Witham Catchment Flood Map Improvements – Phase 2
Lower Witham: Volume 1 - Modelling and Flood Mapping Report

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Witham Catchment Flood Map Improvements

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Introduction

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1 Introduction

PROJECT BACKGROUND

1.1 AECOM (formerly Faber Maunsell) was commissioned by the Environment Agency – Anglian Region to undertake a series of studies to determine flood map improvements and map areas benefiting from defences for the River Witham catchment. Phase 2 of the Witham Flood Map Improvements (FMI) project is concerned with the Lower Witham catchment (watercourses downstream of Lincoln), including the River Bain, Barlings Eau/Stainfield Beck, River Slea and South Forty Foot sub-catchments. This report documents the methodology for determination of flood map improvements in the Lower Witham catchment, excluding the River Bain[1] and South Forty-Foot sub-catchments which were modelled and mapped separately.

1.2 An existing InfoWorks RS (IWRS) hydraulic model of the Lower Witham was constructed previously by Faber Maunsell under the Witham Catchment Strategic Model (WiCSM) commission. This model build included all Environment Agency main river channels in the Lower Witham catchment, including the Barlings Eau, Stainfield Beck and River Slea. The model was calibrated to the November 2000 and February 2001 events.

1.3 This report explains the methodology and approach undertaken in order to develop the Lower Witham WiCSM parent model for the purposes of producing improved flood outlines. The development of the model involved three key elements:

- Development of design hydrology for a suite of return periods up to the 0.1% AEP event;
- Development of the hydraulic model to contain all flow up to the 0.1% AEP event plus climate change;
- Development of the flood mapping model within IWRS.

1.4 The development of the design flood hydrology used in the hydraulic modelling was undertaken at the start of the project, and an interim hydrology report was issued to the Environment Agency for approval. This report (Volume 1) provides a summary of the design flood hydrology, whilst the full interim hydrology report has been finalised to form Volume 2.

1.5 As well as flood map improvements, the scope of work for the Lower Witham study included a summer sensitivity analysis, freeboard analysis, standard of protection assessment and a preliminary hydraulic analysis for a pre-feasibility study in the Barlings Eau catchment. The sensitivity analyses, standard of protection assessment and freeboard analysis are presented in this document. The preliminary analysis for the pre-feasibility study was originally presented to the Environment Agency as an interim technical note in April 2009. A copy of this technical note is included in Appendix A of this report.

1.6 It should be noted that the flood mapping produced by this study downstream of Coningsby supersedes the mapping produced for this area by the River Bain Flood Map Improvements study. This is because the latter did not fully account for the interaction between the Lower Witham floodplains and the River Bain, and the fact that more accurate 2D modelling of flow paths was undertaken in this location for the Lower Witham study.

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[1] Witham Catchment Flood Map Improvements, Faber Maunsell, River Bain: Volume 1 – Modelling and Flood Mapping Report, Environment Agency, March 2009

LOWER WITHAM CATCHMENT DESCRIPTION

1.7 The Lower Witham sub-catchment is located in the central part of the wider Witham catchment as shown in Figure 1.1.

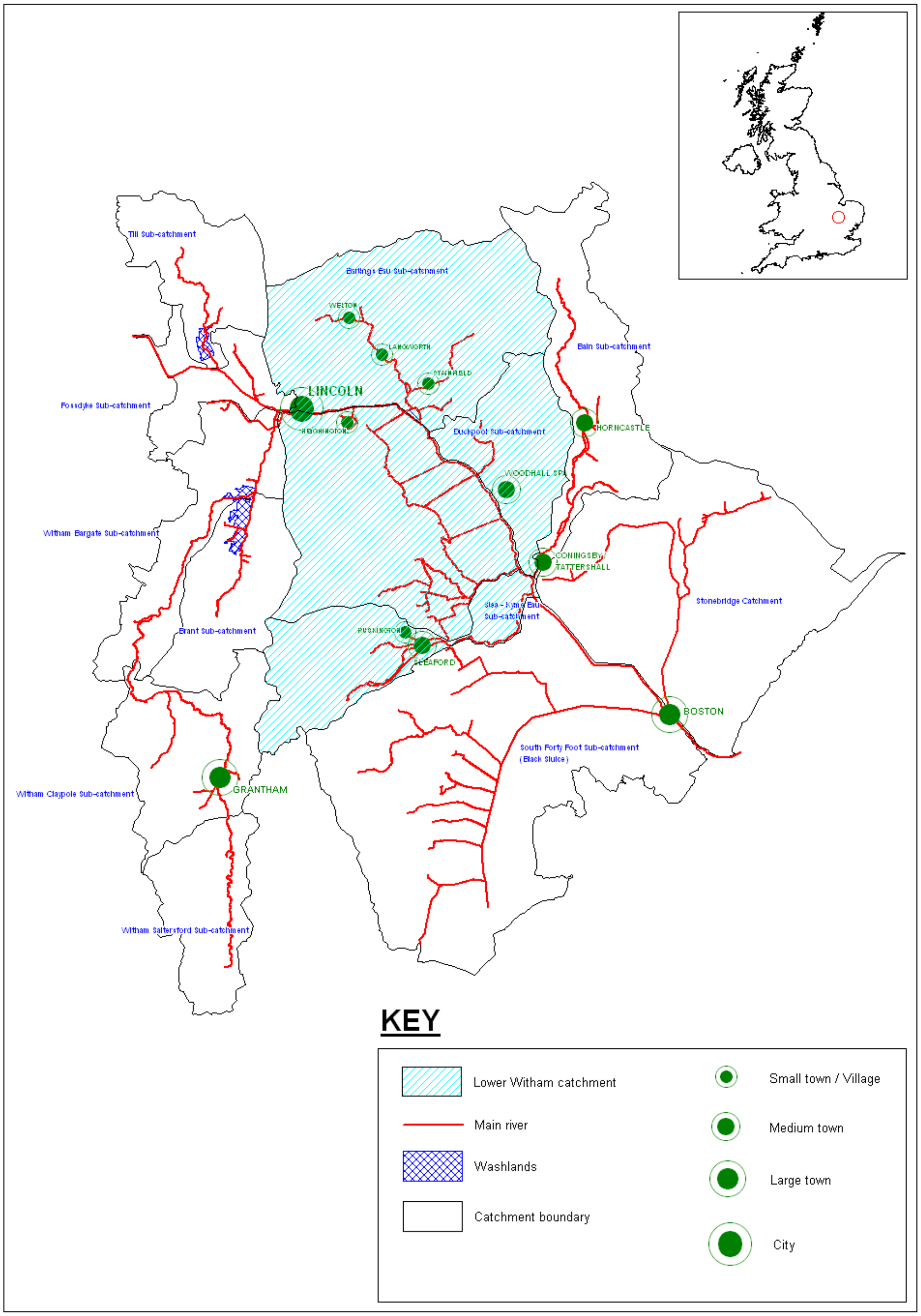

Figure 1.1: Catchment Location Plan

1.8 The primary watercourses of the Lower Witham catchment are:

- River Witham/Old River Witham
- South Delph
- River Bain/Horncastle Canal
- Barlings Eau and Welton Beck
- Stainfield Beck
- Duckpool Drain
- Snakeholme Drain
- Heighington Beck
- Heighington Catchwater / Washingborough Roadside North &South
- River Slea / Kyme Eau
- Billinghay Skirth
- Car Dyke and Delphs drainage system
- Dorrington and Farroway drainage system
- Leasingham Beck
- Ruskington Beck

