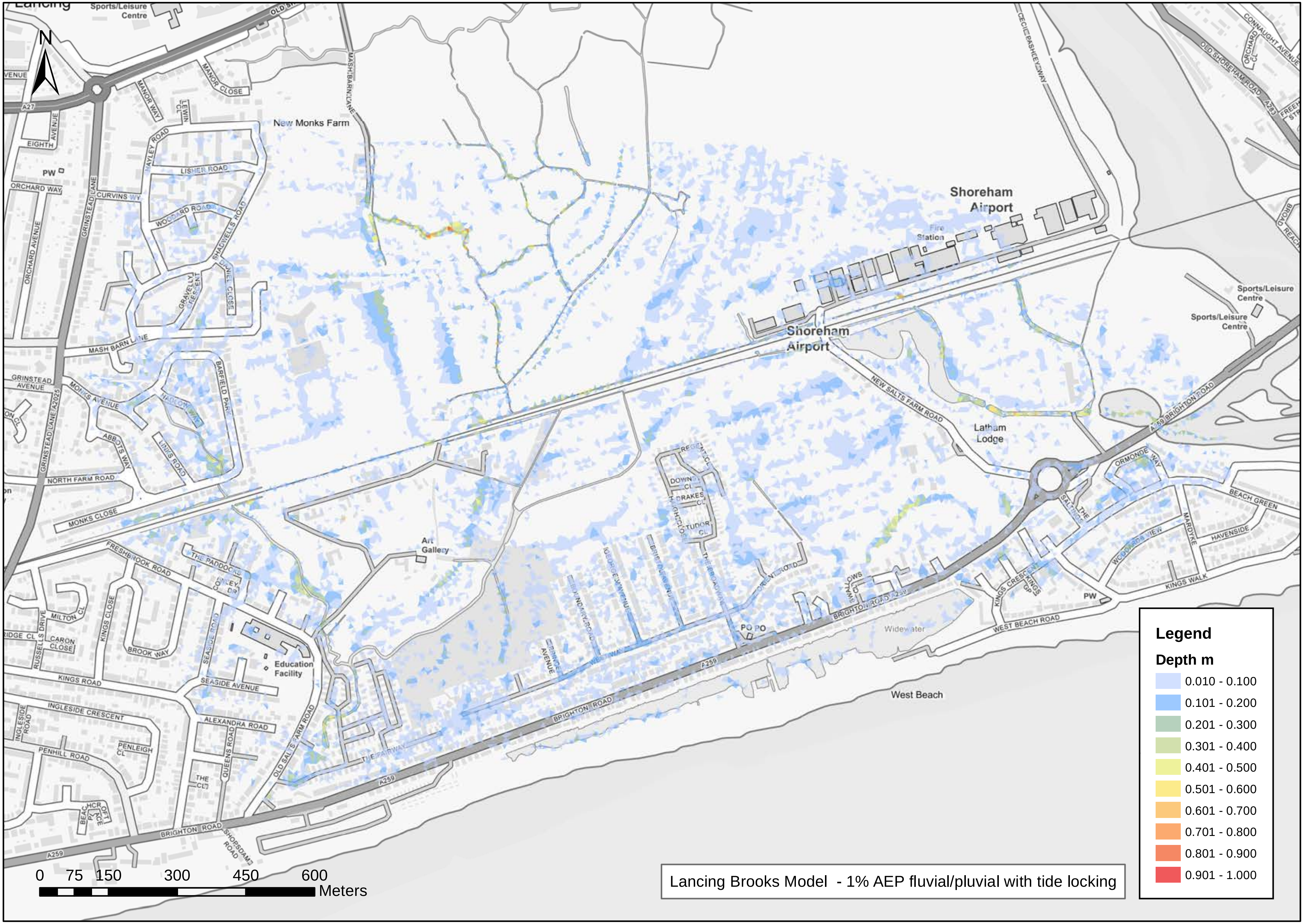
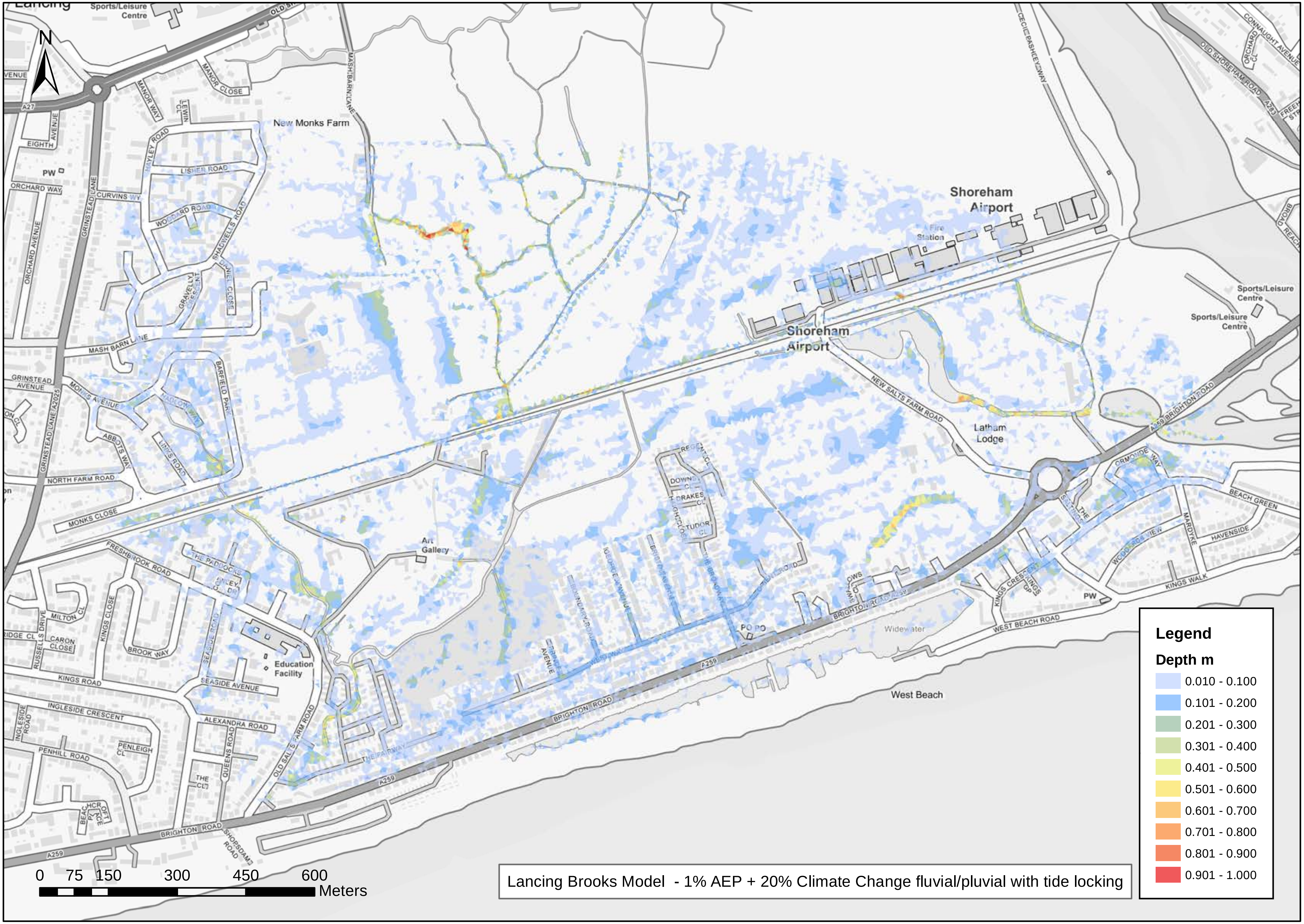
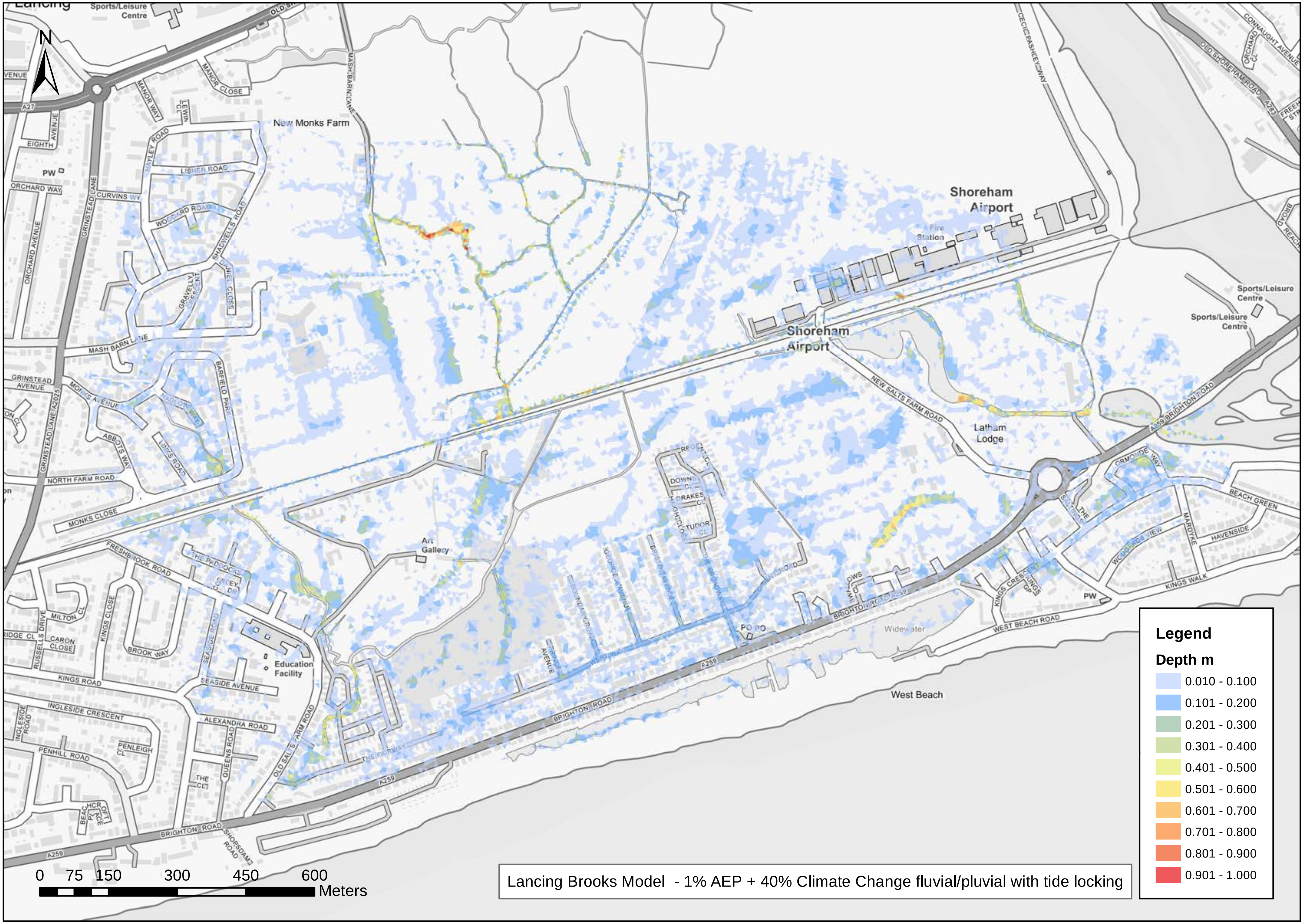

