TALBOT HOTEL, CHIPPING

FLOOD RISK ASSESSMENT

Final Report v2.0

September 2012

Weetwood
4 Queen St
Leeds
LS1 2TW

T: 0113 244 1377
E:info@weetwood.net
www.weetwood.net

Report Title:	**Talbot Hotel, Chipping** Flood Risk Assessment Final Report v2.0
Client:	The Talbot at Chipping Ltd
Date of Issue:	27 September 2012

Prepared by:	Yeshentha Naidoo BEng (Hons) MSc *Technical Engineer*
Checked by:	Jenny Cavill BSc (Int) *Senior Technical Engineer*
Approved by:	Kevin Tilford BSc MSc (Eng) PhD MBA C.WEM MCIWEM *Director*

CONTENTS

	Page
Signature Sheet	i
Contents	ii
List of Tables, Figures & Appendices	iii

1 INTRODUCTION 1

1.1 Site Location 1
1.2 Existing and Proposed Development 1
1.3 Site Levels 2
1.4 Access and Egress 2

2 NATIONAL PLANNING POLICY FRAMEWORK (NPPF) 3

2.1 Flood Zone Designation 3
2.2 Sequential Test 5
2.3 Exception Test 5

3 FLOOD RISK 6

3.1 Chipping Brook 6
3.2 Reservoirs, Canals and Other Artificial Sources 7
3.3 Groundwater 8
3.4 Surface Water 9

4 MITIGATION MEASURES 11

4.1 Flood Mitigation 11
4.2 Compensatory Storage 11
4.3 Access and Egress 12

5 SURFACE WATER 13

5.1 Requirements for Surface Water Drainage 13
5.2 Site Areas 13
5.3 Surface Water Runoff from the Existing Site 13
5.4 Surface Water Runoff from the Redeveloped Site 14

6 SUMMARY 18

7 RECOMMENDATIONS 19

LIST OF TABLES

Table 1: Compensatory Storage Volume - 1 in 100 year Flood Level 11
Table 2: Compensatory Storage Volume - 1 in 1,000 year Flood Level 11
Table 3: Site Areas 13
Table 4: Total Peak Runoff Rate – Existing Site 14
Table 5: Total Peak Runoff Rate – Redeveloped Site 14

LIST OF FIGURES

Figure 1: Site Location.. 1
Figure 2: Environment Agency Flood Map.. 4
Figure 3: Zone 2 and 3 Flood Outlines 5
Figure 4: Detailed Site Flood Outlines 6
Figure 5: Environment Agency Risk of Flooding from Reservoirs Map 8
Figure 6: BGS Groundwater Flooding Hazard Map 9
Figure 7: Potential Compensatory Flood Storage Area.. 12

LIST OF APPENDICES

Appendix A: Topographic Survey
Appendix B: Development Proposals
Appendix C: Modified Rational Method Calculation
Appendix D: Micro Drainage Outputs for Greenfield Runoff
Appendix E: Micro Drainage Storage Volume Calculation – Tank
Appendix F: Micro Drainage Storage Volume Calculation – Cellular Storage

1 INTRODUCTION

Weetwood has been instructed[1] by The Talbot at Chipping Ltd to undertake a Flood Risk Assessment (FRA) for the proposed alterations and extension to the Talbot Hotel and barn in Chipping, in accordance with the requirements of the National Planning Policy Framework (NPPF) and its supporting Technical Guidance.

1.1 SITE LOCATION

The site is located on Talbot Street in Chipping at Ordnance Survey National Grid Reference SD 6227 4335, as shown in **Figure 1**.

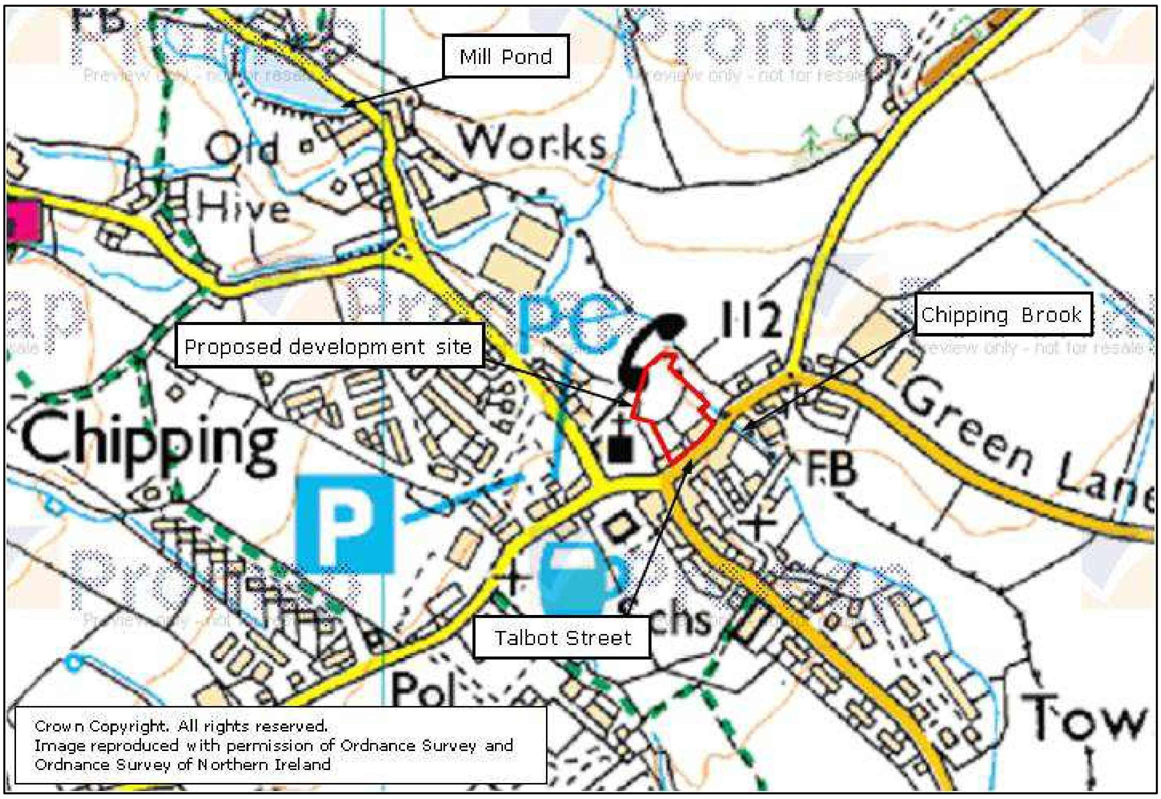

Figure 1: Site Location

1.2 EXISTING AND PROPOSED DEVELOPMENT

The existing site comprises the Talbot Hotel and a barn, as shown in **Appendix A**.

The proposals are for alterations to the Talbot Hotel with an extension to the rear and redevelopment of the barn for residential use (see **Appendix B**). Hotels and residential buildings are classified as *'more vulnerable development'* in Table 2 of the NPPF Technical Guidance.

[1] Acceptance form dated 20 July 2011, Ref: 1937/110718/CC/FP1

1.3 SITE LEVELS

A topographic survey of the site was undertaken by Malcolm Hughes Land Surveyors in June 2011 and is provided in **Appendix A**. According to the topographic survey, site levels range from approximately 110.7 metres above Ordnance Datum (m AOD) in the east, to approximately 112.7 m AOD in the west. An embankment along the west boundary rises up to levels of around 115.7 to 118.6 m AOD.

1.4 ACCESS AND EGRESS

The existing site is accessed via Talbot Street.

Levels along this section of Talbot Street range around 113.44 m AOD in the west to 110.73 m AOD in the east.

2 NATIONAL PLANNING POLICY FRAMEWORK (NPPF)

The aim of the NPPF and its supporting Technical Guidance is to ensure that flood risk is taken into account at all stages in the planning process and is appropriately addressed.