1.9 All of the above watercourses are classified as Environment Agency main river and are therefore included in the hydraulic model, together with a number of other minor tributaries which are also classified as main river. In addition, the model was extended at the Environment Agency's request, to include the following non-main river reach:

- Ruskington Beck (Upper Reaches, chainage 1526m to 4919m)

The network of land drains within the Lower Witham catchment are not explicitly modelled but contribute flow to Internal Drainage Board (IDB) pumping stations, which are represented in the model.

1.10 The principal watercourse in the Lower Witham catchment is the River Witham, which flows between the city of Lincoln at the upstream limit and the major town of Boston at the downstream limit. The other main towns in the Lower Witham catchment are Sleaford, Woodhall Spa and Tattershall with smaller communities scattered throughout the study area.

1.11 The Lower Witham catchment is a complex river system in which the Lower River Witham acts as a central spine into which a series of natural and engineered channels contribute flow. The system is complicated further by a network of pumped IDB drains, draining the surrounding lowland fens via a network of 26 pumping stations. Operated by the Witham First IDB and Witham Third IDB, these pumping stations also contribute significant flow in the system. The lowland fens are mainly rural with a significant number of villages and farms. The main land use is arable farming.

1.12 The Lower Witham catchment as defined by the extents of the Lower Witham sub-model has a total area of 995 km^2. This includes the subcatchment areas associated with the modelled inflows from the River Slea and Barlings Eau, but excludes the catchment areas of the Upper Witham and River Bain. There are in excess of 260 km of modelled watercourses within the Lower Witham sub model.

1.13 The Lower Witham model commences immediately downstream of Lincoln, with the River Witham and South Delph water courses running east in parallel towards Branston Island. Upstream of Branston Island, the River Witham splits, with the course of the Old River Witham looping to the east and the engineered River Witham continuing south. The Barlings Eau sub-catchment drains into the Old River Witham to the north of Branston Island, with the combined flow continuing round the Old Witham loop to rejoin the engineered River Witham and South Delph downstream of Bardney Lock. The River Witham then flows south-east, collecting major inflows from the Rivers Bain and Slea (Kyme Eau), eventually discharging into the tidal River Haven via a gravity sluice (Grand Sluice) in Boston.

1.14 The land to the west of the River Witham is drained into the main channel by via number of 'Delphs' entering the Witham at right angles. The Delphs are engineered drainage channels conveying flow from Car Dyke, which runs parallel to the River Witham and intercepts runoff from the 'Heath' catchment further to the west. Towards the southern end of the Heath, Billinghay Skirth also takes flow from the downstream end of Car Dyke to supplement its primary flow from the Farroway Drain/Dorrington Dyke pumped drainage system.

1.15 A number of additional tributaries also provide significant inputs into the Lower Witham system. Stainfield Beck drains into the Barlings Eau catchment approximately 2km upstream of the confluence with the Old River Witham. Snakeholme Drain also flows into the Old River Witham, approximately 0.5km downstream of the confluence with Barlings Eau. Eight kilometres downstream of Branston Island, Duckpool Drain enters the River Witham from the east.

1.16 The Lower Witham catchment lies to the east of a ridge of limestone running north-south through Lincoln. Immediately to the east of this ridge, the solid geology is Oxford Clay and Kellaway Sands, which underlies the Barlings Eau, River Slea and 'Heath' subcatchments. The remaining subcatchments that lie further east still are underlain by Ampthill Clay, Kimmeridge Clay and Corallian (Clay and Limestone).

1.17 The mudstone and clay areas of the catchment are not very permeable, and as a result cannot store large quantities of water. Consequently these areas will generate runoff and lead to high river flows which can pose a flood risk. The limestone and chalk areas of the catchment are more permeable than the mudstone and clay, and do absorb and store rainwater thereby reducing the flood risk. Most groundwater is therefore located in the limestone and chalk areas.

1.18 The extent of water absorbed and stored by the solid geology is also influenced by the soil moisture conditions at the time. When storage is limited by high soil moisture content, i.e. following long-term wet weather (as in 2000), the runoff rate is higher. This can result in the water stored within the geology emerging as base flow in streams and rivers, or as springs. High groundwater levels have been experienced in limestone areas of the Lower Witham catchment, specifically between Heighington and Sleaford, and can pose an increased flood risk.

1.19 Drift geology in the Barlings Eau subcatchment consists predominantly of till while the fen areas in the south-east are comprised of alluvium and peat. Most of the watercourses flow across areas of alluvium or river terrace deposits. Groundwater flow through the peat is slow; most of the groundwater flow will be through the till and alluvium, although how quickly it flows is dependent on the amount of fine material.

1.20 In the west where there is little drift geology and there is less material to store rainfall. Therefore runoff into rivers is encouraged, causing high river flows. In contrast peat is capable of storing large volumes of rainfall and would slow the speed with which water gets into the river channels.

1.21 The average annual rainfall varies across the Lower Witham catchment between 578 and 631mm of rainfall per year. The highest average rainfall of 631mm per year occurs over the upland areas of the Stainfield Beck catchment. Rainfall totals decrease to 578mm per year in the 'Heath' parts of the Lower Witham catchment, close to Digby and Scopwick. These average annual rainfall figures are particularly low for the United Kingdom. The average annual rainfall for the UK is typically 900 – 1000mm per year, consequently the Lower Witham is classified as a 'dry catchment'.