Lancing Brooks Model - 1% AEP fluvial/pluvial with tide locking
Legend
Depth m
0.010 - 0.100
0.101 - 0.200
0.201 - 0.300
0.301 - 0.400
0.401 - 0.500
0.501 - 0.600
0.601 - 0.700
0.701 - 0.800
0.801 - 0.900
0.901 - 1.000
0 75 150 300 450 600
Meters
N
New Monks Farm
Shoreham Airport
Latham Lodge
Widewater
West Beach
Education Facility
Art Gallery
Sports/Leisure Centre
BRIGHTON ROAD
A259
A27



Shoreham Airport
New Monks Farm
Latham Lodge
West Beach
Widewater
Art Gallery
Education Facility
Legend
Depth m
0.010 - 0.100
0.101 - 0.200
0.201 - 0.300
0.301 - 0.400
0.401 - 0.500
0.501 - 0.600
0.601 - 0.700
0.701 - 0.800
0.801 - 0.900
0.901 - 1.000
0 75 150 300 450 600
Meters
Lancing Brooks Model - 1% AEP + 20% Climate Change fluvial/pluvial with tide locking


N
New Monks Farm
Shoreham Airport
Shoreham Airport
Fire Station
Latham Lodge
Art Gallery
Education Facility
Widewater
West Beach
Sports/Leisure Centre
Sports/Leisure Centre
MASH BARN LANE
NORTH FARM ROAD
MONKS CLOSE
FRESHBROOK ROAD
KINGS ROAD
INGLESIDE CRESCENT
PENHILL ROAD
ALEXANDRA ROAD
SEASIDE AVENUE
OLD SALTS FARM ROAD
NEW SALTS FARM ROAD
BRIGHTON ROAD
A259
A27
GRINSTEAD LANE/A2025
WEST BEACH ROAD
KINGS WALK
BEACH GREEN
HAVENSIDE
OLD SHOREHAM ROAD
THE FAIRWAY
SHOPSDAM ROAD
0 75 150 300 450 600
Meters
Lancing Brooks Model - 1% AEP + 40% Climate Change fluvial/pluvial with tide locking
Legend
Depth m
0.010 - 0.100
0.101 - 0.200
0.201 - 0.300
0.301 - 0.400
0.401 - 0.500
0.501 - 0.600
0.601 - 0.700
0.701 - 0.800
0.801 - 0.900
0.901 - 1.000

New Monks Farm
Shoreham Airport
Fire Station
Latham Lodge
Art Gallery
Education Facility
Widewater
West Beach
Sports/Leisure Centre
N
0 75 150 300 450 600 Meters
Legend
Depth m
0.010 - 0.100
0.101 - 0.200
0.201 - 0.300
0.301 - 0.400
0.401 - 0.500
0.501 - 0.600
0.601 - 0.700
0.701 - 0.800
0.801 - 0.900
0.901 - 1.000
Lancing Brooks Model - 0.1% AEP fluvial/pluvial with tide locking

Appendix R – Correspondence from The EA, LLFA, Adur and BRE

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5 January 2017

References:

A. Ben Daines (Adur & Worthing Councils) email to West Sussex CC LLFA of 4 Jan 16.
B. Tully De'Ath (Dec 2016) Flood Risk Assessment Issue 4- New Salts Farm – Shoreham 11649 For The Hyde Group.
C. Ray Drabble (West Sussex CC LLFA) email to Ben Daines of 19 Dec 16.

Dear Ben,

Pre-Application Query: Tully De'Ath Flood Risk Assessment Issue 4- New Salts Farm – Shoreham 11649 For The Hyde Group

West Sussex County Council (WSCC) in its statutory capacity as Lead Local Flood Authority (LLFA) has been approached by Adur & Worthing Council (Reference A) for comments on the above.

West Sussex CC LLFA has reviewed the above document and this letter summarises our comments.

General

The stated purpose of the Flood Risk Assessment is to demonstrate to the Planners, the Environment Agency (EA) and West Sussex County Council as Lead Local Flood Authority that the proposed development is subject to an acceptable level of flood risk and should not increase the likelihood of flooding elsewhere (paragraph 1.2).

It achieves neither of these objectives as no detail has been provided of the catchment extent, greenfield run-off and post development run-off. This evidence is needed to demonstrate capacity in the drainage network commensurate with acceptable levels of flood risk to both the proposed development and to existing residents in the same drainage catchment. Recognising the particular constraints on the drainage network that are characterised in the Lancing SWMP and Reference C, a full quantitative assessment of the pre and post-development run-off rates is needed to evaluate the depths of water for design storm events, consistent with the non-statutory technical standards.

The analyses will also need to take into consideration the effects both of tide-locking on the Lancing Brooks outfall and the tidal effects upon groundwater levels, both referred to in the FRA. Analyses should cover the full extent of the Lancing Brooks Catchment (Figure 3-2 of the SWMP) and, therefore, the post-development scenario for New Monks Farm that also feeds into the drainage network.

The interconnectivity of the Lancing Brooks and catchment-wide effects of tide-locking on the outfall reinforces the need for an FRA that considers the whole drainage catchment (as recommended to Tully De'Ath staff in summer 2016 by the EA). It is also consistent with best practice for phased developments set out in paragraph 7.2.4 of the SuDS Manual.

Specific Comments on text

Paragraph 3.6 Groundwater

West Sussex County Council Lead Local Flood Authority (WSCC LLFA) is concerned that the data provided for boreholes WLS 107-WLS 111 is not representative of typical overwinter conditions, having been taken between Sep 2016 and Nov 2016 when rainfall was below average for much of UK including the south. Given that infiltration is under consideration as part of the drainage strategy for New Salts farm, infiltration testing should be consistent with recommendations in BRE 365.

Paragraph 4.1 – Development Proposals

The proposals for green roofs with integral attenuation below (Blue Roof) is innovative and welcome but the attenuation & storage provided by this and other components of the proposed drainage needs to be calculated to demonstrate that greenfield run-off rates can be maintained.

The use of permeable paving for roads, parking courts and hard paved areas is considered to provide negligible benefit for critical groundwater and surface water flooding events, given the high groundwater levels associated with much of the development area.

5.3 Lancing Brooks Modelling

This paragraph makes reference to Appendix Q, and modelling of both the 1 in 100+40% CC and the 1 in 1000-year return periods. Appendix Q only contains output images; without the methodology, catchment assessed, input values and other key parameters used to generate the model outputs, they are of no value to the assessment.

Section 6.0 Flood Management and Mitigation

6.3 Floor Levels

WSCC LLFA is concerned at the allowance of only 0.3m freeboard between ground floor level and the existing ground surface given the high risk of ground water / surface water flooding over parts of the site.

6.5 Surface Water Run-off Rates

Consistent with the recommendations in the Lancing SWMP, WSCC LLFA would advise that infiltration into the subsoils is not an appropriate drainage measure for the site. It is unclear how / where water will be attenuated upstream of flow control devices; clarification should be sought.

6.8 Safe Access and Egress

This states*: it may not be possible to provide a dry means of escape from the buildings in the event of a flood. To overcome this the units will have direct access to the first floor, which will be the primary area for refuge in the event of a major costal flood event*.

The above statement is not consistent with our interpretation of NPPF paragraph 103. If this response is to form part of the flood evacuation plan for all properties within New Salts Farm, how are disabled residents being accommodated for?

6.10 Foul Sewers

Confirmation needs to be sought by A&WC from Southern Water whether the proposed infrastructure measures outlined in Appendix C will provide sufficient capacity to address the proposed new development, existing sewerage flooding issues and take into consideration the Southern Water measures proposed to accommodate the allocation of 600 homes at New Monks Farm. Specifically, confirmation from Southern Water should be sought that they have strategically reviewed all the proposed development affecting this sewerage catchment and can provide the capacity needed within the timescale that development is being brought forward.

No reference is made to the issue of groundwater inundation of the foul system and we assess this as being a significant risk. Inundation may result from hydrostatic pressure on the existing foul drains as well as excessive surface water draining down through any existing or new foul sewer manholes. This risk needs to be considered and appropriate mitigation put in place.

13.0 Conclusion

The conclusion states that: *the main flood risk for the development is associated with coastal flooding.* This statement and the tenor of the FRA, as a whole, in our view underplay the significant flood risk from groundwater and surface water.

The site conditions are summarised in paragraph 3.4.2 of the Lancing SWMP that states:

Under conditions of high winter recharge and elevated groundwater levels in the Chalk and in response to upward groundwater pressure from the underlying

Chalk, there may be upward leakage from the Chalk to the upper aquifer and surface water. This occurs through more permeable windows in the Superficial Deposits... Where there is partial connectivity between the two aquifers, the upper alluvial aquifer may become more permanently saturated, leading to areas of marshy ground. These mechanisms are most likely to occur in the southern part of the study area.

It is these conditions of saturated ground water in combination with prolonged rainfall that present a significant issue for surface water drainage for the site and for existing residences in adjacent areas that rely upon the same drainage network. The FRA has not demonstrated that surface water under these conditions can be adequately drained without causing increased flood risk to existing residents in the catchment or risking flooding of the proposed residential properties.

Yours sincerely

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West Sussex Lead Local Flood Authority

Copies to: Adrian Jackson, Environment Agency

Internal: Caroline West, West Sussex County Council

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5 January 2017

References:

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C. Ray Drabble (West Sussex CC LLFA) email to Ben Daines of 19 Dec 16.

Dear Ben,

Pre-Application Query: Tully De'Ath Flood Risk Assessment Issue 4- New Salts Farm – Shoreham 11649 For The Hyde Group

West Sussex County Council (WSCC) in its statutory capacity as Lead Local Flood Authority (LLFA) has been approached by Adur & Worthing Council (Reference A) for comments on the above.

West Sussex CC LLFA has reviewed the above document and this letter summarises our comments.

General

The stated purpose of the Flood Risk Assessment is to demonstrate to the Planners, the Environment Agency (EA) and West Sussex County Council as Lead Local Flood Authority that the proposed development is subject to an acceptable level of flood risk and should not increase the likelihood of flooding elsewhere (paragraph 1.2).

It achieves neither of these objectives as no detail has been provided of the catchment extent, greenfield run-off and post development run-off. This evidence is needed to demonstrate capacity in the drainage network commensurate with acceptable levels of flood risk to both the proposed development and to existing residents in the same drainage catchment. Recognising the particular constraints on the drainage network that are characterised in the Lancing SWMP and Reference C, a full quantitative assessment of the pre and post-development run-off rates is needed to evaluate the depths of water for design storm events, consistent with the non-statutory technical standards.

The existing site is greenfield which currently infiltrates and discharges into the adjacent Lancing Brook system, depending on upon differing ground water scenarios. For the new development this will be replicated, with infiltration as the most representative existing method of surface water disposal. Where this is not possible discharge will be into the existing ditch network. The discharge into the ground will be confirmed via infiltration testing and run-off into the ditches will be controlled to replicate green-field run-off rates for a range of return periods. This requirement can be imposed as of a condition of planning.

The analyses will also need to take into consideration the effects both of tide-locking on the Lancing Brooks outfall and the tidal effects upon groundwater levels, both referred to in the FRA. Analyses should cover the full extent of the Lancing Brooks Catchment (Figure 3-2 of the SWMP) and, therefore, the post-development scenario for New Monks Farm that also feeds into the drainage network.