2.1 FLOOD ZONE DESIGNATION

Table 1 of the NPPF provides the definitions for each of the flood zones, which are summarised as follows:

- **Flood Zone 1: Low Probability.** Land assessed as having a less than 1 in 1000 annual probability of river or sea flooding in any year.
- **Flood Zone 2: Medium Probability**. Land assessed as having between a 1 in 100 and 1 in 1000 annual probability of river flooding or between a 1 in 200 and 1 in 1000 annual probability of flooding from the sea in any year.
- **Flood Zone 3a: High Probability.** Land assessed as having a 1 in 100 or greater annual probability of river flooding (>1%) or a 1 in 200 or greater annual probability of flooding from the sea (>0.5%) in any year.
- **Flood Zone 3b: The Functional Floodplain.** Land where water has to flow or be stored in times of flood. The identification of the functional floodplain should take account of local circumstance and not be defined solely on rigid probability parameters. However, land which would flood with an annual probability of 1 in 20 or greater in any year should provide a starting point for consideration and discussion.

2.1.1 Environment Agency Flood Map

According to the Environment Agency (EA) Flood Map (**Figure 2**) the majority of the site is located in Flood Zone 1. Some land along the north east boundary of the site appears to be located in Flood Zones 2 and 3. The EA flood map does not differentiate between Flood Zone 3a and Flood Zone 3b. The NPPF states that a Strategic Flood Risk Assessment (SFRA) should identify this flood zone.

National generalised modelling (NGM) has been used to produce the flood outlines in the vicinity of the proposed development. NGM is used by the EA to generate flood outlines when more detailed flood modelling and mapping is not available. NGM has a number of limitations which can result in inaccuracies in the flood outlines. The EA has further advised that, although more detailed modelling of this reach of Chipping Brook is planned, no timescale has been set for commencement of the modelling. An e-mail from the EA dated 8 April 2011 states "*The proposed modelling for the Chipping area will not take place this financial year*". The EA Development and Flood Risk Officer was asked whether a FRA could be prepared based on existing information (including estimating the 1 in 100 year water levels by comparison of the NGM flood outlines with the topographic survey). The EA responded[2] by stating that "*I am prepared to accept an estimated 100 year flood flow in your FRA*".

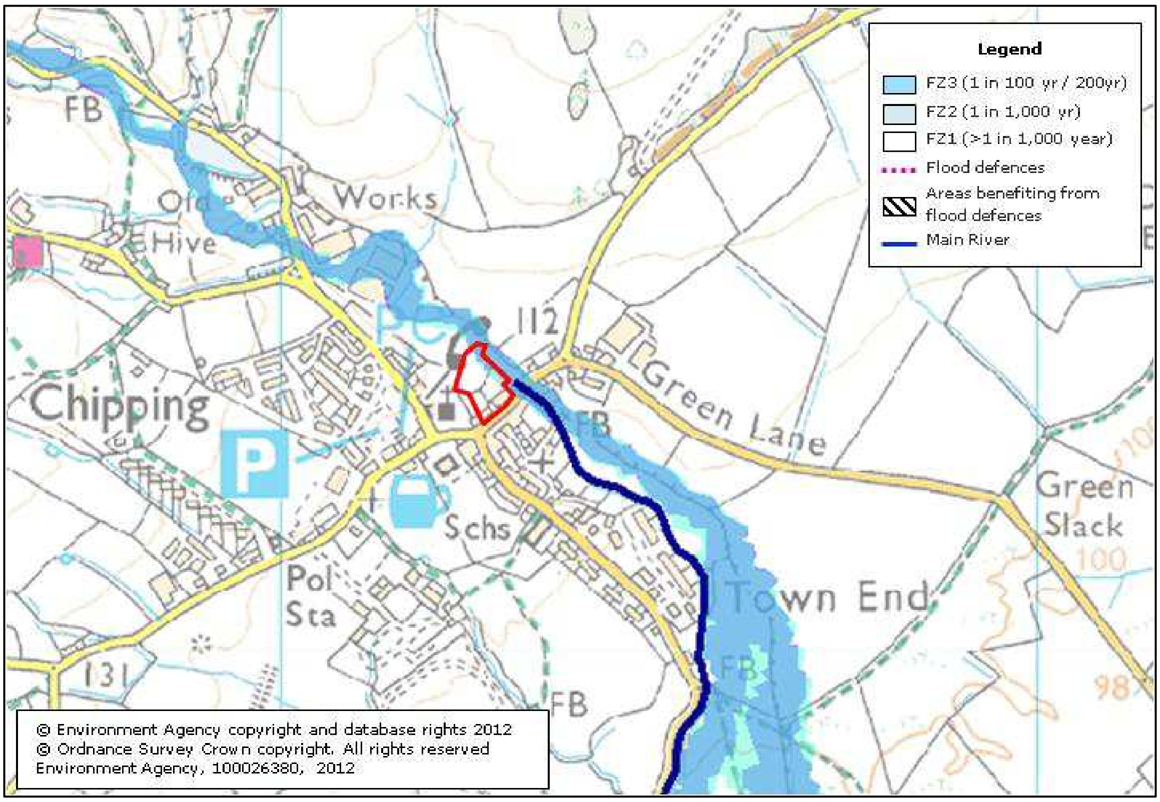

Figure 2: Environment Agency Flood Map

(Source: Environment Agency website)

Figure 3 shows the NGM flood outlines superimposed on an aerial photograph of the site. This is for illustrative purposes only and it does appear to confirm that the northeast boundary of the site is located in Flood Zones 2 and 3.

[2] E-mail from C Welsby (EA) to C Cornmell (Weetwood) dated 18 July2011

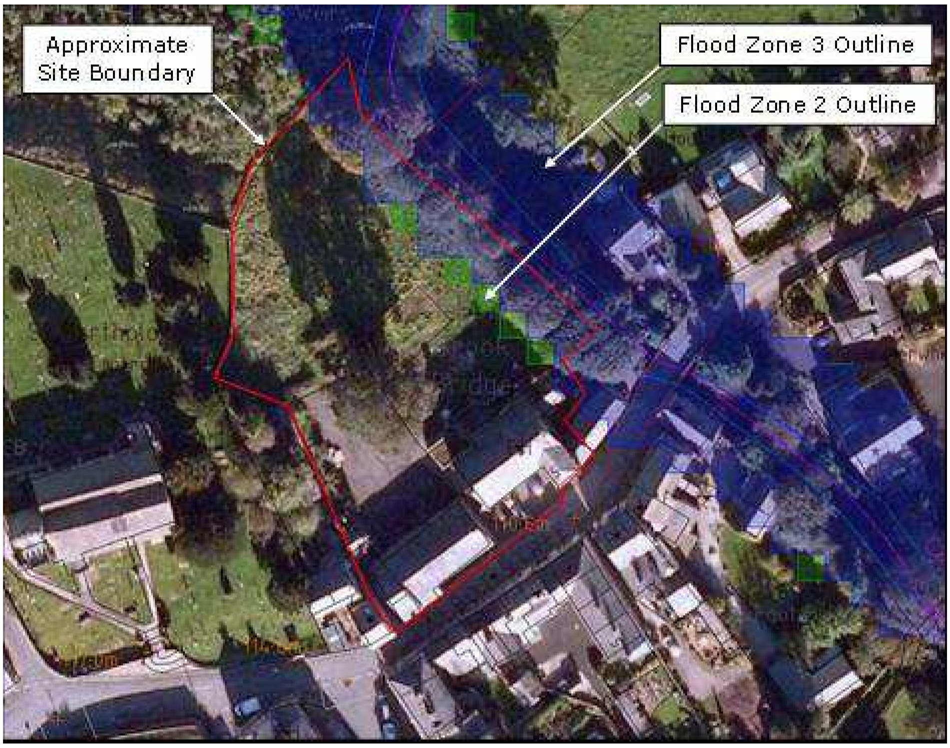

Figure 3: Zone 2 and 3 Flood Outlines

2.1.2 Strategic Flood Risk Assessment

A Level 1 SFRA was published by Ribble Valley Borough Council (RVBC) in May 2010. The SFRA has been reviewed and the information therein has been used to inform this FRA.