SCOPE OF STUDY & KEY OUTPUTS

1.22 The scope of the study covers the following items, all of which are documented in this report unless otherwise stated:

- Development of design hydrology for the following return periods: 50%, 20%, 10%, 5%, 4%, 2%, 1.33%, 1%, 0.5% and 0.1% Annual Exceedance Probability (AEP) events, as well as the

1% and 0.1% AEP events representing climate change (for 2115) based on current DEFRA guidance;

- Development of the InfoWorks RS hydraulic model (with defences) to contain all flow up to the 0.1% AEP event plus climate change;
- Development of the flood mapping model within IWRS;
- Model simulations and flood mapping for the 'with defences' case for the full suite of design hydrology;
- Model simulations and flood mapping for the 'without defences' case for 1% and 0.1% AEP events, with and without climate change (To be reported separately, see 1.25 below);
- Determination of Areas Benefiting from Defences (ABDs) for the 1% AEP event by comparison the 'with defences' and 'without defences' flood mapping (To be reported separately, see 1.25 below);
- Model simulations of sensitivity scenarios as advised by the Environment Agency;
- Analysis of the actual and indicative Standard of Protection (SoP) and Annual Exceedance Probability (AEP) for all modelled watercourses;
- Analysis of freeboard requirements for all modelled watercourses;
- Preliminary analysis in preparation of a pre-feasibility study in the Barlings Eau catchment.

1.23 The following items are the key deliverables for the Lower Witham flood map improvements study:

- Defended flood extents in ArcGIS .shp format for the 50%, 20%, 10%, 5%, 4%, 2%, 1.33%, 1%, 0.5%, 0.1%, 1%+climate change and 0.1%+climate change AEP events;
- Undefended flood extents in ArcGIS .shp format for the 1%, 0.1%, 1%+climate change and 0.1%+climate change AEP events;
- Areas Benefiting from Defences ArcGIS .shp format polygon for the 1% AEP event;
- Tabulated results (peak levels and flows) for the 'with defences' scenario all return periods, plus long sections for all lengths of modelled channel;
- An ArcGIS .shp format file identifying those defences with a shortfall in SoP within the Lower Witham catchment, plus tabulated results of the freeboard analysis;
- InfoWorks RS models and results files for the Lower Witham Catchment as used in the derivation of the above outputs;
- A Technical Report.

1.24 The following are provided as additional to the key deliverables detailed in the brief:

- Tabulated results identifying the Annual Exceedance Probability of overtopping at each cross-section within the model.

Undefended Modelling and Mapping

1.25 At an early stage in the project AECOM identified the possibility of using a TUFLOW 2D model to generate the undefended mapping for the Lower Witham, South Forty Foot, and Stonebridge catchments. This approach sought to overcome the potential problems of 1D modelling of the undefended scenario such as model stability and interaction of flows between the three sub-catchments. Following discussions with the Environment Agency, it was agreed that this TUFLOW 2D approach would be appropriate and that the undefended modelling, flood mapping and derivation of areas benefitting from defences would be presented separately in a standalone report covering all three sub-catchments.

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Hydrology

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2 Hydrology

INTRODUCTION

2.1 The first step in producing design flood mapping is to derive estimates of the magnitude of extreme flood events, in order to generate flows to drive the hydraulic model.

2.2 The hydrological assessment was carried out prior to the commencement of the hydraulic modelling and flood mapping aspects of this project, and delivered to the Environment Agency in the form of an Interim Hydrology Report [2]. A revised version of this report, incorporating changes as a result of comments provided by the Environment Agency, is provided as Volume 2 of the Final Report[3].

2.3 A brief summary of the hydrological assessment is provided below. Any changes to the methods used, along with any finalisations that were not covered in the Interim Hydrology Report, are additionally detailed.

HYDROLOGICAL MODELLING

2.4 The Lower Witham catchment has previously been divided into subcatchments as part of the Witham Catchment Strategic Model (WiCSM) commission. The majority of these subcatchments were unaltered following review. However, there were some exceptions:

- The boundaries of the Queen Dyke (LW_QD) and Metheringham Delph (LW_MD) subcatchments were adjusted following review against the FEH CD-ROM v2. This resulted in a larger, elongated subcatchment associated with LW_MD. Catchment descriptors were updated accordingly.
- No subcatchments previously existed for Heighington Catchwater. The FEH CD-ROM defined the area that bounded such that it lies between the Sandhill Beck pumped subcatchment to the north (LW_Sandhill Beck) and the Heighington Beck catchment to the south (LW_HI).
- A review of Anglian Water sewer plans and information from the Environment Agency indicated that only the eastern part of the previously defined Heighington catchwater catchment actually drains to Heighington Catchwater, whilst the western part drains under Washingborough Road into the Sandhill Beck IDB catchment. Therefore the Heighington Catchwater subcatchment was split into two parts – LW_HCW_1 to the east and LW_HCW_2 to the west.

2.5 Initially, hydrometric data was collated for the Lower Witham catchment which was used to develop the hydrological models. This included gauging station details (including theoretical rating equations), level and flow data, rainfall data and soil moisture deficit (MORECS) data. A complete historical review was also undertaken to provide guidance during the flood estimation process.

2.6 A complete review of the available flow gauging stations was carried out in order to determine their potential use in statistical flood frequency analysis. Six gauging stations were deemed suitable enough to estimate *QMED* from their records, and to also use in developing growth curves.

2.7 The six level and flow gauging stations providing information on river flows are listed in Table 2.1 with a summary of the type of gauge.

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[2] Faber Maunsell (2008) Witham Flood Map Improvements – Phase 2, Lower Witham - Hydrology Report.

[3] AECOM (2009) Witham Catchment Flood Map Improvements – Phase 2, Lower Witham: Volume 2 - Hydrology Report

2.8 A number of points were established throughout the catchment where peak flow estimates were required; the flows within the hydraulic model were scaled to achieve the peaks at these flow estimation points (FEP). The sites were selected with reference to available flood data at gauges, with additional sites chosen in order to correctly map flood extents on the ungauged tributaries. The chosen sites give a good spatial distribution of flow estimation points, which increases confidence in the floodplain maps produced.

2.9 The locations where peak flows were derived in the Lower Witham catchment are listed in Table 2.2 and illustrated in Figure 2.1.