Tide locking and high ground water levels have been considered in the JBA assessment of the Lancing ditches. Additional updated modelling has been recently completed and the results are discussed later in this letter.

The Monks Farm development is located to the north of this site and discharges into the main Lancing Brook channel downstream the NSF site. It is assumed that the EA and the LLFA will impose greenfield run-off rates to the Monks Farm development, and as a consequence would have an insignificant effect on our analysis.

The interconnectivity of the Lancing Brooks and catchment-wide effects of tide-locking on the outfall reinforces the need for an FRA that considers the whole drainage catchment (as recommended to Tully De'Ath staff in summer 2016 by the EA). It is also consistent with best practice for phased developments set out in paragraph 7.2.4 of the SuDS Manual.

Within the NSF site there are two surface water disposal options available, infiltration and discharging to the Lancing ditches. For the majority of the time both methods will be used, however there will be occasions where infiltration will the principle method of disposal, and periods when discharging to the ditch system will be appropriate. The drainage strategy developed accommodates both options. In addition, significant volumes of flood storage will be provided on the site when both options become limited. However, both discharge methods are tidal influenced and as a consequence one or both disposal methods are likely to be available during low tide scenarios. It is recognised that there can be infrequent exceedance events where both options may not available for a longer than normal period of time and mitigation measures in terms of the unit types are designed to accommodate this exceedance scenario.

Specific Comments on text

Paragraph 3.6 Groundwater

West Sussex County Council Lead Local Flood Authority (WSCC LLFA) is concerned that the data provided for boreholes WLS 107-WLS 111 is not representative of typical overwinter conditions, having been taken between Sep 2016 and Nov 2016 when rainfall was below average for much of UK including the south. Given that infiltration is under consideration as part of the drainage strategy for New Salts farm, infiltration testing should be consistent with recommendations in BRE 365.

WSL4,5 & 6 have been installed for 12 months. They demonstrate that shallow infiltration would be possible for this area of the site throughout the year, although it is recognised that infiltration near WSL5 should be avoided due to regular high ground water levels. The monitoring of the ground water across the site will continue for the foreseeable future and the results will be used to confirm the detailed design.

With regards to infiltration, the ground water levels rise and fall with the tide which therefore demonstrates that the ground is suitable for infiltration. We have also discussed the suitability of infiltration with Southern Testing (Soils Engineers) who confirmed that based upon the intrusive investigation works undertaken to date shallow infiltration via the likes of permeable paving would be effective. During extreme high ground water events (at surface level) where infiltration performs in a less efficient manner conveyance swales are provided to accommodate temporary additional water. As part of the detailed design a BRE365 infiltration test will be undertaken where infiltration is to be used.

It should be noted that when the ground levels are particularly high the infiltration system would be able to connect to the Lancing ditch system.

Paragraph 4.1 – Development Proposals

The proposals for green roofs with integral attenuation below (Blue Roof) is innovative and welcome but the attenuation & storage provided by this and other components of the proposed drainage needs to be calculated to demonstrate that greenfield run-off rates can be maintained.

Generally, infiltration will be used and consequently greenfield run-off calculations would not apply. However, during high ground water events, or when discharging to the Lancing Brooks diches, flow control devices will be used to ensure the maximum run-off rate does not exceed the greenfield rate for a range of return periods. The details for the flow control devices would be developed in accordance with WSCC requirements and the guidance within the CIRA SuDS manual, all of which will be provided during the detailed design stage

The use of permeable paving for roads, parking courts and hard paved areas is considered to provide negligible benefit for critical groundwater and surface water flooding events, given the high groundwater levels associated with much of the development area.

As mentioned above for the majority of the time shallow infiltration should be possible. The sub-base material of the permeable roads will be designed to accommodate a 6 hour high ground water event (to match the tides) whilst accommodating a 1 in 100+CC rainfall event. Shallow swales will be located adjacent to the road should an exceedance event occur, which link into the adjacent ditch drainage system.

5.3 Lancing Brooks Modelling

This paragraph makes reference to Appendix Q, and modelling of both the 1 in 100+40% CC and the 1 in 1000-year return periods. Appendix Q only contains output images; without the methodology, catchment assessed, input values and other key parameters used to generate the model outputs, they are of no value to the assessment.

The new modelling of the Lancing Brooks has taken the raw data used in the original SWMP model (provided by CH2MHill) and JBA (who produced the ADUR SFRA) have undertaken a more detailed analysis in accordance with the requirements set down by the EA.

The results of the JBA analysis is summarised within the FRA. As the data is so large the modelling would need to be reviewed via a data file. We can provide the model to yourselves or if there is any specific output required please specify. This data will, also be reviewed by the EA.

We have attached an Explanation Note prepared by JBA which gives additional details of the modelling and confirms that the catchment is the same as the SWMP study area.

Section 6.0 Flood Management and Mitigation

6.3 Floor Levels

WSCC LLFA is concerned at the allowance of only 0.3m freeboard between ground floor level and the existing ground surface given the high risk of ground water / surface water flooding over parts of the site.

With reference to the Drainage Strategy drawing within the FRA there are significant bioretention areas/detention basins across the site which been located in areas which are designed to flood first.

With the ground floor level set at 300mm above the existing ground level it is not expected that anything other than an exceedance event would flood the ground floor. The ground floor areas will be used for garage/storage/utility areas, which will be constructed using flood resilient material. All habitable rooms will be a first floor level. However, FFL's to each dwelling will be assessed on and individual basis during detailed design with the opportunity for raising further should the localised topography dictate.

Additional modelling has been undertaken by JBA to establish the effects of reduced infiltration across the catchment area. This updated modelling assessment is based on a conservative assumption that only 50% of the catchment infiltrates i.e. 50% of the rainfall event will run off site to the surrounding Lancing Brooks network. Results indicating the depth of flooding (during a 100+CC event) including a tide lock scenario demonstrate that the majority of the site is free of flooding with approximately 30% of the site area showing flooding up to the depth range of 10-200mm. It should be noted that these results are based on the current topography and doesn't allow for the raising localised areas or include any of the proposed mitigating measures such as conveyance swales and bioretention areas/detention basins. Refer to modelling output image "50% Run off with new channel and tide locking, 100yr+40% CC event"

6.5 Surface Water Run-off Rates

Consistent with the recommendations in the Lancing SWMP, WSCC LLFA would advise that infiltration into the subsoils is not an appropriate drainage measure for the site. It is unclear how / where water will be attenuated upstream of flow control devices; clarification should be sought.

Ground water monitoring indicates that shallow infiltration is possible and with over 12 months of monitoring on the eastern part of the site infiltration will work throughout the year. It is recognised that there may be periods of time or locations where infiltration will be less effective, in which case surface water will discharge to the Lancing Brook ditch system via a controlled outfall. The Drainage Strategy drawing within the FRA provides an indicative layout of new swale connections to the existing ditch network. Flow controls and non-return values would be located at this junction, the details of which will be finalised as part of the detailed design.

6.8 Safe Access and Egress

This states*: it may not be possible to provide a dry means of escape from the buildings in the event of a flood. To overcome this the units will have direct access to the first floor, which will be the primary area for refuge in the event of a major costal flood event*.

The above statement is not consistent with our interpretation of NPPF paragraph 103. If this response is to form part of the flood evacuation plan for all properties within New Salts Farm, how are disabled residents being accommodated for?

All units only have accommodation at first floor level and above. Consequently, any wheel chair residents would already have access to the higher levels. This approach has been accepted on a number of other schemes within Adur.

6.10 Foul Sewers

Confirmation needs to be sought by A&WC from Southern Water whether the proposed infrastructure measures outlined in Appendix C will provide sufficient capacity to address the proposed new development, existing sewerage flooding issues and take into consideration the Southern Water measures proposed to accommodate the allocation of 600 homes at New Monks Farm. Specifically, confirmation from Southern Water should be sought that they have strategically reviewed all the proposed development affecting this sewerage catchment and can provide the capacity needed within the timescale that development is being brought forward.

No reference is made to the issue of groundwater inundation of the foul system and we assess this as being a significant risk. Inundation may result from hydrostatic pressure on the existing foul drains as well as excessive surface water draining down through any existing or new foul sewer manholes. This risk needs to be considered and appropriate mitigation put in place.

Southern Water have undertaken a capacity assessment of their sewerage system and it is for them to comment on what they have allowed for in their analysis. However, it is our experience that an allowance for surface/ground water ingress into the sewerage is included within their capacity calculations. Likewise, any new development which has been included within the Local Development Plan would also be included as a matter of course.

The NSF development will provide a new foul drainage system within the site and as a consequence we can control the pipe and jointing specification for the private drainage system. As part of the detailed design stage additional testing can be undertaken to establish If an enhanced jointing specification is required. However it is anticipated that much of the new foul system will be offered for adoption by Southern Water and as a consequence the detailing of the drainage system would need to be agreed with them.

13.0 Conclusion

The conclusion states that: *the main flood risk for the development is associated with coastal flooding.* This statement and the tenor of the FRA, as a whole, in our view underplay the significant flood risk from groundwater and surface water.

The site conditions are summarised in paragraph 3.4.2 of the Lancing SWMP that states:

Under conditions of high winter recharge and elevated groundwater levels in the Chalk and in response to upward groundwater pressure from the underlying Chalk, there may be upward leakage from the Chalk to the upper aquifer and surface water. This occurs through more permeable windows in the Superficial Deposits... Where there is partial connectivity between the two aquifers, the upper alluvial aquifer may become more permanently saturated, leading to areas of marshy ground. These mechanisms are most likely to occur in the southern part of the study area.

It is these conditions of saturated ground water in combination with prolonged rainfall that present a significant issue for surface water drainage for the site and for existing residences in adjacent areas that rely upon the same drainage network. The FRA has not demonstrated that surface water under these conditions can be adequately drained without causing increased flood risk to existing residents in the catchment or risking flooding of the proposed residential properties.

There are two methods of surface water disposal for the site. The SWMP states that the ditches are only partially influenced by high ground water and the Lancing Brooks system has been modelled to replicate 1 in 100 +CC event without flooding the site in a tide lock scenario.

Additional modelling has been undertaken by JBA to establish the effects of reduced infiltration across the catchment area. This updated modelling assessment is based on a conservative assumption that only 50% of the catchment infiltrates i.e. 50% of the rainfall event will run off site to the surrounding Lancing Brooks network. Results indicating the depth of flooding (during a 100+CC event) including a tide lock scenario demonstrate that the majority of the site is free of flooding with approximately 30% of the site area showing flooding up to the depth range of 10-200mm.

Significant areas on the site are to be proved to accommodate surface flooding and exceedance flood events have been considered within the design of the unit types. Any surface flooding on the site would be held within it and the additional ditch network proposed combined with enhanced levels of maintenance provided to the ditches should reduce the flood risk for the adjacent residential areas.

It is recognised that the FRA does not have the same level of detail that would normally be required for a planning application. However, it should be noted that the purpose of this FRA is to advise on the suitability of the New Salts Farm to be allocated within the Local Development Plan. Within any future planning application, a detailed FRA prepared in accordance with the requirements of NPPF. However, we believe there is sufficient detail (with the provision of the additional Lancing Brooks modelling discussed above) to demonstrate that a development on the New Slats Farm could be designed such that it would not pose an unacceptable level of flood risk both within and beyond the site.