2.2 SEQUENTIAL TEST

The aim of the Sequential Test (as outlined in Chapter 10 of the NPPF and paragraphs 3-5 of the Technical Guidance) is to encourage development to be located in areas at the lowest probability of flooding. The Sequential Test requirements at the site are discussed in further detail in **Section 3.1**.

2.3 EXCEPTION TEST

The Exception Test should be applied for 'more vulnerable' development within Flood Zone 3. Although the hotel is classified as a 'more vulnerable' development, it should be noted that the parts of the hotel extension which appear to be located within Flood Zone 3 are a store room, function room and kitchen. Buildings used for storage, assembly, leisure, restaurants and cafés are classified as 'less vulnerable' development according to Table 2 of the NPPF Technical Guidance. The Exception Test requirements are also discussed in further detail in **Section 3.1**.

3 FLOOD RISK

3.1 CHIPPING BROOK

Chipping Brook flows in a south-easterly direction along the north east boundary of the site.

3.1.1 Estimated Water Level and Flood Zone Classification

As discussed, the EA flood outlines in the vicinity of the proposed development site as shown in **Figure 2** and **Figure 3** have been produced by NGM. The 1 in 100 year and 1 in 1,000 year flood levels have been estimated by superimposing the current EA flood map onto the topographic survey of the site in **Appendix B**. The results are shown in **Figure 4**.

The maximum ground level within the EA's flood outline in the vicinity of the proposed extension is 111.24 m AOD. The level falls to 110.68 m AOD in the south (i.e. downstream end) of the site. The 1 in 100 year water level is between 111.24 m AOD and 110.68 m AOD and has been estimated as 111.00 m AOD. Likewise, the 1 in 1,000 year flood level at the site is estimated at 111.12 m AOD.

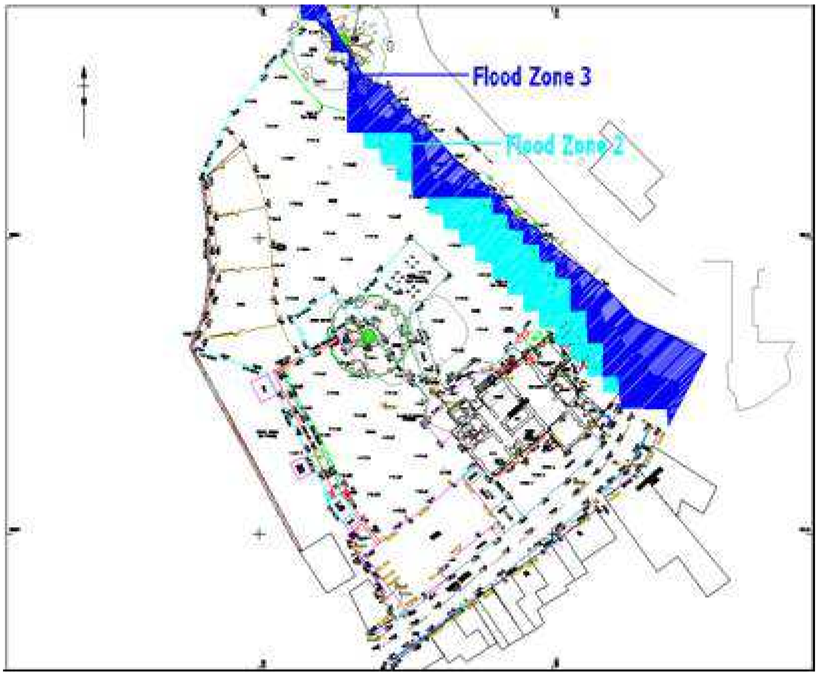

Figure 4: Detailed Site Flood Outlines

3.1.2 Discussion of Results

The barn, existing hotel and the majority of the proposed hotel extension are shown to be located in Flood Zone 1. These aspects of the development therefore satisfy the Sequential Test.

A small portion of the extension will be located within Flood Zones 2 and 3. Paragraph 4.14 of the SFRA states "*Following discussion with the EA, it is proposed that all **rural/undeveloped** sites within Flood Zone 3 should, at this stage, be identified as "potential" Flood Zone 3b*". The site is, however, already developed and therefore does not meet this definition for "potential" Flood Zone 3b areas. It is concluded that the areas of the site within Flood Zone 3b should be classified as Zone 3b developed.

The parts of the hotel extension which are located within Flood Zone 3 are a store room, function room and kitchen. This type of development is classified as 'less vulnerable' development according to Table 2 of the NPPF Technical Guidance. The proposed extension cannot be located entirely outside of Flood Zone 3 due to other constraints at the site, particularly the root protection zones for the trees within the site. As the development cannot be located elsewhere, it is concluded that the requirements of the Sequential Test are satisfied. The Exception Test is not required for "less vulnerable" development within Flood Zone 3.

3.1.3 Historical Flood Records

The EA confirmed[3] that they do not hold any records of historic flooding at the site. No historic flood records for Chipping are recorded in the SFRA (paragraph 4.4 and Table 1 of the SFRA).

The British Hydrological Society (BHS) Chronology[4] has one record of flooding in Chipping in 1851, as follows:

"*In the summer of 1851 Chipping was hit by a destructive and unique flood. The flood was quick, localised and all but put John Evans [the owner of Kirk Mill] out of business. Alfred Weld, a local landowner, later recalled that 'when the flood came down, it presented a perpendicular breast of two yards in height'. The flood was responsible for the gash in the flank of Parlick [Fell] and wreaked havoc throughout the village. Pots and pans were carried down the valley; Kirk Mill was four feet six inches deep in water. A mark was left on the side of the Talbot [inn] at the flood's highest point. Wooden bridges over Chipping Brook were washed away and the stone bridges were severely damaged.*"

This event occurred 160 years ago and no details of the contributing factors which caused this flood event are available. The catchments and watercourses may have undergone significant changes since this event took place.

3.2 RESERVOIRS, CANALS AND OTHER ARTIFICIAL SOURCES

Reservoir or canal flooding may occur as a result of the facility being overwhelmed and/or as a result of dam or bank failure.

[3] E-mail from A Cottam (EA) to C Cornmell (Weetwood) dated 24 August 2011
[4] British Hydrological Society Chronology http://www.dundee.ac.uk/geography/cbhe/

The EA's *'risk of flooding from reservoirs'* map indicates that the site is not at risk of flooding from reservoirs (**Figure 5**). However, a mill pond is located approximately 400 m to the northwest of the site. In the event of a breach of the mill pond's retaining bank structure, flows would be intercepted by Chipping Brook. The volume of water in the pond is negligible in comparison with the flows that would be experienced in Chipping Brook. It is concluded that the site is not at risk of flooding from canal/reservoir flooding.