Gauge	River	Comments
Langworth Ultrasonic	Barlings Eau	- Systematic underestimation of flow by ultrasonic gauge - Developed correction in the form of an alternative regression equation
Creampoke Farm	Stainfield Beck	- Model required to extend rating curve - Seasonal factors accounted for
Heighington	Heighington Beck	- Model required to extend rating curve
Dunston	Dunston Beck	- Model not available to check rating curve - Accepted current EA rating based on performance against check gaugings
Scopwick	Scopwick Beck	- Model not available to check rating curve - Accepted current EA rating based on performance against check gaugings
Wilsford	Slea	- Model not available to check rating curve - Accepted current EA rating based on performance against check gaugings

Table 2.1: Gauging stations on main rivers in the Lower Witham catchment

Site Code	Flow Estimation Point	NGR
Site 1A	River Witham	NGR 498300 371200
Site 1B	South Delph	NGR 498300 371200
Site 2	Barlings Eau at Langworth	NGR 506600 376600
Site 3	Welton Beck	NGR 505150 379400
Site 4	Apley Beck	NGR 509500 373550
Site 5	Sambre Beck	NGR 509600 373450
Site 6	Stainfield Beck at Creampoke Farm	NGR 512700 373900
Site 7	Snakeholme Drain	NGR 510650 371300
Site 8	Duckpool Drain	NGR 516300 368150
Site 9	Reeds Beck	NGR 515900 365250
Site 10	Woodhall Sewer	NGR 517900 361650
Site 11	Kirkstead Mill Beck	NGR 518800 360250
Site 12	Park Beck	NGR 518900 359550
Site 13	Heighington Catchwater	NGR 504150 370500
Site 14	Heighington Beck at Heighington	NGR 504042 369515
Site 15	Branston Delph	NGR 506050 369150
Site 16	Car Dyke	NGR 508200 364600
Site 17	Dunston Beck	NGR 505890 362250
Site 18	Metheringham Delph	NGR 509400 362300
Site 19	Queens Dyke	NGR 510500 357500
Site 20	Car Dyke	NGR 512141 358032
Site 21	Scopwick Beck	NGR 507160 358040
Site 22	New Cut	NGR 514600 353950
Site 23	Digby Beck	NGR 510000 355400
Site 24	North Hills Drain	NGR 511150 354250
Site 25	Ruskington Beck	NGR 510200 349850
Site 26	Leasingham Drain	NGR 510400 349650
Site 28	Slea at Slea Upper	NGR 500850 343000
Site 29	Slea at Cranwell Line Sluice	NGR 505500 345850
Site 30	River Bain	NGR 522630 358510
Site 31	Slea at Slea Lower	NGR 510610 347930

Table 2.2: Flow Estimation Points in the Lower Witham catchment

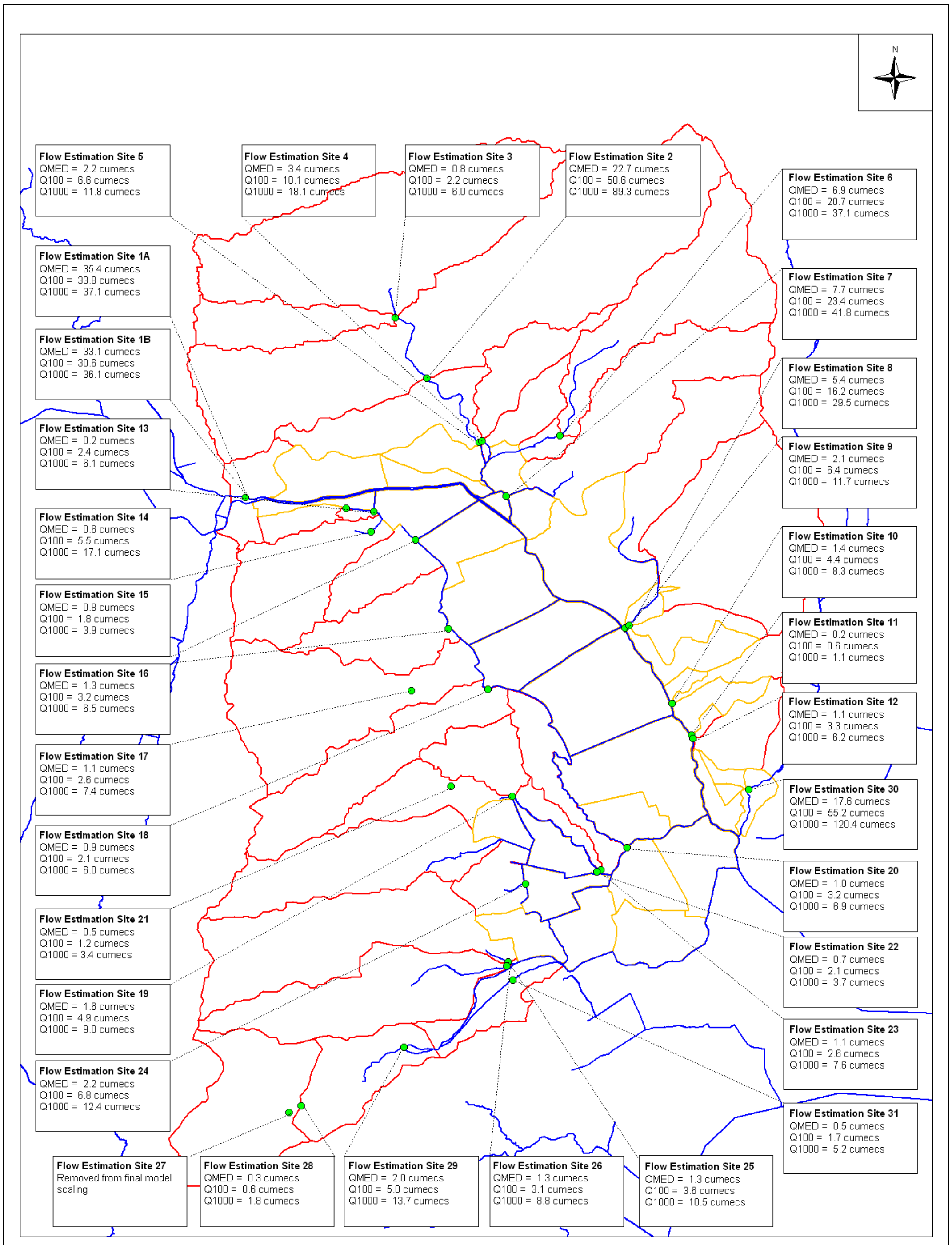

Figure 2.1: Flow Estimation Points in the Lower Witham Catchment

Estimation of peak flows

2.10 Sites 1A, 1B and 30 were termed flow estimation points in that they required input of the correct design flows. However, as they are major tributary inflows derived from other Witham Flood Map Improvement project models, the flows from them were determined prior to the development of the Lower Witham design event hydrological models. The sites were added to the Lower Witham FMI model as Flow-Time boundaries, based on the flow outputs from the relevant upstream models.

2.11 It was previously stated that the Heighington Catchwater subcatchment had been subdivided further into eastern (LW_HCW_1) and western (LW_HCW_2) parts. However, no flow estimation point or subsequent boundary unit was created for LW_HCW_2. This was because the runoff generated from this subcatchment was accounted for in the scaling factor derived for the pumped subcatchment of Sandhill Beck (LW_Sandhill Beck) since the two subcatchments are hydraulically connected. The only boundary unit created for Heighington Catchwater was termed site 13, and exclusively represents the runoff generated from LW_HCW_1.