Yours sincerely

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28 February 2017

References:

A. WSCC LLFA letter to Ben Daines, Adur-Worthing Council dated 5 Jan2017.
B. Email: Andrew Picton (Tully De'Ath) Ray Drabble (WSCC LLFA) dated 8 Feb 17.
C. Email Ray Drabble (WSCC LLFA) Andrew Picton (Tully De'Ath) dated 9 Feb 17.
D. Tully De'Ath (Feb 2017) High Groundwater Scenario (No filtration).
E. JBA Consulting (Feb 2017) New Salts Farm Road, Shoreham Modelling Report and appended Flood Estimation Report

Dear Andrew,

Pre-Application Query: Tully De'Ath Flood Risk Assessment for The Hyde Group; Further Evidence Review

West Sussex County Council (WSCC) in its statutory capacity as Lead Local Flood Authority (LLFA) has been approached by Tully De'Ath (Reference B) for comments on further evidence (References D and E) provided in support of Hyde Homes' proposal to develop New Salts Farm.

WSCC LLFA has now reviewed the additional evidence and this letter summarises our comments / questions.

1. High Groundwater Scenario (No filtration)

Storage estimate

1.1 The scenario estimates the storage for the above to be 11,950m^3. Following further discussion with Tully De'Ath, the LLFA has estimated the storage required using a CIRIA method based upon calculation of storage on a 1m^2 basis that produced a figure of 6,086m^3. It is noted, however, that the half empty time is 32 hours. In order to satisfy the half empty rule, a minimum capacity of 6,853m^3 would be required, particularly recognising that for a worst case scenario, discharge to the Lancing Brooks may not be possible (see assumptions in modelling in section 2, below and EA Surface water flood map).

Ground raising / flood risk to adjacent properties

1.2 At a meeting attended by Tully De'Ath, Bowyer Planning and WSCC LLFA, the documentation supporting the above scenario was provided in hard copy and discussed. It was explained that some ground-raising would be necessary towards the south east of the site by 0.4-0.5m to achieve the levels necessary to drain the site by gravity. How would such ground raising be achieved without increasing flood risk to the adjacent existing properties and what construction / profile is envisaged for the boundary?

Detention basin in relation to groundwater levels

1.3 The scenario shows the base level of the detention basin to be 0.9mAOD. Analyses of a selection of the borehole data commissioned by Tully De'Ath has shown the ground water level to be frequently higher than 0.9m AOD (see Table 1 below).

Date	Time	Level 107AOD	Level 108AOD	Level109AOD	Level5AOD
13/01/2017	02:00:00	1.364	no data	1.374	1.316
20/11/2016	06:00:00	1.31	no data	1.262	1.267
14/01/2017	03:00:00	1.296	no data	1.219	1.335
12/01/2017	14:00:00	1.271	no data	1.219	1.236
17/11/2016	04:00:00	1.201	no data	1.072	0.94
20/11/2016	07:00:00	1.199	no data	1.258	0.906
16/10/2016	14:00:00	1.185	no data	0.979	1.167
19/10/2016	16:00:00	1.184	no data	0.9	1.271
12/01/2017	13:00:00	1.183	no data	1.001	1.552
16/11/2016	15:00:00	1.179	no data	1.052	1.096
13/01/2017	03:00:00	1.179	no data	1.34	0.91
17/11/2016	16:00:00	1.177	no data	1.072	0.996
16/11/2016	03:00:00	1.17	no data	1.077	0.962
17/11/2016	03:00:00	1.167	no data	0.794	1.345
14/01/2017	04:00:00	1.16	no data	1.249	0.891
17/10/2016	15:00:00	1.143	no data	0.989	1.015
17/10/2016	02:00:00	1.14	no data	0.83	1.285

Table 1; Selected analyses of borehole data

1.4 The highest 17 ground water level readings for WLS107 (selected arbitrarily) are shown against data for WLS 109, and WLS5; there was no data available for WLS 108. The following can be noted;

- During October and November 2016 and January 2017, groundwater levels exceeded the proposed base level of the detention basin of 0.9 AOD;
- There is reasonable correspondence between the groundwater levels across Area 1; indeed, it is suggested that differences probably relate to a time lag in the tidal influence than to any other factor; (e.g. the readings for 13/01/2017 for WLS 5 for midnight, 0100 and 0200 are 1.633, 1.541 and 1.316, respectively).
- The tidal influence extends across the whole of Area 1.

- If infiltration drainage were to be deemed feasible, then best practice suggests that the base of any ground infiltration should be at a minimum elevation of 2.3m AOD[1].
- For the purposes of storage design, the base of the detention basin cannot be lower than 1.3mAOD.

1.5 The data in Table 1 also needs to be taken in context, having been recorded when ground water levels reached their lowest levels for many years. Winter conditions more typically are illustrated by plates1 and 2 below that show saturated groundwater conditions.

Plate 1 taken near WLS111 winter 2015.

Plate 2 taken west of WLS109 winter 2015.

[1] CIRIA Report 156 paragraph 4.4 (g).

1.7 Based upon the above, for design purposes, Tully De'Ath would need to establish the seasonal high groundwater levels based upon best available knowledge over recent years.

Other Design Considerations

1.8 Tully De'Ath is also requested to demonstrate how its design for highways and parking areas will satisfy recommended best practice for water quality treatment in chapter 26 of the SuDS Manual.

1.9 The scheme shows swales designed with a slope of 1/500; this is outside the design recommendations of the SuDS manual that states longitudinal slopes should be constrained to 0.5%-6%. How does Tully De'Ath justify this diversion from best practice guidance?

2. New Salts Farm Road, Shoreham Modelling Report

2.1 Section 2 of the above report states:

> *An existing 1D ISIS model of the area had been developed to inform the 2XXX Surface Water Management Plan. This model was based on cross section topographical survey and run for steady state flow estimates that did not appear to be based on any recognised method of flow estimation. The model was not suitable for the purposes of supporting a site-specific Flood Risk Assessment. Therefore, a new model and new hydrology was required.*

2.2 Notwithstanding whether or not the original model was accurate, the LLFA disputes the limited catchment used by the current model as being representative of the inputs to the Lancing Brooks.

2.3 Section 2.1 of the above report refers to the Catchment boundary as follows:

> *The catchment boundary was based on two neighbouring FEH catchments which are located within in the proposed development site. These are identified as the south-western and south-eastern FEH catchments in Figure 2-1 below. The entire Lancing Brooks catchment was not used as it was considered too large an area and may not represent the sites characteristics as accurately.*

2.4 The LLFA challenges this assumption and considers that the limited catchment used by the model does not accurately reflect inputs to the Lancing Brooks drainage network.

2.5 Section 2.1 further states:

> *The Lancing SWMP indicates that surface water runoff from the A27 drains via a series of outfalls into the Lancing Brooks. However, this will flow south and eastwards from the A27 into the Northern catchment area shown in Figure 2-1. As the outflow for this FEH catchment is*

approximately 1km to the north-east of the site, the outflow from the A27 was not considered in the model.

2.6 The LLFA is not prepared to accept the above assumption without robust evidence to underpin this challenge to previous studies.

3. Notes on Flood Estimation Report (Appendix A to Modelling Report)

Observations are as follows:

Re: 1.7 Other data available; this is very sparse, given the range of previous studies for the site;

Re: 1.8 Hydrological Understanding of Catchment; provides insufficient detail to explain model inputs and outputs.

Re: 2.1-2.2 What does Catch 1 relate to in the figure shown at the top of Section 2? No area is given. Also it refers in 2.3 to the area as being of 'low relief' but the Lancing Brooks catchment includes Lancing Hill at 81m.

Re: 5.2 Assumptions, limitations and uncertainty

…it is assumed that the 2D direct rainfall boundary accounts for the area that contributes the runoff to the relevant section of the Lancing Brook system that presents flood risk to the site.

The LLFA considers this to be an invalid assumption. Our evidence indicates that the catchment draining to the southern outfall includes the whole catchment shown on the figure incorporated in section 2 of the document although it is acknowledged that a proportion of the flow from Honeyman's Hole flows east and out through the Northern outfall.

Other watercourse – it is assumed that the remainder of the Lancing Brook with a channel gradient draining away from the site do not contribute to flood risk to the site. This includes much of the drainage to the west and north of the airport and all areas north of the A27.

As above, the LLFA does not consider this to be a valid assumption.

The LLFA is also interested to know how the A27 runoff has been taken into consideration as this represents a separate sub-catchment and to which the principal run-off coefficient would not apply.

4. Additional Comments

4.1 The LLFA comments need to be considered alongside comments from the District Engineer and the Environment Agency both of which have a statutory regulatory role with regard to proposed development in relation to flood risk and drainage. Details of this proposal have been circulated to these bodies.

4.2 In view of the above, any further iterations of the proposed drainage strategy should be forwarded to the Local Planning Authority which is the body responsible for undertaking detailed technical review of applications.

Yours sincerely

Ray Drabble
Flood Risk Engineer (Sustainable Drainage)
West Sussex Lead Local Flood Authority

Copies to: Ken Argent, Adur-Worthing Council
Ben Daines, Adur-Worthing Council
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Internal: Caroline West, West Sussex County Council

This document outlines the response to the 'Pre-Application Query: Tully De'Ath Flood Risk Assessment for The Hyde Group; Further Evidence Review' written by Ray Drabble of West Sussex County Council (dated 28th February 2017)

1.1

Tully De'Ath have taken the most onerous set of conditions; assuming no infiltration and taking the worst case scenario in terms of storm events; climate change and tide lock and have demonstrated that the development can be delivered.

For this scenario we have undertaken further detailed analysis and the results show a reduction from the original storage requirement of 11,950m³ to a revised figure of 8,990m³ using the Micro Drainage design suite allowing detail design of a fully integrated storm water drainage system.

In practice infiltration is likely to be available which will contribute to the reduction in the volume of storage required.

Results of insitu infiltration testing undertaken on 7th and 8th of March 2017 have shown that infiltration rates are variable across the site with values ranging between $3.18x10^{-5}$ to $2.6x10^{-6}$m/s. These infiltration rates are suitable for pervious pavement design.

Additional testing will be undertaken as further areas of the site are deigned to produce 'area specific' solutions.

In response to concerns stated later in this Pre-Application Query response (Section 1.4) revised detention basin base levels have been derived using both Tully De'Ath and LLFA storage volume figures

1.2

The raising of levels in the south west corner of the site relates to finished floor levels to 3 or possibly 4 houses (and associated garages) and not general ground levels in that area.

The general ground levels around the buildings will remain as existing and it should be noted that in accordance with the Flood Risk Assessment there will also be opportunity for any 'exceedance' surface water to move freely within the sub-floor void therefore not increasing the risk of flooding to adjacent existing properties. Under these circumstances it is considered that the boundary condition remains as existing.

However, if during the detailed design process, any issues arise at specific locations these can be overcome by the introduction of localised bunds, probably no more than 200-300mm high to protect adjacent dwellings.

1.3

The ground water level data for WLS108 is included within the Excel file previously issued to WSCC. For the ease of reference, we have added the data to Table 1.