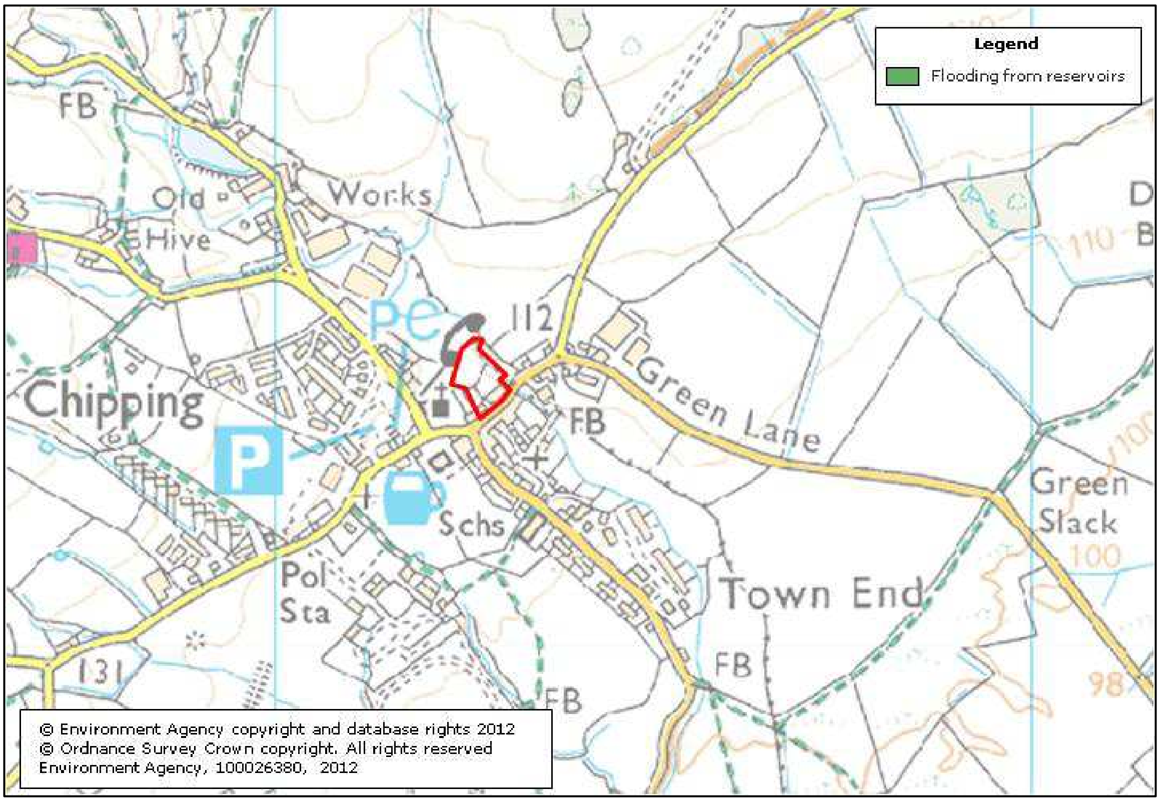

Figure 5: Environment Agency Risk of Flooding from Reservoirs Map

(Source: Environment Agency website)

3.3 GROUNDWATER

Groundwater flooding generally occurs during intense, long-duration rainfall events, when infiltration of rainwater into the ground raises the level of the water table until it exceeds ground levels. It is most common in low-lying areas overlain by permeable soils and permeable geology, or in areas with a naturally high water table.

The SFRA states that groundwater flooding "*is not considered by the Environment Agency to be a significant flood risk factor in the RVBC area*".

The British Geological Society Groundwater Flooding Susceptibility Map (**Figure 6**) indicates that the site is at low susceptibility to groundwater flooding.

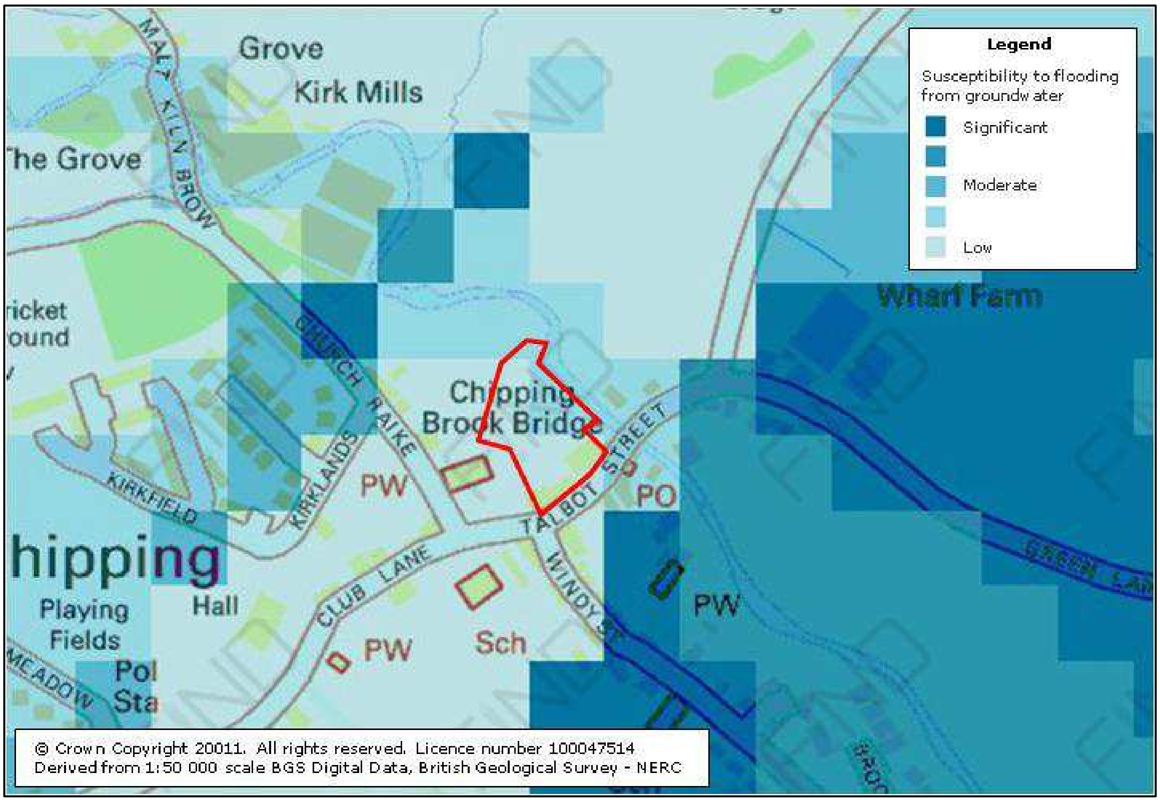

Figure 6: BGS Groundwater Flooding Hazard Map

(Source: Findmaps Website)

3.4 SURFACE WATER

Surface water flooding comprises pluvial, sewer and highway drains and gullies.

3.4.1 Pluvial

Pluvial flooding results from rainfall-generated overland flow, before the runoff enters any watercourse or sewer, or where the sewerage/drainage systems and watercourses are overwhelmed and therefore unable to accept surface water. Pluvial flooding is usually associated with high intensity rainfall events but may also occur with lower intensity rainfall where the ground is saturated, developed or otherwise has low permeability resulting in overland flow and ponding within depressions in the topography.

The Soilscapes maps produced by the National Soils Research Institute at Cranfield University[5] indicate that the site is located on slowly permeable, seasonally wet loamy and clayey soil, with impeded drainage. However, Chipping Brook is located along the northeast boundary of the site, therefore any overland flow of floodwaters would be expected to be directed into Chipping Brook.

[5] Soilscapes http://www.landis.org.uk/soilscapes/

3.4.2 Sewer

Sewer flooding can occur when the capacity of the sewer system is overwhelmed by heavy rainfall, becomes blocked or is of inadequate capacity, resulting in flooding of land and/or property. Normal discharge of sewers and drains through outfalls may be impeded by high water levels in receiving waters.