2.12 The Lower Witham Hydrology Report[2] stated that Site 27 – Slea at Wilsford would be a flow estimation point. Subsequent review of the hydraulic model has revealed that this site is not on the modelled reach, and it has therefore been removed.

2.13 Site 31 – Slea at Slea Lower is a flow estimation point that was added during the development of the hydrological model in conjunction with the hydraulic model development. The Slea at Slea Lower is a relatively impermeable sub-catchment of the Slea. Being relatively impermeable, this subcatchment peaks earlier than the rest of the Slea catchment, and it was decided that it would be appropriate to derive a peak flow estimate for it. Further explanation is provided in Section 2.30 below.

2.14 A subcatchment within the Lower Witham system (model subcatchment LW_HC contributing flow to 'Gibsons Cut') that was delineated during the development of the Lower Witham Strategic Model has not been assigned a flow estimation point. This is because the subcatchment is too small for application of the FEH statistical method. An unscaled FEH/FSR rainfall-runoff model was applied to this subcatchment – this was considered acceptable as the volume generated from this would be precautionary but would still be a negligible addition to the overall volume in the Lower Witham.

2.15 An estimate of *QMED* at each site was made using standard FEH methods. For all sites, a pooled growth curve was generated either by creating a pooling group at that site, or by transferring a pooled growth curve from a suitable gauged analogue site within the Lower Witham catchment. Consequently, a pooled flood frequency curve was generated at each FEP. At the six gauged sites on the Lower Witham, single site flood frequency curves were also generated as alternatives. The final choice of peak flood estimates was based on knowledge about flow and recurrence as determined from the historical record. For the majority of sites, the pooled flood frequency curve estimates were used. The only exception was for Heighington at Heighington Beck, where the single site growth curve was found to be more appropriate. This was due to a combination of the urbanisation and permeability of the site. The final peak flow estimates are provided in Table 2.3.

2.16 Estimation of peak flows for the 0.1% AEP event used a hybrid approach combining statistical and rainfall-runoff methods, and the final peak flow estimates are also provided in Table 2.3. Full details are given in the Lower Witham: Volume 2 – Hydrology Report[3].

FES	Annual Exceedance Probability (%)											
	50%	20%	10%	5%	4%	2%	1.33%	1%	1% CC	0.5%	0.1%	0.1% CC
Site 1A	35.4	29.5	29.3	n/a	30.6	32.6	33.0	33.8	35.2	34.8	37.1	37.6
Site 1B	33.1	26.9	27.2	n/a	28.2	29.9	30.6	30.6	33.6	32.2	36.1	37.1
Site 2	22.7	30.8	35.7	40.3	41.7	46.2	48.8	50.6	60.7	55.1	89.3	107.2
Site 3	0.8	1.2	1.4	1.6	1.7	1.9	2.0	2.2	2.6	2.4	6.0	7.2
Site 4	3.4	5.0	6.1	7.3	7.6	8.8	9.6	10.1	12.1	11.5	18.1	21.7
Site 5	2.2	3.3	4.0	4.7	4.9	5.7	6.2	6.6	7.9	7.4	11.8	14.1
Site 6	6.9	10.3	12.6	14.9	15.6	18.1	19.6	20.7	24.8	23.4	37.1	44.6
Site 7	7.7	11.6	14.2	16.8	17.6	20.4	22.1	23.4	28.0	26.6	41.8	50.1
Site 8	5.4	8.1	9.8	11.6	12.2	14.2	15.3	16.2	19.5	18.4	29.5	35.4
Site 9	2.1	3.2	3.9	4.6	4.8	5.6	6.0	6.4	7.7	7.3	11.7	14.1
Site 10	1.4	2.2	2.7	3.2	3.3	3.8	4.2	4.4	5.3	5.0	8.3	9.9
Site 11	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.7	0.6	1.1	1.3
Site 12	1.1	1.7	2.0	2.4	2.5	2.9	3.2	3.3	4.0	3.8	6.2	7.5
Site 13	0.2	0.6	0.9	1.2	1.3	1.8	2.1	2.4	2.9	3.3	6.1	7.3
Site 14	0.6	1.5	2.0	2.7	3.0	4.0	4.8	5.5	6.6	7.6	17.1	20.5
Site 15	0.8	1.0	1.2	1.4	1.4	1.6	1.8	1.8	2.2	2.1	3.9	4.6
Site 16	1.3	1.8	2.1	2.4	2.5	2.8	3.0	3.2	3.8	3.6	6.5	7.8
Site 17	1.1	1.4	1.7	1.9	2.0	2.3	2.5	2.6	3.1	2.9	7.4	8.9
Site 18	0.9	1.1	1.3	1.5	1.6	1.8	2.0	2.1	2.5	2.3	6.0	7.2
Site 19	1.6	2.4	3.0	3.5	3.7	4.3	4.7	4.9	5.9	5.6	9.0	10.7
Site 20	1.0	1.6	1.9	2.3	2.4	2.8	3.0	3.2	3.8	3.6	6.9	8.3
Site 21	0.5	0.6	0.8	0.9	0.9	1.0	1.1	1.2	1.4	1.3	3.4	4.1
Site 22	0.7	1.0	1.2	1.5	1.6	1.8	1.9	2.1	2.5	2.3	3.7	4.5
Site 23	1.1	1.5	1.7	2.0	2.0	2.3	2.5	2.6	3.2	3.0	7.6	9.2
Site 24	2.2	3.4	4.1	4.9	5.1	5.9	6.4	6.8	8.1	7.7	12.4	14.9
Site 25	1.3	2.0	2.3	2.7	2.8	3.2	3.4	3.6	4.3	4.0	10.5	12.6
Site 26	1.3	1.7	2.0	2.3	2.4	2.8	3.0	3.1	3.8	3.5	8.8	10.6
Site 28	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	1.8	2.1
Site 29	2.0	2.8	3.2	3.6	3.8	4.5	4.7	5.0	6.0	5.5	13.7	16.5
Site 30*	17.6	24.1	29.6	35.9	38.5	41.8	50.6	55.2	66.2	66.4	120.4	141.6
Site 31	0.5	0.8	1.0	1.2	1.3	1.5	1.6	1.7	2.0	1.9	5.2	6.2

Table 2.3: Design flow (m^3s^{-1}) estimates at the Flood Estimation Points

*Note that the peak flow from the River Bain is totalled from the channel section and corresponding floodplain sections

Generation of hydrographs – direct runoff dominated gravity subcatchments

2.17 The generation of design event hydrographs for direct runoff dominated gravity subcatchments was made using the FSR/FEH rainfall-runoff method. Full details regarding the choice of this method are given in the Final Report - Volume 2. Analysis of hydrometric data from gauged direct runoff dominated gravity subcatchments in the Lower Witham catchment provided design values of the time-to-peak parameter and the Catchment Wetness Index parameter. These were incorporated into the FSR/FEH rainfall-runoff model prior to scaling the hydrographs to the statistical flows. Full details regarding the estimation of these parameters are given in the Final Report - Volume 2.