Date	Time	Level 107AOD	Level 108AOD	Level109AOD	Level5AOD
13/01/2017	02:00:00	1.364	0.929 - #	1.374	1.316
20/11/2016	06:00:00	1.31	0.751	1.262	1.267
14/01/2017	03:00:00	1.296	0.681	1.219	1.335
12/01/2017	14:00:00	1.271	0.663	1.219	1.236
17/11/2016	04:00:00	1.201	0.571	1.072	0.94
20/11/2016	07:00:00	1.199	0.840	1.258	0.906
16/10/2016	14:00:00	1.185	0.429	0.979	1.167
19/10/2016	16:00:00	1.184	0.380	0.9	1.271
12/01/2017	13:00:00	1.183	0.451	1.001	1.552
16/11/2016	15:00:00	1.179	0.463	1.052	1.096
13/01/2017	03:00:00	1.179	0.997 - #	1.34	0.91
17/11/2016	16:00:00	1.177	0.513	1.072	0.996
16/11/2016	03:00:00	1.17	0.479	1.077	0.962
17/11/2016	03:00:00	1.167	0.367	0.794	1.345
14/01/2017	04:00:00	1.16	0.885	1.249	0.891
17/10/2016	15:00:00	1.143	0.466	0.989	1.015
17/10/2016	02:00:00	1.14	0.358	0.83	1.285

Table 1; Selected analyses of borehole data

= same tide event

1.4.1

The detention basin sits directly over WLS108 where high ground water levels have consistently been below the 0.9m A.O.D. Over the monitoring period there was only one 3-hour occasion where the ground water levels were recorded above 0.9m A.O.D., which reached a level of 0.997m A.O.D.

However, with reference to point 1.1 it is proposed to raise the base level of the detention basin to 1.20m A.O.D. (based on the Tully De'Ath storage calculation of 8,990m³) or 1.35m A.O.D. (based on the WSCC storage calculation of 6,853m³). In addition, the detention basin could be lined which will add further protection to infiltration from very high ground water events.

1.4.2

It is recognised that there a time lag between high tide level and high ground water level; however, all the ground water levels are recorded on an hourly basis across the site. The data shows that high water levels recorded in WLS108 are typically 1 hour later than WSL107 & WLS109

1.4.3

It is agreed there is a tidal influence across the site, although it does vary across the development area, however it does not affect the proposed drainage strategy for the site.

1.4.4

The drainage strategy has been developed with the main aim of providing pervious pavements so that all external hardstanding areas such as access roads and car parking would infiltrate into the natural soils with the aim of replicating existing conditions and achieving the key message contained in the CIRIA SuDS Manual 2015 *"SuDS should be designed to maximise the opportunities and benefits that can be secured from surface water management".* In the occurrence of exceedance events swales are provided adjacent to these areas to perform a dual function of infiltration and also conveyance to the detention basin(s).

It was also considered that where localised areas would possibly not allow for infiltration the pervious pavement could be lined with an impermeable membrane and used as a conveyance system discharging to the detention basin(s).

It is important to reiterate that the current drainage strategy allows for both options in line with SuDS Manual which offers guidance on the selection of pervious pavements (ref Chapter 20 and Table 20.1) i.e. a Type A 'total infiltration' system where ground conditions allow or a Type C 'no infiltration' system where infiltration is not suitable

The High Groundwater Scenario (No Infiltration) assessment assumes a Type C pervious pavement.

1.4.5

This statement does not accurately reflect the site conditions/constraints. It is possible to set the detention basin base level lower than 1.3m A.O.D. However, based on WSCC storage calculations of 6,853m³ the detention basin could be set at 1.35m A.O.D. which meets their requirements.

1.5

The areas of surface ponding are known to us and future layouts will take this into account. These areas will be set aside for open space.

1.6

This point is not referenced in the letter.

1.7

Ground water monitoring has been ongoing for over 12 months on the eastern side of the site, which started in December 2015. Based upon the Met Office data for the south coast area the annual rainfall for 2012 & 2014 was recorded as above average, and average for 2013 & 2015. In addition, the rainfall in January of 2016 was also above the normal levels. As a consequence, it would be reasonable to assume that the water levels recorded at the start of the monitoring period would be at or above the seasonal average. In reviewing the ground water levels over the 12-month period levels do not appear to have dropped.

The monitoring across the site will continue and will be used to inform the design moving forward as LLFA have established.

1.8

With reference to Chapter 8 within our Flood Risk Assessment, permeable paving does provide sufficient surface water treatment to comply with the guidance within Chapter 26 of the SuDS Manual

1.9

The swales have been incorporated in the drainage strategy as an additional benefit to the principal of either 'total infiltration' system or a 'no infiltration' system for the external hardstanding areas. If the 'total infiltration' system is utilised the pervious pavement is designed to hold the required storm event within the sub-base of the road/parking whilst infiltration occurs. The 'no infiltration' system is designed to convey the surface water to the detention basin(s) within the sub-base.

Both systems work independently without the need for swales, the reason for their inclusion is to provide an additional element of protection for either attenuation or conveyance for some exceedance events.

It is therefore considered that as the swales do not form a fundamental part of the drainage network a departure from the guidance relating to the longitudinal gradient is appropriate.

2.1 – 2.6

Information updated in Appendix A – Flood Estimation Report.

In their letter dated 28/02/17 the LLFA dispute the catchments used in the modelling as being representative of the inputs to the Lancing Brooks (2.2), and requests robust evidence be prepared to support the extent of the model domain used in this study (2.5).

To this end additional modelling has been undertaken to sensitivity test the assumption on catchment and drainage paths as discussed in section 2.1 to 2.6 of the letter. This is outlined in the updated Flood Estimation Report. Notably Sections 1, 2 and Appendix B have been updated to provide more evidence for the approach taken.

To test the assumption of contributing area for the direct rainfall approach a larger extent model was developed. The Design domain was increased in area from 6.7km^2 to approximately 11.6km^2. Both the design model domain and the sensitivity model domain were run 2D only (without any embedded 1D channels or culverts) for the 1:100 AEP event with runoff based on SPRHOST throughout the active domain.

This sensitivity test was undertaken to identify flow paths into the contributing area that may not have been accounted for within the original design domain.

Two of these flow paths have been identified to drain outside of the previous domain.

Two other flow paths have been identified to drain to inside the previous domain. Of these, the southern flow path flowing Monks Avenue, has potential to contribute a small volume of additional flow to the top of Brook in this location. This brook drains to the south and then passes east across the study site. Given the small area concerned and the noted presence of urban drainage infrastructure in this area connected to soakaways (Chapter 5 of Surface

Water Management Plan) it is considered that the additional volume of flow this are may contribute to flooding at the site to be negligible.

The second red flow path runs parallel to the south of the A27 carriageway. This flow path runs east and crosses Marsh Barn Lane where it would be picked up by the network of brooks to the west of Shoreham Airport. These brooks continue to drain east and outfall to the estuary. They do not flow though study site, therefore their omission from the model domain is considered immaterial.

In conclusion, it is considered that the design model domain is appropriate of the study site.

3.0

Re: 1.7 - observation answered in section 1.4 of revised report.

Re: 1.8 - observation answered in section 1.5 of revised report.

Re: 2.1-2.2 – note added in section 2.1 of revised report.

Re: 5.2 – information updated in section 5.2 of revised report.

4.1

Noted.

4.2

Noted.

In summary, having reviewed the points raised in the letter, additional analysis and modelling has been undertaken.

The results demonstrate that there is a viable technical solution to develop the site for residential housing without increasing the risk of flooding within and beyond the site.

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21 March 2017

Dear Julian,

Pre-Application Query: Tully De'Ath Flood Risk Assessment for The Hyde Group; Further Evidence Review (March 2017).

WSCC LLFA has now reviewed Tully De'Ath's response (Reference G) to our earlier letter (Reference F) together with Reference H.

This letter summarises the LLFA's current position with respect to the above.

Notwithstanding the sensitivity testing undertaken on the model, the interpretation provided by Reference H of drainage flows across the site is not consistent with the LLFA's understanding of the Lancing Brooks network (the latter updated by recent survey work of drainage across the New Monks Farm site). The modelling report outputs included in Reference I show extensive surface water flooding issues for 1%AEP +CC events. It is considered that these plots do not accurately represent the potential flood risk from surface water under a worst case scenario but they do provide an indication of the surface water management issues for the site.

Monitoring of groundwater levels during 2016 has shown these to be repeatedly higher than 0.8m AOD across the site over winter months. The LLFA does not hold data for New Salts Farm for 2012 / 2013; however, there is evidence to suggest that groundwater levels (GWLs) at New Salts Farm would have been significantly higher than has been recorded for the site during 2015-16. In view of the inherent difficulties with draining an essentially flat area of coastal plain, it is considered that the assessment of flood risk needs to be based upon a worst case scenario. Furthermore, the tidal influence upon GWLs across the site is evident from the monitoring data; it is reasonable, therefore, to expect that the effects of sea-level rise (SLR) will result in increasing GWLs over time because of the tidal influence – particularly over the lifetime of the development. Contrary to Tully De'Aths assertion that tidal influence does not affect the proposed drainage strategy for the site, it is the LLFA view that tidal influence and the related consideration of SLR effects upon GWLs **should** influence the proposed drainage strategy for the site. It is, however, acknowledged that further research into this issue is needed. This issue is being given some consideration in a forthcoming GWL study commissioned by the LLFA to inform a better understanding.

While Tully De'Ath has provided a theoretical drainage solution, it is based upon a design that has inherent problems that are likely to inhibit its effective implementation in practice; examples include the divergence from the SuDS manual recommendation that longitudinal slopes should be constrained to 0.5%-6%; gradients for the swales are shown on the Drainage Strategy at 0.2% but given the very flat nature of the site, it is possible that gradients would be even lower than 0.2%. A further issue is the suggestion in Reference G (section 1.4.1) of lining the detention basin to add further protection to infiltration from very high ground water events. This would defeat the scope for infiltration potentially worsening drainage for this part of the site as well as introducing additional concerns regarding flotation of the basin.

Conclusion

Concerns regarding the effective implementation of the proposed drainage strategy, at the margins of design tolerance, need to be set in a wider context of uncertainty regarding the baseline conditions that have informed the design; specifically the hydraulic modelling undertaken where assumptions made do not reflect documented data for Lancing Brooks; and the GWL monitoring data for the site that has been unseasonably low. On this basis LLFA reservations regarding development of New Salts Farm remain; namely that there is a high risk of surface water flooding during a 1% AEP + CC event.

Yours sincerely

Ray Drabble
Flood Risk Engineer (Sustainable Drainage)
West Sussex Lead Local Flood Authority

Copies to: Ken Argent, Adur-Worthing Council
Ben Daines, Adur-Worthing Council
Adrian Jackson, Environment Agency
Dinny Shaw, Bowyer Planning

Internal: Caroline West, West Sussex County Council

References:

A. WSCC LLFA letter to Ben Daines, Adur-Worthing Council dated 5 Jan2017.
B. Email: Andrew Picton (Tully De'Ath) Ray Drabble (WSCC LLFA) dated 8 Feb 17.
C. Email Ray Drabble (WSCC LLFA) Andrew Picton (Tully De'Ath) dated 9 Feb 17.
D. Tully De'Ath (Feb 2017) High Groundwater Scenario (No filtration).
E. JBA Consulting (Feb 2017) New Salts Farm Road, Shoreham Modelling Report and appended Flood Estimation Report
F. WSCC LLFA letter to Andrew Picton, Tully De'Ath dated 26 Feb 17.
G. Tully De'Ath response to WSCC LLFA letter dated 26 Feb 17.
H. FEH Calculation Record (v3) dated 8/03/17.