United Utilities (UU) stated in an e-mail dated 28 July 2011 "*we have no record of public sewer flooding of properties in this vicinity as a result of overloaded sewers. i.e. no properties on the 'at risk' register as compiled for our Regulator.*"

3.4.3 Highway Drains and Gullies

Lancashire County Council confirmed in an e-mail dated 28 July 2011 "*There are no known problems with the highway drainage on Talbot Street in Chipping.*"

4 MITIGATION MEASURES

4.1 FLOOD MITIGATION

Recognising that the proposals are to extend the existing hotel, the client has indicated that the finished floor level of the proposed buildings needs to be set at 111.60 m AOD in order to match the floor level of the existing building. This level is 600 mm above the estimated 1 in 100 year flood level determined in **Section 3.1**.

4.2 COMPENSATORY STORAGE

It must be shown that there will be no loss of flood storage capacity at the site as a result of development. This is to ensure that flood risk is not increased elsewhere.

Approximately 28 m^2 of the proposed extension footprint is shown to encroach into the existing Flood Zone 3 outline. The total volume of water that may be displaced by the proposed extension in a 1 in 100 year event has been estimated in **Table 1**. Compensatory storage must be provided elsewhere on the site to offset the loss of floodplain storage.

Table 1: Compensatory Storage Volume - 1 in 100 year Flood Level

Area in FZ3 (m^2)	Existing Ground Level (m AOD)	100yr Water level (m AOD)	Water Depth (m)	Potential Volume Displace (m^3)
27.8	110.99	111.00	0.01	0.14

Normally, compensatory storage would only be provided for up to the 1 in 100 year plus climate change event. However, in this case the 1 in 100 year plus climate change flood level is not known. Compensatory storage should therefore be provided for up to the 1 in 1,000 year flood level in order to ensure that the proposed development does not increase flood risk elsewhere. The additional storage volume required is calculated in **Table 2**.

Table 2: Compensatory Storage Volume - 1 in 1,000 year Flood Level

Area in FZ2 (m^2)	1000yr Water level (m AOD)	100yr Water level (m AOD)	Water Depth (m)	Potential Volume Displaced (m^3)
115.0	111.12	111.00	0.12	8.6

Recognising the calculations in **Table 1** and **Table 2**, land at the site post development should be re-profiled such that an additional 0.14 m^3 of storage is provided at a level of 110.99-111.00 m AOD and 8.6 m^3 of storage is provided at a level of 111.00-111.12 m AOD.

An area currently outside of (but with connectivity to) Flood Zone 3 is required for compensatory storage. An area to the north of the proposed extension (as shown in red in **Figure 7**) should be lowered in order to provide compensatory flood storage. Some re-profiling within the Flood Zone 3 area may be required to ensure that floodwaters would naturally return to the channel. An alternative location may need to be found if excavations at this point would have a detrimental impact on tree roots.

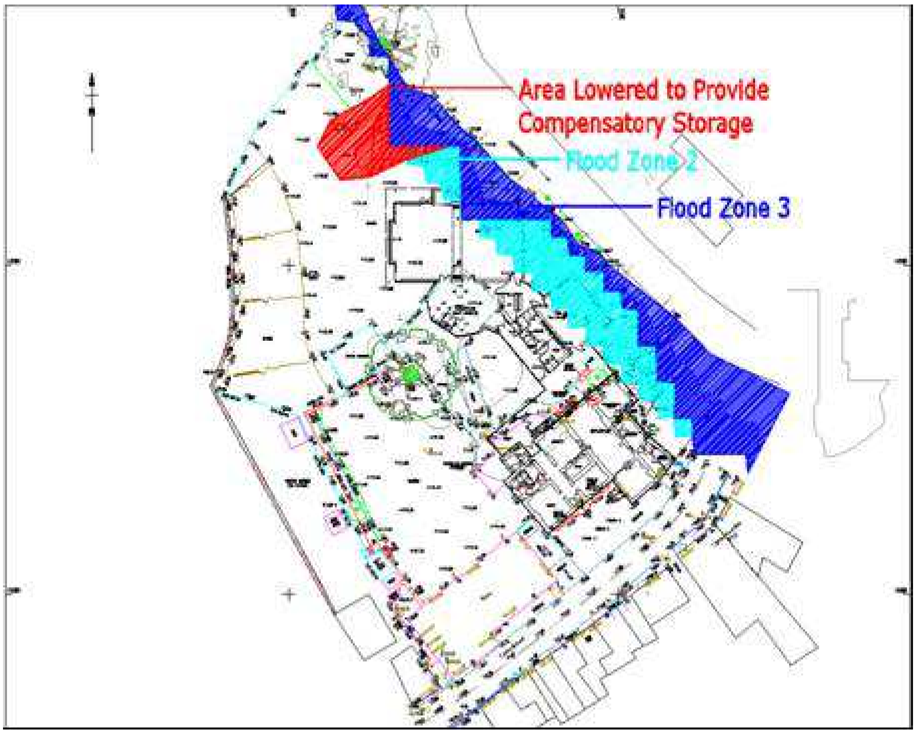

Figure 7: Potential Compensatory Flood Storage Area

4.3 ACCESS AND EGRESS

Dry access and egress to the site can be provided via Talbot Street, which runs west into areas outside the floodplain.

5 SURFACE WATER

5.1 REQUIREMENTS FOR SURFACE WATER DRAINAGE

Surface water arising from a developed site should, as far as is practicable, be managed in a sustainable manner to mimic the surface water flows arising from the site prior to the proposed development.

Development of the site should be such that the peak flow rates of surface water leaving the developed site are no greater than the rates prior to development. Opportunities to reduce surface water runoff, and the associated flood risk, should also be identified and climate change taken into consideration.

Recognising the above, and the requirements of the EA[6], Building Regulations Approved Document H, the Code for Sustainable Homes Technical Guide (Category 4) and the requirement placed upon local planning authorities within the NPPF and its supporting Technical Guidance to promote the use of Sustainable Drainage Systems (SUDS), surface water runoff from the proposed site should demonstrate:

- No increase in existing flow rates discharged to watercourse/public sewer
- The use of SUDS as the preferred method of dealing with surface water
- How runoff up to the 1 in 100 year event plus an allowance for climate change will be dealt with without increasing flood risk elsewhere

5.2 SITE AREAS

The existing and proposed impermeable and permeable areas at the site are shown in **Table 3**. This indicates that the extent of impermeable area at the site will increase by 0.134 ha following redevelopment.

Table 3: Site Areas

	Existing Site	Redeveloped Site
Impermeable Area (ha)	0.222	0.356
Permeable Area (ha)	0.248	0.114
TOTAL	0.470	0.470

5.3 SURFACE WATER RUNOFF FROM THE EXISTING SITE

The UU public sewer record indicates that a combined sewer is located along the north-east boundary of the site. However it is assumed that surface water is more likely to be discharge directly into Chipping Brook.

5.3.1 Existing Site Runoff Rate

The peak runoff rates for the existing site are summarised in **Table 4**.

[6] Preliminary Rainfall Runoff Management for Developments, R&D Technical Report W5-074/A/TR/1 Revision C, 2005

- The Modified Rational Method[7] has been used to calculate the runoff from the impermeable surfaces at the existing site, as detailed in **Appendix C**.
- Greenfield runoff rates from permeable surfaces have been calculated using the ICP SUDS method within Micro Drainage. Details of the Micro Drainage input parameters and the output results are provided in **Appendix D.**

Table 4: Total Peak Runoff Rate – Existing Site

Return Period	Runoff Rate from Impermeable areas (l/s)	Runoff Rate from Permeable areas (l/s)	Total Peak Runoff Rate from Existing Site (l/s)
1 in 2 year	46.9	0.5	47.4
1 in 30 year	84.7	0.9	85.6
1 in 100 year	106.3	1.1	107.4

5.4 SURFACE WATER RUNOFF FROM THE REDEVELOPED SITE

Table 3 indicates that impermeable areas at the site will increase following redevelopment. The following sections describe how surface water runoff from the redeveloped site may be managed in accordance with the requirements of NPPF and its supporting Technical Guidance.