Generation of hydrographs – pumped subcatchments

2.18 The generation of design event hydrographs for pumped subcatchments was made using the FSR/FEH rainfall-runoff method. These boundaries were scaled by a factor derived by analysis of pumped volumes during observed events. Full details of this method are given in the Final Report - Volume 2. The unit hydrograph and time-to-peak values for the pumped subcatchments were set based on the method described by Samuels (1993)[4]

[4] Samuels, P.G., (1993). The Hydraulics and Hydrology of Pumepd Drainage Systems, An Engineering Guide, HR Wallingford Report SR 331, MAFF

Generation of hydrographs – groundwater dominated gravity subcatchments

2.19 The generation of design event hydrographs for groundwater dominated gravity subcatchments was made using an alternative method which derives a typical hydrograph shape from observed events, as set out by Archer et al. (2000)[5]. Full details regarding the choice of this method, and an outline of its basic application, are given in the Final Report - Volume 2 (Chapter 6). However, during the development of the hydrological model in combination with the hydraulic model, some limitations in the method were uncovered and a number of adaptations were made. These are discussed in the following paragraphs.

2.20 As stated in the Final Report - Volume 2, the typical hydrograph shape is derived by combining observed hydrographs from events recorded by the gauging stations of the groundwater dominated subcatchments. The output is dimensionless as it was necessary to remove the effect of magnitude from the source observed events. The output at Dunston gauge is given in Figure 2.2.

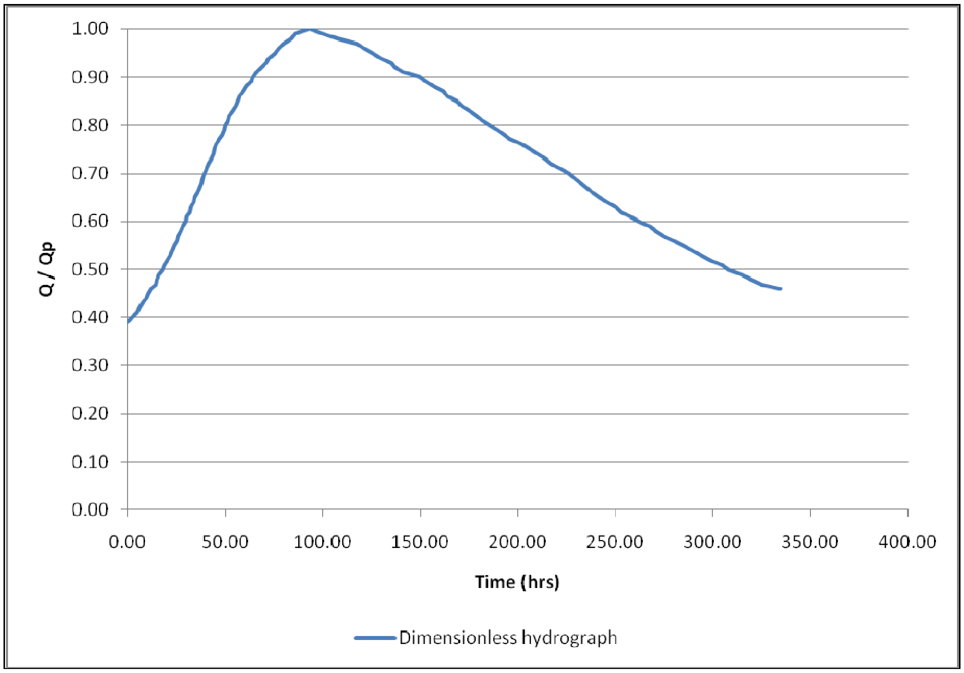

Figure 2.2: Dimensionless hydrograph following hydrograph shape analysis at Dunston Beck gauging station

2.21 This output raised a number of issues:

- If the observed events are relatively small, then the ratio of the starting and end flow / baseflow to the peak will be relatively large;
- Additionally, due to the need to centralise the peaks of the source observed hydrographs, and the varying time bases of the events themselves, the rising and falling limbs of the dimensionless typical hydrographs are truncated;
- The method states that to generate a design hydrograph, the ordinate/ratio at each time interval is multiplied by the required peak flow. Therefore, for rarer events, the starting and end flow / baseflow is significantly larger than for less rare events;
- If the starting and end flow / baseflow of the design hydrograph from the permeable catchments is greater than the flow at which spilling into storage areas occurs, then these storage areas will fill earlier than expected, and continue to fill at the end of the event, potentially causing unrealistic flood extents.

A series of adaptations were applied to overcome the issues stated above.

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[5] Archer, D., Foster, M., Faulkner, D. and Mawdsley, J. (2000) The synthesis of design flood hydrographs. In: Proc. ICE/CIWEM Conf. Flooding – Risks and Reactions. Terrace Dalton, London

2.22 The first issue to address was the truncation of the rising limb. The truncation removes the lead in period where hydrograph is rising from the baseflow condition. To overcome this, the average time of the peak flow from the observed hydrographs was calculated at each gauge. The typical dimensionless hydrograph was then shifted along the time axis, causing the initial ordinate to become applicable at a time beyond that of 0 hours. This is shown in Figure 2.3.