I. New Salts Farm Flood Risk Assessment Issue 4.

Andrew Picton

To: Andrew (ajp@tullydeath.com)
Subject: FW: New Salts Farm - WLS3
Attachments: GroundwaterExplanationMay17.docx

From: Ray Drabble [mailto:Ray.Drabble@westsussex.gov.uk]
Sent: 26 May 2017 16:17
To: Andrew Picton <ajp@tullydeath.com>
Cc: Dinny Shaw <DinnyShaw@boyerplanning.co.uk>; Ben Daines <ben.daines@adur-worthing.gov.uk>; Ken Argent <ken.argent@adur-worthing.gov.uk>; Julian Turner <jct@tullydeath.com>; Kevin Macknay <kevin.macknay@westsussex.gov.uk>
Subject: RE: New Salts Farm - WLS3

Andrew,

Further to your telephone call earlier this week, I have now completed our additional note on groundwater concerns, stressing the need to ensure that appropriate seasonal maxima are used for hydraulic / storage calculations: this is attached. I believe that this addresses the observation made in paragraph 1.12 of the Response to ALP025D and Update on flood mitigation and drainage discussions for New Salts Farm (19th April 2017).

In view of the published preliminary findings of the Planning Inspector to the Adur LP, I do not intend at this point to address further matters raised in the most recent correspondence / Peer review. No doubt this will be re-visited in the future when the LP is reviewed.

Kind regards

Ray Drabble
Flood Risk Engineer (Sustainable Drainage)
Economy, Infrastructure and Environment
Highways and Transport
West Sussex County Council

CALL IM EMAIL

Location: Western Area Office, Drayton Lane, Nr. Chichester, West Sussex. PO20 2AJ.
Contact: Internal: 24077 | **External:** +44 (0)330 2224077 | **Mobile:** +44 (0)7590183138 | **E-mail:** Ray.Drabble@westsussex.gov.uk

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Explanation of evidence to suggest the need to review the New Salts Farm Groundwater Monitoring Data in the context of wider Time-Series Data

1. In its letter entitled: Pre-Application Query: Tully De'Ath Flood Risk Assessment for The Hyde Group; Further Evidence Review (March 2017) and dated 26 March 2017, the LLFA included the following statement:

   The LLFA does not hold data for New Salts Farm for 2012 / 2013; however, there is evidence to suggest that groundwater levels (GWLs) at New Salts Farm would have been significantly higher than has been recorded for the site during 2015-16.

2. Subsequently, following a request from Tully De'ath, the LLFA agreed to provide an explanation of the evidence referred to in this statement;

3. Time series data for groundwater levels has been plotted for two EA boreholes in close proximity to New Salts farm, namely Sussex Pad, 1.6km away and Crosshaw Recreation Ground in South Lancing, 2.8km distant (see Figure 1).

   Figure 1 Locations of EA Boreholes at Crosshaw Recreation Ground and Sussex Pad.

4. It is stressed that these data are not intended to be proxy data for New Salts Farm; rather they show a relationship between groundwater levels and rainfall that, it is suggested, allow some basic conclusions to be made with regard to groundwater levels (in the context of previously provided photographic evidence) at New Salts Farm.

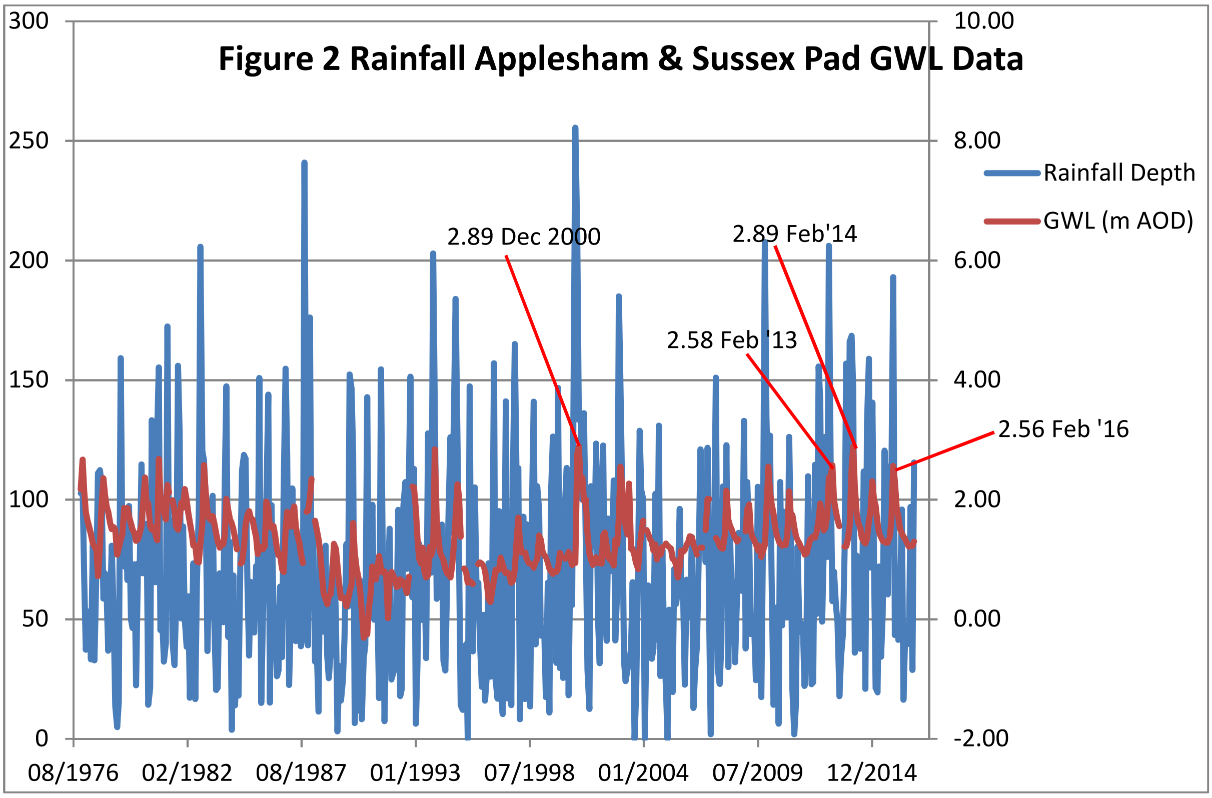
5. Figure 2 shows monthly rainfall data for Applesham Farm overlaid with EA averaged groundwater monitoring data for Sussex Pad; Figure 3 shows averaged monthly groundwater level data for Crosshaw.

6. Three observations are made in relation to these data:

   - Peaks in groundwater levels broadly coincide with or follow peaks in monthly rainfall or sustained high rainfall over a period of weeks; Thus record high monthly rainfall in October 2000 of 266mm (sandwiched between high rainfall during September and November) resulted shortly after in a peak in groundwater level of 2.89m AOD at Sussex Pad in Dec 2000. The same groundwater level was recorded in Feb 2014 following heavy rainfall during October (157mm) December (166mm) January (163mm) and February 146mm). Figure 2 also shows that low groundwater levels correspond to reduced monthly rainfall. Similarly the peak groundwater level of 5.47m AOD was recorded at Crosshaw in February 2000 following the very wet autumn. More recently, the wet autumn / winter of 2013/14 resulted in a peak in average groundwater levels in February 2014 (Figure 3).
   - From their relative high level in January 2016, groundwater levels at both sites were in decline during the remainder of the year with the exception of June at Crosshaw which saw a slight increase in levels (in response to higher than average rainfall for the month).
   - The correspondence between the peaks in groundwater levels at Crosshaw and Sussex Pad suggest that groundwater levels over the wider area are likely to show similar relative peaks, troughs, rises and falls for the same time period, albeit the absolute groundwater levels may be very different and local idiosyncrasies will apply

7. From the above observations, it is reasonable to suppose that groundwater levels recorded as part of the groundwater monitoring study in support of the New Salts Farm development that were taken between November 2015 and February 2016 were not at their seasonal maximum. It is suggested that levels would have been significantly higher during the wet autumn / winter of 2013/14. This is further corroborated by photographic evidence (plate 1) that shows extensive surface water flooding in March 2014. This notwithstanding, the peak groundwater level for WLS5 was recorded on 8 Feb '16 at 0.352m above ground level (1.62m AOD). The monitoring report also states that the groundwater level at WLS 5 consistently rose above ground level at high tide indicating the groundwater level is periodically artesian. Given that WLS5 is located 1.27m AOD, this is indicative of the drainage challenges for the eastern portion of the site. On 10 Feb '16, records from WLS 2 show groundwater level being recorded at 0.15m below ground level (estimated to be 1.6m AOD from LiDAR also suggest seasonal groundwater levels can exceed 1.6m).

8. The current drainage proposals for New Salts Farm are based upon monitored ground water levels during 2015/16 that the above evidence suggests are below their recent seasonal maxima. The recorded groundwater level of 1.62m AOD at WLS5, approximately 250m from the centre of the proposed detention basin, means that there is ~0.27m depth loss in the storage for the basin (based upon a basin level of 1.35m AOD) and the hydraulic calculations for the extreme storm event would need to take this into consideration. It is suggested that further allowance needs to be made for yet higher seasonal groundwater levels to avoid risk of design-failure.



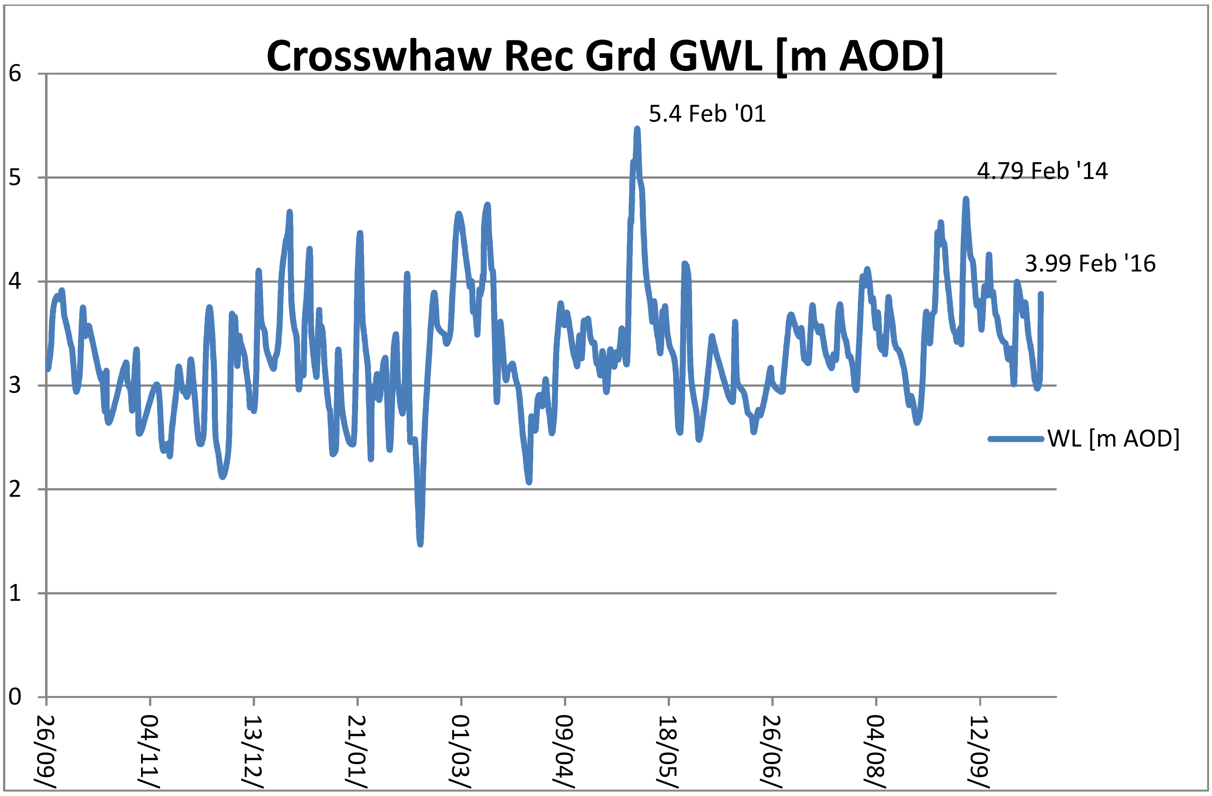


Plate 1 Aerial photo taken Mar 2015 showing western portion of New Salts Farm Site.