5.4.1 Surface Water Discharge Rate

The surface water drainage arrangements for any site should be such that the peak flow rates of surface water leaving a developed site are no greater than the rates prior to the proposed development.

It is proposed to limit runoff rates from the proposed impermeable areas to **46.9 l/s**. This is the existing 1 in 2 year flow rate from the existing impermeable areas, as calculated in **Appendix C** and shown in **Table 4**. This will ensure that rates of runoff from the site do not increase following redevelopment. The drainage system for the proposed site will be designed to manage flows in up to the 1 in 100 year event including an allowance for climate change. The existing permeable areas will continue to drain at greenfield runoff rates.

The total proposed peak runoff rates from the proposed site are shown in **Table 5**.

Table 5: Total Peak Runoff Rate – Redeveloped Site

Return Period	Runoff Rate from Impermeable areas (l/s)	Runoff Rate from Permeable area (l/s)	Total Peak Runoff Rate from Proposed Site (l/s)
1 in 2 year	46.9	0.2	47.1
1 in 30 year	46.9	0.4	47.3
1 in 100 year	46.9	0.5	47.4

[7] The Wallingford Procedure, Volume 4, 1981

Comparison of the total peak runoff rates from the existing site (**Table 4**) with those from the redeveloped site (**Table 5**) indicates that redevelopment will provide for significant betterment in terms of reduced surface water flows as encouraged by the NPPF and its supporting Technical Guidance.

5.4.2 Disposal of Surface Water

Building Regulations Approved Document Part H sets out a hierarchy of preferred methods for the disposal of surface water runoff[8]. These are listed below in order of preference:

1. Disposal by infiltration – According to the Soilscapes maps produced by the National Soils Research Institute at the Cranfield University[9], soil conditions at the site are described as "*slowly permeable, seasonally wet, acid, loamy and clayey soils, with impeded drainage*". No additional soakaway tests or site investigation work has been undertaken at the site. On this basis, infiltration methods are not considered suitable for the disposal of surface water from the site.
2. Disposal to a watercourse – It is assumed that runoff from the existing site currently discharges to Chipping Brook. As recommended by the building regulatins hierarchy, in the absence of suitable conditions for infiltration, surface water from the developed site shall be discharged to Chipping Brook. Land drainage consent will be needed for any new outfalls.
3. Disposal to a public sewer – Following redevelopment of the site it should not be necessary to discharge surface water runoff into the public sewer system.

5.4.3 SUDS Options and Storage Calculations

In order to restrict runoff rates from the proposed impermeable areas as set out in **Section 5.4.1**, attenuation storage will be provided. SUDS elements may be used to provide the required storage.

SUDS aim to mimic natural drainage and can achieve multiple objectives such as removing pollutants from urban runoff at source, controlling surface water runoff from developments, and ensuring that flood risk is not increased downstream. Combining water management with green space can provide amenity and biodiversity enhancement. Typical SUDS components include surface or subsurface storage with flow limiting devices, roadside swales, detention basins and infiltration areas or soakaways.

The surface water storage facilities described in the following sections have been modelled using the Detailed Design module of Micro Drainage Source Control.

[8] Building Regulations Approved Document H Section 3 page 45
[9] http://www.landis.org.uk/soilscapes/

5.4.3.1 Storage Volume Calculation

The required storage volume has been sized to store the 1 in 100 year storm event including a 30% increase in rainfall intensity in order to allow for climate change in accordance with Table 5 of the NPPF Technical Guidance.

The parameters used in the storage calculation along with the Micro Drainage Source Control output results are provided in **Appendix E**. This indicates that a storage volume of **61.7 m^3** would be required.

The development will provide 42 no car parking spaces occupying an area of around 480 m^2. A porous sub base with 30% porosity and 450 mm deep would provide 64.8 m^3 of storage.

Alternatively, cellular storage could be provided beneath the proposed parking areas. The depth of storage units modelled is 520 mm. The results are provided in **Appendix F** and indicate a storage volume requirement of 47.5 m^3 with the storage units filling to a depth of 0.104 m.

5.4.4 Maintenance of SUDS

In the past local planning authorities and water companies have been reluctant to adopt SUDS. With no arrangements in place that require local planning authorities or water companies to adopt SUDS their maintenance has subsequently been the responsibility of the developer.

The Flood and Water Management Act (2010) is currently being implemented through a series of Commencement Orders. Section 32 introduces Schedule 3: Sustainable Drainage. This introduces:

- New standards for the design, construction, operation and maintenance of new rainwater drainage systems
- A new 'approving body' (generally a unitary, county or county borough local authority)
- A requirement for the approving body to approve most types of rainwater drainage systems before any construction work with drainage implications can start, subject to: (i) the system being constructed in line with an approved drainage plan to national standards; (ii) the approving body being satisfied the drainage system has been built and functions in accordance with the drainage plan, and (iii) the system being a sustainable drainage system, as defined by regulations.

However, this provision is awaiting commencement following further work by DEFRA on arrangements for adoption and maintenance of SUDS, including technical guidance. At present it is envisaged that implementation of these arrangements will be Autumn 2012.

In the meantime, other options for maintenance of SUDS include:

- SUDS elements within the curtilage of residential dwellings (e.g. soakaways, permeable paving) will be the responsibility of the owner of the property.

- The pipe network, designed to Sewers for Adoption (6^{th} edition) standard, will be adopted by the sewerage undertaker.

5.4.5 Final Drainage Layout

The purpose of this FRA is to demonstrate that a surface water drainage strategy is feasible for the site given the development proposals and the land available. The proposals provide the opportunity for the inclusion of SUDS elements, ensuring that there will be no increase in surface water runoff from the proposed development.

This FRA has demonstrated that, not only can the required storage be accommodated within the site layout, but that various options are feasible and ample land is available, providing flexibility for the final drainage solution. A final decision on the types of storage to be provided will be made at the detailed drainage design stage.

6 SUMMARY

There are proposals for alterations to the Talbot Hotel with an extension to the rear, and redevelopment of the barn for residential use on an area of land located to the rear of the existing Talbot Hotel, in Chipping.

According to the EA flood map the majority of the proposed development site is located within Flood Zone 1. However, both Flood Zones 2 and 3 encroach partially into the eastern boundary of the site, adjacent to the watercourse.

Chipping Brook flows in a south-easterly direction adjacent to the north-eastern boundary of the site. The Flood Zones have been derived from the EA's NGM and are subject to some uncertainty. A detailed hydraulic model of the Chipping Brook is due to be commissioned by the EA but no modelled data is available to support this study.

The site is considered to be at a low risk of reservoir/canal, groundwater and surface water flooding.

It is recommended that Finished Floor Levels are set to a minimum of 111.60 m AOD. This will provide a 0.62 m of freeboard above the estimated 1 in 100 year flood levels.

The footprint of the proposed hotel extension will encroach in to the existing Flood Zone 3 outline. From the development plans, an estimation of the total volume of flood plain lost as a result of the development was found to be 8.6 m^3 for the 1 in 1,000 year event.

The required compensatory storage could be provided by lowering an area to the north of the proposed extension.

Dry access and egress to the proposed site is expected to be maintained following redevelopment.

It is proposed to limit runoff rates from the proposed impermeable areas to the existing 1 in 2 year flow rate, with storage provided for the 1 in 100 year event including an allowance for climate change. A scheme for the provision and implementation of a surface water regulation system following the principles set out in this FRA should be submitted to and approved in writing by the LPA, prior to the commencement of development.