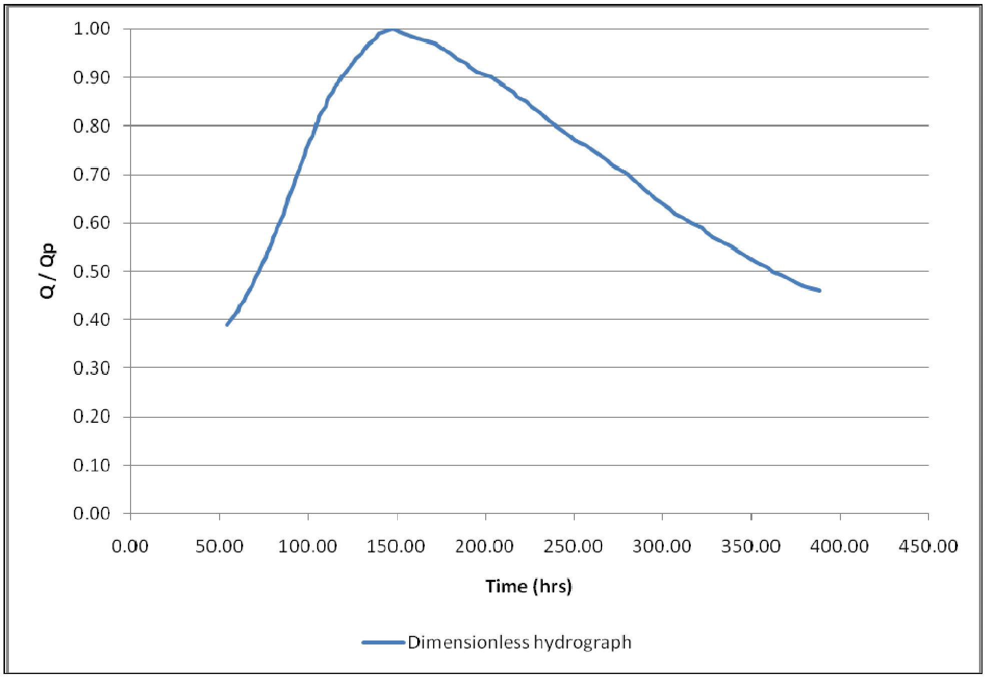

Figure 2.3: Dimensionless hydrograph at Dunston Beck gauging station following lateral shift

2.23 The second issue then becomes the lack of a complete lead-in section to the typical dimensionless hydrograph. In the hydraulic model, each subcatchment has a separate flow component required to ensure the model will start. Therefore all event inflows should commence at zero flow. In reality, the separate flow component is not necessarily equal to the baseflow, so consideration was given to what the more typical baseflow value would be in the groundwater dominated subcatchments. This was done by calculating the average ratio of initial to peak flow from the observed events at each gauge. The typical hydrograph was then ramped up to this ratio (ordinate) after the first 10 hours, as seen in Figure 2.4.

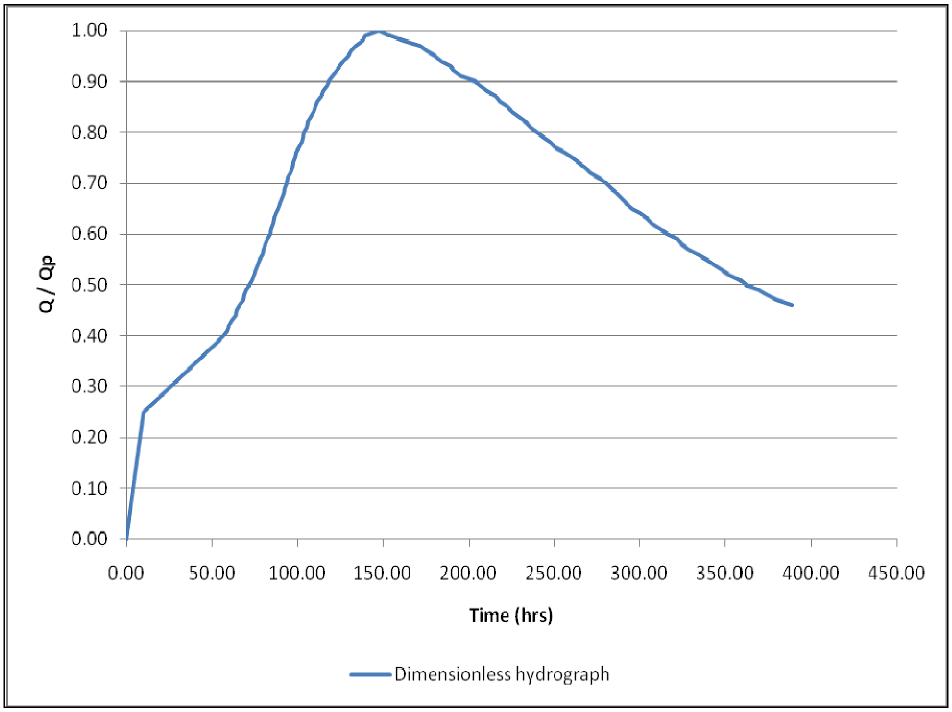

Figure 2.4: Dimensionless hydrograph at Dunston Beck gauging station with lead in sections added based on proposed method

2.24 The final issue is that the falling limb needs to be extended to an ordinate that will not cause spilling at the highest return period event at the end of the event. To do this, the falling limb was extrapolated based on the gradient over the final few hours of the existing falling limb, to an ordinate at which the channel would not be spilling during the 0.1% AEP plus climate change event. Figure 2.5 shows the extrapolated falling limb.

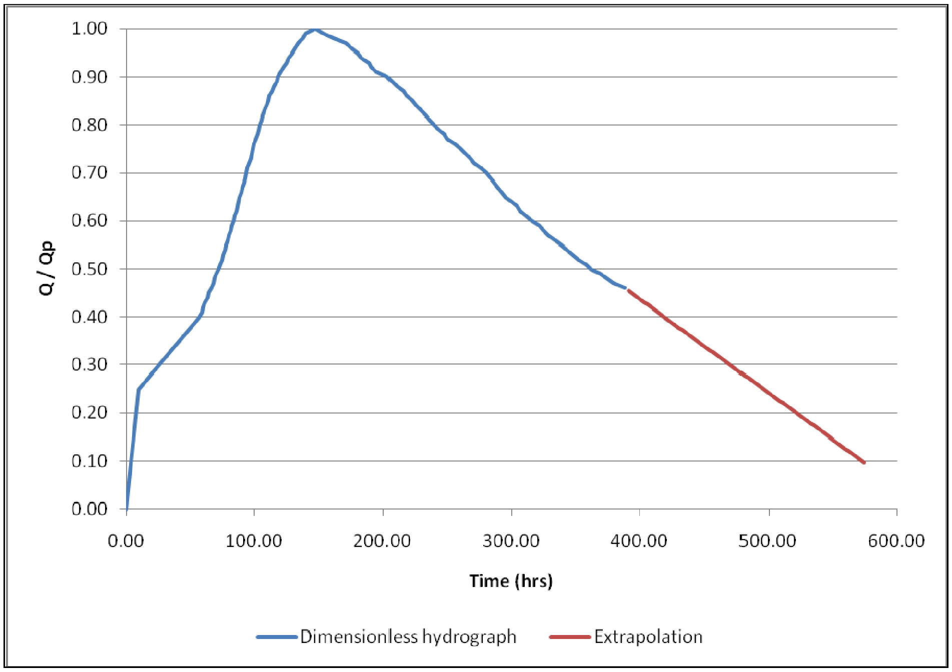

Figure 2.5: Dimensionless hydrograph at Dunston Beck gauging station showing extrapolation of falling limb

2.25 These adaptations provide an appropriate inflow hydrograph from the groundwater dominated subcatchments of Dunston Beck and Heighington Beck (and the subcatchments to which they provide analogue hydrograph shapes to). They have retained the typical shape derived from the application of the basic method, whilst using information from the observed data to extend the rising and falling limbs.