9. The LLFA is grappling with understanding better the relationship between groundwater levels and surface water flooding, particularly in the coastal plain. To this end, a groundwater study has been commissioned study to address this knowledge gap. The long term aim is to, where necessary, make applications to the RFCC for Local Levy funding for capital works to characterise and where appropriate, alleviate groundwater flooding issues.

10. The study will focus on selected groundwater flood risk monitoring pilot test locations characterisation of the local geological and hydrogeological setting and assessment of the availability, suitability and integrity of existing hydrometric data to characterise the hydrogeological regime(s). At least one of these locations will be on the coastal flood plain and the relationship between tidal influence and groundwater levels will be further investigated and consideration given to the long-term implications of sea-level rise.

Ray Drabble
Drainage Engineer
West Sussex County Council Lead Local Flood Authority

Ray Drabble
West Sussex County Council
Western Area Office
Drayton Lane
Chichester
West Sussex
PO20 2AJ

AJP/rw/11649
25th July 2017

Dear Ray

Re: New Salts Farm Road Land – Shoreham
In response to Ray Drabble letter issued on 26th May 2017

The WSCC letter provides ground water monitoring results for two boreholes located approximately 2.8 km and 1.6 km away from the development site. WSCC make the point that this data is not intended to be proxy data for New Salts Farm rather to show the relationship of ground water levels and rainfall events.

Our response to the items raised within the WSCC letter are based upon onsite borehole data which commenced in November 2015.

It is acknowledged that the two borehole locations provided indicate that there is a direct relationship between ground water levels and rainfall events, which is not uncommon. The ground water levels for the two locations provided show the long-term variations spanning across 40 years, which typically fluctuate by 1m- 2m over a year. What is not clear is whether these boreholes are influenced by the tides in a similar fashion to the NSF site, where water levels vary between 1-2m with the tides.

WSCC make specific observations within the EA boreholes regarding the ground water levels during 2016, which were at relatively high level in January and then continued to fall throughout the year. However, based upon the data set we have obtained on site the high tide ground water levels have remained relatively constant throughout 2016.

WSCC suggest that the ground water levels at the start of the New Salts Farm monitoring period were not at the seasonal maximum. The ground water monitoring was started in November 2015 where the rainfall in the Sussex area in 2015 was recorded as being above average. In addition, the rainfall in January 2016 was also above normal levels. Consequently, we would expect the ground water levels to be relatively high at the start of the monitoring period. In reviewing the data for the boreholes which have been installed for 19 months, high tide water levels have remained relatively constant, with only short term fluctuations, usually associated with spring tides.

Within the Lancing SWMP (the chapter that discusses flooding on the West Beach Estate), it is noted that due to the close proximity of the sea, variations within the ground water levels will reduce and there will be a lesser variation in the maximum and minimum levels. Within the SWMP it also states that '*ground water levels within the superficial despots are likely to remain high for most of the year, although there will be some recession through the autumn months'.* This statement backs up the findings of our own monitoring which indicates that the high and low ground water levels do remain relatively constant.

Continued 1/…

Ray Drabble
West Sussex County Council

AJP/rw/11649
25th July 2017

Continued …/2

As we have previously advised WSCC, and as accepted by Adur in their document ALP029 'ADC Comments on Homework Responses 28th April 2017,' the photographic evidence of surface water flooding is misleading as it does not cover the New Salts Farm development site. The photo appears to cover the Old Salts Farm, which is specifically noted within the SWMP as an area which is '*associated with widespread waterlogged ground*'. New Salts Farm is not mentioned as suffering from this form of flooding, which has been confirmed by the farmer who has been managing the farm since 2000.

The data collected from the monitoring wells demonstrates that as you move northwards across the site the tidal influence reduces. Adjacent to the southern boundary (WLS6) the change in level fluctuation is in the order of 2.5m whereas the northern sections (WLS108 & WLS109) of the site the level fluctuate between 1.0m and 1.3m. As well as the distance from the coast, other factors such as lower permeable geology will also affect the tidal influence, as can be seen in WLS108 (location of bioretention basin) where ground water levels have not risen above 1.0m AOD.

The ground water monitoring results clearly demonstrates that the depth of ground water levels do vary across the site. The location of the bioretention basin will be located where the high ground water levels are typically at 0.7m AOD, and have not risen above 1.0m AOD. The proposed base level of the bioretention basin is 1.1m AOD and (following confirmation that soakage is possible) will not be lined to allow for infiltration.

In the event that ground water levels rise above the base level of the detention basin an overflow (set at a level of 1.2m) will direct ground water flows into the adjacent watercourse discharging at greenfield run-off rates. Refer top drawing 11649-CIV-120B.

Surface water calculations have demonstrated the allowing for full infiltration (normal condition) water levels within the basin would reach a level of 1.204m AOD during a 1 in 100+CC event. Assuming no infiltration (worst case) the method of surface water disposal would be via the overflow, which would restrict the discharge to greenfield run-off rates. In this scenario, the water level in the basin would rise to 1.359m AOD. The top of the bank of the basin is proposed at 1.6m AOD with the adjacent roads set at a level of 1.8m AOD, consequently there is additional freeboard of storage built into design of the basin to take account of exceedance events, unusually high ground water levels and the effects of sea level rise associated with climate change for the life time of the buildings.

To provide an independent check on our flood and drainage strategy the BRE were consulted to review our proposals. It was the BRE's opinion that the flood risks on the site can be managed using the methods outline in our proposals.

In conclusion, the local ground conditions on New Salts farm appear to influence the ground water levels by smoothing out the fluctuations caused by rainfall events. Consequently, the results of the two EA boreholes do not represent the conditions found on the proposed development site.

To assist in collating all the information which has been developed over the last six months, our Flood Risk Assessment has been updated (Issue 5) to include the latest ground water monitoring results, updated drainage proposals and correspondence between all the consultees.

Yours Sincerely
for Tully De'Ath

Andrew Picton

Re: New Salts Farm - Road Construction & Levels

1 message

Ken Argent <ken.argent@adur-worthing.gov.uk> 29 June 2016 at 08:54
To: Andrew Picton <ajp@tullydeath.com>
Cc: Kevin Macknay <kevin.macknay@westsussex.gov.uk>, Julian Turner <jct@tullydeath.com>, Sarah Poulter <sarah.poulter@hyde-housing.co.uk>, "Jackson, Adrian" <adrian.jackson@environment-agency.gov.uk>

Andrew.

i promised to give this product a bit more though after our last meeting, i have copied the EA into my comments which follow:

The system clearly is suitable for weak ground, it also clearly acts as a permeable layer, but the literature does not state that the subgrade can be permanently saturated, as a result high ground water periods during the wet months, unless you utilise the ATRA anchor and tendon.(page 5)

But

Whilst you will strive to install all the necessary services at the time of construction, we all know what utility providers are like, they seem to be attracted to new pristine surfaces, - what effect will trench or localized excavations have on the geogrid integrity, once the tendons and anchors are damaged or the geogrid is cut out?.

Ken Argent | Engineer | Engineering Team | Adur & Worthing Councils
01903221374

On 25 April 2016 at 15:34, Andrew Picton <ajp@tullydeath.com> wrote:

> Kevin, Ken
>
> Further to our meeting last week I have attached some product literature for the geogrid system we are looking to use on this site. There are a number of suppliers of the Geoweb system but they are all fundamentally the same. We have successfully used this system on a number of private estate roads where we were required to provide a no-dig road, built directly onto the top soil, or where the ground conditions were very poor (CBR values were non-existent). Generally we have used this system on a permeable road, with base infiltration and attenuation provided within the structure of the road, which is similar to what is proposed for this development.
>
> As typical with estate road design, the construction phase is the more onerous loading situation. Usually a sacrificial impermeable DBM layer sits on top of the Geogrid system during the construction phase which is either left in place with holes punctured into it to allow percolation, or is removed when the permeable surfacing is installed.

We are working up some site specific details and will send them across to you shortly.

I have also attached a copy of the New Salts Farm topographical survey for you information. It was mentioned at our meeting that Phase 1 was located in a low part of the site. Upon closer inspection of the levels, the majority of Phase 1 is relatively high (typically 1.8m AOD) with levels falling to the north. The ditch adjacent to the northern boundary of Phase 1 is the local low point where levels beyond the ditch rise (to the north) up to 1.6.-1.8m AOD before falling again towards the northern boundary.

Kind regards

Andrew Picton | Associate Director

Tully De'Ath consultants

Phone 01342 828000 | Mobile 07739 265802 | tullydeath.com

Tully De'Ath Consultants, Sheridan House, Hartfield Road, Forest Row, East Sussex, RH18 5EA

Gmail Chris Banks <bankssolutionsuk@gmail.com>

Fwd: Pre-Application Query: Tully De'Ath Flood Risk Assessment Issue 4- New Salts Farm – Shoreham 11649 For The Hyde Group

Ben Daines <ben.daines@adur-worthing.gov.uk> 31 January 2017 at 13:26
To: Chris Banks <bankssolutionsuk@gmail.com>

Chris,

Further to the comments received from West Sussex County Council and the Environment Agency yesterday, please also find comments from Ken Argent, Adur District Council's engineer regarding the New Salts Farm Flood Risk Assessment. I assume you will number this up as an Adur District Council response.

Thanks,

Ben Daines
Senior Planning Officer, Adur & Worthing Councils
Phone: 01273 263065
Email: ben.daines@adur-worthing.gov.uk
Website: http://www.adur-worthing.gov.uk/planning-policy/
Worthing Town Hall, Chapel Road, Worthing, West Sussex, BN11 1HA

---------- Forwarded message ----------
From: **Ken Argent** [redacted]>
Date: 31 January 2017 at 10:25
Subject: Re: Pre-Application Query: Tully De'Ath Flood Risk Assessment Issue 4- New Salts Farm – Shoreham 11649 For The Hyde Group
To: Ben Daines <ben.daines@adur-worthing.gov.uk>

Ben,

I have read the Submission, and am now in a position to comment upon its content. I am afraid I am no more positive about the document than my colleagues at WSCC and the EA both of whom have already commented back to you.

It is interesting that many of the items raised and minuted from the 27th June meeting have remained un-actioned by Hyde. Item 3.3 on the minutes requires both north and south ditches to be modelled, However.