7 RECOMMENDATIONS

This FRA has demonstrated that the proposed development may be completed without conflicting with the requirements of the NPPF and its supporting Technical Guidance subject to the following:

- Finished floor levels to be set at a minimum of 111.60 m AOD
- Compensatory storage should be provided in accordance with the principles set down in this FRA
- The detailed drainage design, developed in accordance with the principles set down in this FRA, should be submitted to and approved by the local planning authority prior to the commencement of development

APPENDIX A: Topographic Survey


1050N
1000N
1000E
950E
CHIPPING BROOK
Tillotsons Arms (PH)
Talbot Street
Barn
PLAYGROUND (SOFT SURFACE)
GRASS
TARMAC
TREES
Edge of tree canopy
DENSE BUSHES
PRIVATE GARDEN (NO ACCESS)
SHED
GLASS HOUSE
STONE Ht3.5m
N

APPENDIX B: Development Proposals

Dimensions should not be scaled. All dimensions to be checked on site by the contractor before commencement of the relevant part of the work

1:200 — 0m 1m 2m 3m 4m 5m 6m 7m 8m 9m 10m 11m

Rev	Date	Description
E	06.09.12	Position of internal staircase to existing hotel finalised, toilet layouts amended and external fire escape stair indicated.
D	30.08.12	Position of external bin store amended.
C	29.08.12	Position of new stair in existing hotel indicated with amendments to male and female toilets.
B	21.08.12	Minor amendments to scheme to suit existing and proposed
A	20.07.12	Amended scheme further to client meeting.

Talbot Hotel
Chipping

Proposed Site Plan

Drwg. No.: 1723.SK.200 Rev.: E

Date: Jun 12 Scale: 1:200@A1

IWA Architects

Waterloo Mill, Waterloo Road, Clitheroe, Lancashire, BB7 1LR.

t: + 44 (0) 1200 423487
f: + 44 (0) 1200 458278
e: admin@iwarchitects.co.uk
w: www.iwarchitects.co.uk

APPENDIX C: Modified Rational Method Calculation

The Modified Rational Method[10] has been used to calculate the runoff from the impermeable surfaces at the existing site.

The following parameters have been obtained from the maps in Volume 3 of the Wallingford Procedure:

M5-60 minute rainfall depth:	19 mm
Ratio of M5-60 to M5-2 day rainfall:	0.3
Average Annual Rainfall:	1300 mm
Winter Rain Acceptance Potential/ Soil Type :	0.4
The Urban Catchment Wetness Index (UCWI) value:	135

A time of concentration of 3.5 minutes has been used comprising a time of entry of 3 minutes and a time of flow of 0.5 minutes.

A rainfall estimation calculation has been carried out to convert the M5-60 minute rainfall to the 5-minute duration rainfall for the 1 in 2 year, 1 in 30 year and 1 in 100 year return period events. The calculated rainfall intensities for these events are 68.6, 123.9 and 155.5 mm/hr respectively.

The flow rate as given by the Modified Rational Method is:

$$Q=2.78 \times C_v \times C_r \times \text{rainfall intensity} \times \text{impermeable area}$$

where:

C_v is the volumetric runoff coefficient = P_r/PIMP = 0.85
where P_r is Percentage Runoff and PIMP is Percentage Impermeable Area
C_r is the routing coefficient = 1.3
Impermeable Area = 0.222 ha

The flow rates for the impermeable areas at the existing site are shown in the table below:

Flow Rates for Impermeable Areas, Existing Site

Return Period	Flow Rate for 0.222 ha impermeable area (l/s)
1 in 2 year	46.9
1 in 30 year	84.7
1 in 100 year	106.3

[10] The Wallingford Procedure, Volume 4, 1981

APPENDIX D: Micro Drainage Outputs for Greenfield Runoff

ICP SUDS Mean Annual Flood

Input

Return Period (years)	100	SAAR (mm)	1300.000	Urban	0.000
Area (Ha)	0.248	Soil	0.450	Region Number	10

Results	**l/s**
QBAR Rural	2.2
QBAR Urban	2.2
Q 100 years	4.7
Q 1 year	2.0
Q 30 years	3.8
Q 100 years	4.7

APPENDIX E: Micro Drainage Storage Volume Calculation - Tank

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	0.765	0.765	46.7	47.3	O K
30 min Summer	0.893	0.893	46.7	55.2	O K
60 min Summer	0.889	0.889	46.7	54.9	O K
120 min Summer	0.710	0.710	46.7	43.9	O K
180 min Summer	0.532	0.532	46.6	32.9	O K
240 min Summer	0.420	0.420	44.5	25.9	O K
360 min Summer	0.308	0.308	38.1	19.0	O K
480 min Summer	0.255	0.255	32.3	15.8	O K
600 min Summer	0.223	0.223	28.1	13.8	O K
720 min Summer	0.201	0.201	24.8	12.4	O K
960 min Summer	0.171	0.171	20.3	10.6	O K
1440 min Summer	0.137	0.137	15.1	8.4	O K
2160 min Summer	0.110	0.110	11.1	6.8	O K
2880 min Summer	0.095	0.095	9.0	5.9	O K
4320 min Summer	0.077	0.077	6.6	4.8	O K
5760 min Summer	0.067	0.067	5.3	4.1	O K
7200 min Summer	0.060	0.060	4.5	3.7	O K
8640 min Summer	0.055	0.055	3.9	3.4	O K
10080 min Summer	0.051	0.051	3.5	3.2	O K
15 min Winter	0.874	0.874	46.7	54.0	O K
30 min Winter	0.999	0.999	46.8	61.7	O K
60 min Winter	0.939	0.939	46.6	58.0	O K
120 min Winter	0.626	0.626	46.7	38.7	O K

Storm Event	Rain (mm/hr)	Time-Peak (mins)
15 min Summer	109.649	14
30 min Summer	75.673	22
60 min Summer	49.937	40
120 min Summer	31.760	72
180 min Summer	23.941	102
240 min Summer	19.434	130
360 min Summer	14.501	188
480 min Summer	11.758	248
600 min Summer	9.982	308
720 min Summer	8.726	368
960 min Summer	7.050	490
1440 min Summer	5.206	734
2160 min Summer	3.834	1100
2880 min Summer	3.080	1460
4320 min Summer	2.259	2172
5760 min Summer	1.816	2904
7200 min Summer	1.533	3656
8640 min Summer	1.335	4288
10080 min Summer	1.188	4992
15 min Winter	109.649	15
30 min Winter	75.673	24
60 min Winter	49.937	42
120 min Winter	31.760	74

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
180 min Winter	0.413	0.413	44.2	25.5	O K
240 min Winter	0.315	0.315	38.7	19.5	O K
360 min Winter	0.237	0.237	29.9	14.6	O K
480 min Winter	0.198	0.198	24.5	12.2	O K
600 min Winter	0.174	0.174	20.8	10.8	O K
720 min Winter	0.157	0.157	18.2	9.7	O K
960 min Winter	0.135	0.135	14.8	8.3	O K
1440 min Winter	0.109	0.109	10.9	6.7	O K
2160 min Winter	0.089	0.089	8.1	5.5	O K
2880 min Winter	0.077	0.077	6.5	4.7	O K
4320 min Winter	0.063	0.063	4.8	3.9	O K
5760 min Winter	0.054	0.054	3.8	3.4	O K
7200 min Winter	0.049	0.049	3.2	3.0	O K
8640 min Winter	0.045	0.045	2.8	2.8	O K
10080 min Winter	0.042	0.042	2.5	2.6	O K

Storm Event	Rain (mm/hr)	Time-Peak (mins)
180 min Winter	23.941	102
240 min Winter	19.434	130
360 min Winter	14.501	188
480 min Winter	11.758	248
600 min Winter	9.982	308
720 min Winter	8.726	368
960 min Winter	7.050	488
1440 min Winter	5.206	734
2160 min Winter	3.834	1088
2880 min Winter	3.080	1448
4320 min Winter	2.259	2156
5760 min Winter	1.816	2936
7200 min Winter	1.533	3624
8640 min Winter	1.335	4288
10080 min Winter	1.188	4976

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.300	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+30

Time / Area Diagram

Total Area (ha) 0.356

Time (mins)	**Area (ha)**
0-4	0.356

Model Details

Storage is Online Cover Level (m) 1.500

Tank or Pond Structure

Invert Level (m) 0.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	61.8	0.700	61.8	1.400	61.8	2.100	61.8
0.100	61.8	0.800	61.8	1.500	61.8	2.200	61.8
0.200	61.8	0.900	61.8	1.600	61.8	2.300	61.8
0.300	61.8	1.000	61.8	1.700	61.8	2.400	61.8
0.400	61.8	1.100	61.8	1.800	61.8	2.500	61.8
0.500	61.8	1.200	61.8	1.900	61.8		
0.600	61.8	1.300	61.8	2.000	61.8		

Hydro-Brake® Outflow Control

Design Head (m) 1.000 Hydro-Brake® Type Md5 SW Only Invert Level (m) 0.000
Design Flow (l/s) 46.9 Diameter (mm) 268

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	9.7	1.200	49.1	3.000	74.9	7.000	114.4
0.200	24.7	1.400	51.9	3.500	80.9	7.500	118.4
0.300	37.3	1.600	55.0	4.000	86.5	8.000	122.3
0.400	43.7	1.800	58.2	4.500	91.7	8.500	126.0
0.500	46.3	2.000	61.2	5.000	96.7	9.000	129.7
0.600	46.7	2.200	64.1	5.500	101.4	9.500	133.2
0.800	46.0	2.400	67.0	6.000	105.9		
1.000	46.8	2.600	69.7	6.500	110.2		

APPENDIX F: Micro Drainage Storage Volume Calculation – Cellular Storage

Summary of Results for 100 year Return Period (+30%)

Half Drain Time : 11 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	0.080	0.080	0.0	46.6	46.6	36.6	O K
30 min Summer	0.090	0.090	0.0	46.7	46.7	40.9	O K
60 min Summer	0.085	0.085	0.0	46.6	46.6	38.7	O K
120 min Summer	0.056	0.056	0.0	46.4	46.4	25.5	O K
180 min Summer	0.029	0.029	0.0	46.3	46.3	13.1	O K
240 min Summer	0.010	0.010	0.0	46.2	46.2	4.5	O K
360 min Summer	0.000	0.000	0.0	42.2	42.2	0.0	O K
480 min Summer	0.000	0.000	0.0	34.2	34.2	0.0	O K
600 min Summer	0.000	0.000	0.0	29.0	29.0	0.0	O K
720 min Summer	0.000	0.000	0.0	25.4	25.4	0.0	O K
960 min Summer	0.000	0.000	0.0	20.5	20.5	0.0	O K
1440 min Summer	0.000	0.000	0.0	15.1	15.1	0.0	O K
2160 min Summer	0.000	0.000	0.0	11.1	11.1	0.0	O K
2880 min Summer	0.000	0.000	0.0	9.0	9.0	0.0	O K
4320 min Summer	0.000	0.000	0.0	6.6	6.6	0.0	O K
5760 min Summer	0.000	0.000	0.0	5.3	5.3	0.0	O K
7200 min Summer	0.000	0.000	0.0	4.5	4.5	0.0	O K
8640 min Summer	0.000	0.000	0.0	3.9	3.9	0.0	O K
10080 min Summer	0.000	0.000	0.0	3.5	3.5	0.0	O K
15 min Winter	0.096	0.096	0.0	46.7	46.7	43.7	O K
30 min Winter	0.104	0.104	0.0	46.8	46.8	47.5	O K
60 min Winter	0.089	0.089	0.0	46.7	46.7	40.7	O K

Storm Event	Rain (mm/hr)	Time-Peak (mins)
15 min Summer	109.649	14
30 min Summer	75.673	22
60 min Summer	49.937	40
120 min Summer	31.760	72
180 min Summer	23.941	102
240 min Summer	19.434	128
360 min Summer	14.501	0
480 min Summer	11.758	0
600 min Summer	9.982	0
720 min Summer	8.726	0
960 min Summer	7.050	0
1440 min Summer	5.206	0
2160 min Summer	3.834	0
2880 min Summer	3.080	0
4320 min Summer	2.259	0
5760 min Summer	1.816	0
7200 min Summer	1.533	0
8640 min Summer	1.335	0
10080 min Summer	1.188	0
15 min Winter	109.649	14
30 min Winter	75.673	24
60 min Winter	49.937	42

Summary of Results for 100 year Return Period (+30%)

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
120 min Winter	0.041	0.041	0.0	46.4	46.4	18.6	O K
180 min Winter	0.005	0.005	0.0	46.2	46.2	2.4	O K
240 min Winter	0.000	0.000	0.0	40.8	40.8	0.0	O K
360 min Winter	0.000	0.000	0.0	30.5	30.5	0.0	O K
480 min Winter	0.000	0.000	0.0	24.7	24.7	0.0	O K
600 min Winter	0.000	0.000	0.0	21.0	21.0	0.0	O K
720 min Winter	0.000	0.000	0.0	18.3	18.3	0.0	O K
960 min Winter	0.000	0.000	0.0	14.8	14.8	0.0	O K
1440 min Winter	0.000	0.000	0.0	10.9	10.9	0.0	O K
2160 min Winter	0.000	0.000	0.0	8.1	8.1	0.0	O K
2880 min Winter	0.000	0.000	0.0	6.5	6.5	0.0	O K
4320 min Winter	0.000	0.000	0.0	4.7	4.7	0.0	O K
5760 min Winter	0.000	0.000	0.0	3.8	3.8	0.0	O K
7200 min Winter	0.000	0.000	0.0	3.2	3.2	0.0	O K
8640 min Winter	0.000	0.000	0.0	2.8	2.8	0.0	O K
10080 min Winter	0.000	0.000	0.0	2.5	2.5	0.0	O K

Storm Event	Rain (mm/hr)	Time-Peak (mins)
120 min Winter	31.760	74
180 min Winter	23.941	100
240 min Winter	19.434	0
360 min Winter	14.501	0
480 min Winter	11.758	0
600 min Winter	9.982	0
720 min Winter	8.726	0
960 min Winter	7.050	0
1440 min Winter	5.206	0
2160 min Winter	3.834	0
2880 min Winter	3.080	0
4320 min Winter	2.259	0
5760 min Winter	1.816	0
7200 min Winter	1.533	0
8640 min Winter	1.335	0
10080 min Winter	1.188	0

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.300	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+30

Time / Area Diagram

Total Area (ha) 0.356

Time (mins)	**Area (ha)**
0-4	0.356

Model Details

Storage is Online Cover Level (m) 0.720

Cellular Storage Structure

Invert Level (m) 0.000 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	480.0	0.0	0.500	480.0	0.0
0.100	480.0	0.0	0.600	0.0	0.0
0.200	480.0	0.0	0.700	0.0	0.0
0.300	480.0	0.0	0.800	0.0	0.0
0.400	480.0	0.0			

Hydro-Brake® Outflow Control

Design Head (m) 1.000 Hydro-Brake® Type Md5 SW Only Invert Level (m) -0.890
Design Flow (l/s) 46.9 Diameter (mm) 268

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	9.7	1.200	49.1	3.000	74.9	7.000	114.4
0.200	24.7	1.400	51.9	3.500	80.9	7.500	118.4
0.300	37.3	1.600	55.0	4.000	86.5	8.000	122.3
0.400	43.7	1.800	58.2	4.500	91.7	8.500	126.0
0.500	46.3	2.000	61.2	5.000	96.7	9.000	129.7
0.600	46.7	2.200	64.1	5.500	101.4	9.500	133.2
0.800	46.0	2.400	67.0	6.000	105.9		
1.000	46.8	2.600	69.7	6.500	110.2		