Paragraph 1.2 of the FRA Version 4 states *The purpose of the report is to demonstrate to the Planners, the Environment Agency (EA) and West Sussex County Council as Lead Local Flood Authority that the proposed development is subject to an acceptable level of flood risk and should not increase the likelihood of flooding elsewhere.* The document does not consider this and indeed at paragraph 5.3 it states *Much of the catchment to the north of the site was not included as part of the analysis as it would not contribute to the flooding on the site.*

Paragraph 3.6 of the FRA Version 4 states that ground water levels were recorded for a maximum of 11 months. during which time one recorder failed for 3 months and a second for one month, a further 4 wells were installed but monitored for 3 months. This is hardly a complete study. However the fact that WLS5 was subjected to artesian flooding and WLS6 flooded to surface level twice – appears to cause very little concern. Interesting that the data on FIGURE 5 does not show the two events mentioned.

Fig 2 J12495 Depicts ground water levels these appear to show that ground water levels peak two hours after high tide, except in WS1. This has been assessed "and is probably unaffected by tidal action" I would suggest that the ground water level at this location is directly influenced by its proximity to the ditch, a small increase in recorded level is clearly seen, is it more likely that this small peak would coincide with backflow as the tide flaps close.

I attach 2 photographs taken in 2015 one is near the Location of WLS111 the second slightly west of WLS109, both would tend to show that the data loggers have not been exposed to a wet winter. Note this is post ditch clearance – so the ditches were in a good clear condition.

Paragraph 3.7 Why was the data logger not replaced? It is also stated here that *All recorded levels remained in-channel with the exception of one event in Ditch 3 where the water level was recorded just above the bank level. The same event created a similar 200mm rise in water level within Ditch 3,* clearly an error.

The data logger was installed at The Road Bridge, this location was clearly identified as a pinch point on the drainage system, Hyde acknowledged that they were the riparian owners of the ditch at this point and this was minuted at minute 4.5, No action to improve conditions was taken.

Whilst I do not dispute that the Pynford Housedeck system, is specifically designed for areas with poor ground conditions, nowhere in the literature does it say that the para 6.4 statement localized interstitial mixing of soil particles and the cement paste occurs at the soil/pile interface. This means that in the final case there is no preferential pathway for water migration in the long term, is suitable for tidally influenced artesian pressure.

Issues concerning Safe egress have been identified by others

Issues concerning Foul Sewer capacities, once they incorporate Monks Farm Flows have been identified by others.

Water butts are frequently mentioned, these may retain water in the summer, but who empties their butts during the winter, when it rains? Their capacity should be ignored..

Permeable paving and the use of geocells for road construction has been questioned previously Suds drainage systems should have 1m between ground water levels and the underside of drainage layers (soakaways). Clearly that's not feasible here.

Figure 6 J12495 depicts ground water levels at three locations and shows the tidal lag between high tide and ground water level, It should also be pointed out here that whilst Southern Water confirm that actual rainfall was almost twice the long term average for the month of January a 4.94 m tide is not a spring tide indeed it is almost the lowest high tide level achievable all year. Why was the graph for Monday 10th January when the predicted high tide level was 6.1m not be provided

Ken Argent
Engineer, Adur & Worthing Councils
Phone[redacted]
Email: [redacted]
Website: www.adur-worthing.gov.uk
Address: Engineering Team, Worthing Town Hall, Chapel Road, Worthing, West Sussex, BN11 1HA

Andrew Picton

From: Dinny Shaw <DinnyShaw@boyerplanning.co.uk>
Sent: 22 March 2017 16:01
To: Julian Turner; Andrew Picton; Sarah Poulter <sarah. poulter@hyde-housing. co. uk>; Andrew Williams; tom.shaw@hyde-housing.co.uk
Subject: Fwd: New Salts Farm FRA ongoing correspondence
Attachments: Aerial flooding Mar 2014.JPG

And below are Ken's comments. There do appear to be some concessions here.

Andrew / Julian - please can you review as with WSCC comments

Thank you

Dinny

Sent from my iPhone

Begin forwarded message:

From: Ben Daines <ben.daines@adur-worthing.gov.uk>
Date: 22 March 2017 at 15:55:20 GMT
To: Dinny Shaw <DinnyShaw@boyerplanning.co.uk>
Cc: James Appleton <james.appleton@adur-worthing.gov.uk>, Moira Hayes <moira.hayes@adur-worthing.gov.uk>, Ray Drabble <Ray.Drabble@westsussex.gov.uk>, Ken Argent <Ken.Argent@adur-worthing.gov.uk>
Subject: New Salts Farm FRA ongoing correspondence

Dinny,

Please see the response from Ken Argent below:

I have quickly read through Ray's letter to Andrew Picton dated 28 February, and the response from Dinny Shaw dated 10th March, received whilst I was on leave.

It is obvious that Ray has been treated to more data than I have but I understand most of his arguments, I will use the same numbering that both he and Dinny have used.

1.1 I have no problems with the differing volumes of storage, but, the statement that storage may be reduced because of infiltration is a must, how else will the stored water be disposed of? My concern is that this year ground water did not erupt along the A27 therefore in my opinion we had low ground water levels, therefore the site has not experienced a hard year.

1.2 I accept that with no habitable accommodation on the ground floor flooding is not an issue - except that you are acknowledging building homes that may be damp / wet for periods of the year. I see no issue raising ground levels for 3-4 properties.

1.4.1 what is the point of lining the detention basin, surely you want infiltration to drain it? Or is this to be a basin with a limited outflow, which means a wet area with a clay bottom for kids to play in and perforate?

Andrew Picton

From: Dinny Shaw <DinnyShaw@boyerplanning.co.uk>
Sent: 10 March 2017 21:56
To: ray.drabble@westsussex.gov.uk; ben.daines@adur-worthing.gov.uk
Cc: Andrew Picton; Julian Turner
Subject: Fwd: NSF - Tully De'Ath response to Comments from WSCC and the EA
Attachments: image001.jpg; image001.jpg; image001.jpg; image001.jpg; image002.png; image003.png

Ray / Ben

Please see below latest correspondence from EA regarding New Salts Farm for your information.

Kind regards

Dinny

Sent from my iPhone

Begin forwarded message:

> **From:** "Griggs, David" <David.Griggs@environment-agency.gov.uk>
> **Date:** 10 March 2017 at 15:17:42 GMT
> **To:** Dinny Shaw <DinnyShaw@boyerplanning.co.uk>
> **Cc:** "'ajp@tullydeath.com'" <ajp@tullydeath.com>, "Julian Turner (jct@tullydeath.com)" <jct@tullydeath.com>
> **Subject: RE: FW: NSF - Tully De'Ath response to Comments from WSCC and the EA**
>
> Hi Dinny,
>
> Providing that finished floor levels to all habitable accommodation can be provided above 5.42mAOD to mitigate the tidal flood risks at the site, we would likely have no objections to the New Salts Farm development on flood risk grounds.
>
> Although there are potentially other environmental issues that we have not considered in depth at this stage (e.g. biodiversity, groundwater protection), we have not identified any critical uncertainties with regards to these that would preclude residential development at the site.
>
> Kind regards,
>
> **David Griggs**
> **Planning Advisor | Sustainable Places**
> Environment Agency | Solent & South Downs
> Telephone: 02030 259625
>
> **Environment Agency**
> Romsey Office
> Canal Walk
> Romsey
> SO51 7LP
>
> Our **climate change allowances** for planning were updated on 19 February 2016. The guidance is accessible here: Flood risk assessment: Climate change allowances

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Ms S Poulter
Hyde Housing Association
6th Floor, Telecom House
121-135 Preston Road
Brighton
BN1 6AF

19 April 2017
Your Ref.
Our Ref. P108116

Dear Sarah

NEW SALTS FARM, FLOOD RISK ASSESSMENT

I refer to recent discussions with regards to the flood risk assessments for the New Salts Farm site in West Sussex. Consultants' reports have been prepared and these have been reviewed by BRE in order to form an opinion on the management of flood risk at the proposed development. The site is potentially exposed to all sources of flooding including coastal, river, surface water and groundwater.

Significant analysis of the risks has been undertaken in support of the development, with exchanges between the planning authorities and the developer since submission of the flood risk assessment (FRA) in December 2016. The range of reports and correspondence have been reviewed by BRE. It is understood that the main concerns of the planning authority focus on surface water and groundwater risks, with measures in place in order to manage coastal and river risks. The latter issues are not considered within this letter.

The proposed site development includes measures to include allowance for seal level rise of up to 96 mm and its impact on groundwater levels. This has an impact on the bioretention basin but even under the '1%AEP plus climate change' it has capacity for storage.

The design of buildings and the management of the built environment includes measures such as raised principal living level and use of sustainable drainage to help manage groundwater and surface water flood risk. At present the detailed building designs are not available and therefore this matter would need to be revisited at a later stage, however, the principle is accepted by BRE as meeting the need for resilience in flood risk areas. The detailed building design and construction would be required to meet all requirements of building regulations, any measures to manage flood risk should not compromise other aspects of performance and building use.

For the design of the bioretention basin lining has previously been proposed as a solution to comments made by West Sussex County Council regarding the effect of ground water levels. However, Hyde Housing and their consultants have undertaken geotechnical testing, which have demonstrated that there is infiltration potential in the natural soils. Removing the liner allows the infiltration potential to be used. To use as much of the infiltration potential as possible the run-off needs to be temporarily retained in the basin rather than immediately discharged to the channel. Measures have been included to allow this to function effectively, there will be no discharge to the channel for smaller more frequent storm events.

BRE considers that the proposed drainage strategy for the site appears to work under the modelling scenarios undertaken, which include allowances for climate change. The inclusion of the bioretention basin is key to the drainage strategy, but additional measures to increase the permeability of the developed ground will also help to manage surface water flood risk.

The inclusion of permeable pavements and swales are suitable SuDS devices for inclusion in the drainage strategy; both these measures are recognised SuDS measures. It is understood that neither measure was specifically included within the modelling of surface water flood risk; instead a zero infiltration assessment was included. However, well designed, constructed and maintained SuDS measures that are part of an overall system should be add further to the management of water on site. The impact of each measure on source control, site control and regional control should be considered for the SuDS system. The SuDS should be designed in accordance with the CIRIA Manual or equivalent guidance.

BRE has been provided with all the issues raised by West Susses County Council as the Local Lead Flood Authority. This includes the concerns over surface water and ground water flood risk. Hyde Housing Association and their consultants have undertaken modelling and produced evidence to suggest that the flood risk to the proposed development and the impact of the development have been adequately addressed to date. The modelling undertaken includes climate change factors.

BRE’s opinion is that the flood risk can be managed on the site using the approaches set out by Hyde Housing Association and their consultants. However, BRE would suggest some further work should be undertaken to inform detailed design development of the site and to consolidate and add to the current evidence base. The additional work would include the following:

- Surface water risk: an estimate of the storage requirements for the SUDS design and a comparison of the greenfield and developed site design hydrographs.

To support detailed discussions going forward and a full planning application a single report containing all flood risk assessment and modelling data would be useful in order to set out all flood risk on the site and the measures to manage that risk. It will also assist communication between the parties involved. BRE has provided this initial assessment based on available and supplied evidence to date in order to assist ongoing planning procedures. The review considers only the New Salts Farm development and not any neighbouring sites.

BRE has no responsibility for the site development or the modelling, design or construction of the buildings or infrastructure on the site. BRE does not carry out design work, but has used the available evidence to form an opinion on the proposed approach.

Yours sincerely

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