2.26 For the groundwater dominated subcatchments of Scopwick Beck and the Slea at Wilsford, further adaptation was needed because the falling limb of the typical hydrographs did not recede at a rate suitable to apply the extrapolation step. This is apparent in Figure 2.6, where the extrapolation has been applied to 600 hours and yet is only at an ordinate of 0.8.

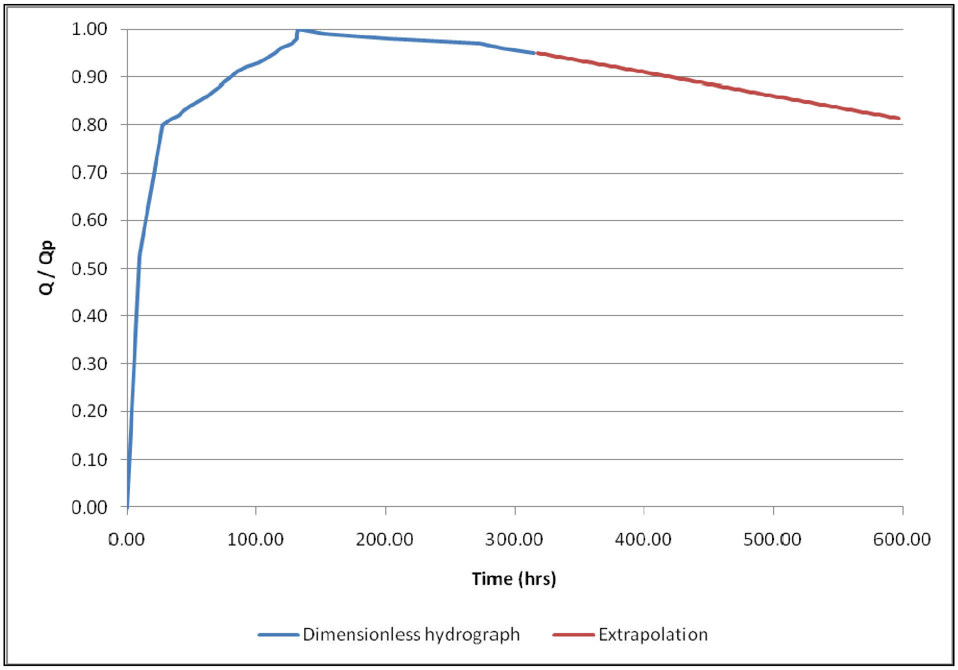

Figure 2.6: Dimensionless hydrograph at Scopwick Beck gauging station showing how extrapolation of falling limb does not cause reduction to a suitably low ordinate within an appropriate time period

2.27 In order to overcome this issue, the time taken to return to the starting ordinate was calculated for each observed event, based on the standard extrapolation process described previously. An average value was obtained, and it was assumed that the hydrograph would return to zero over this time period. An amalgamation of the extrapolated falling limb and the extension to zero at the calculated time base was made, resulting in an appropriate hydrograph to apply in the design event scenarios., as shown in Figure 2.7

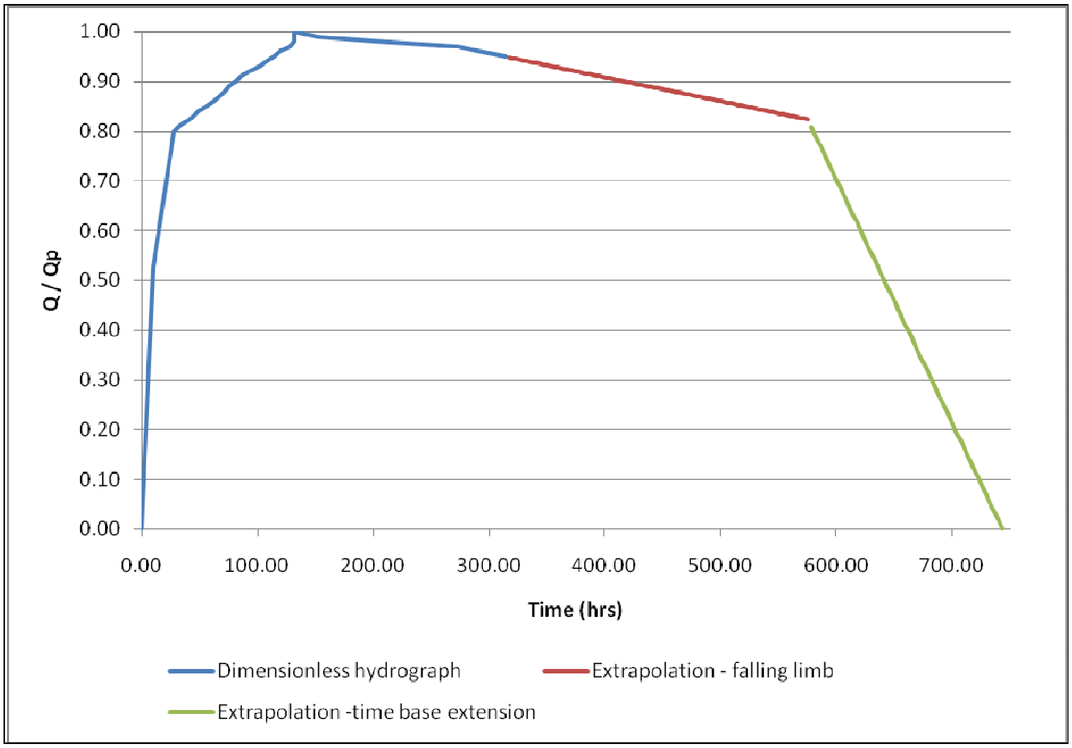

Figure 2.7: Dimensionless hydrograph at Scopwick Beck gauging station showing how extrapolated falling limb was reduced to zero over calculated average time base

2.28 In summary, the typical hydrograph method as proposed by Archer et al. (2000)[5] was deemed to be the most appropriate approach for deriving hydrograph shapes in groundwater dominated subcatchments of the Lower Witham catchment. However, it has limitations which mean the user must apply it with necessary adaptations. The most significant limitations result from the following factors:

- The number of observed events available from which to derive the typical hydrograph may be limited, and are often not of a magnitude close to that of the rarer AEP events;
- Very few observed events result from a single design profile storm of consistent duration and similar antecedent conditions.

2.29 The adaptations that have been applied as described above are considered to be appropriate to the requirements of this study.

Slea hydrological inputs

2.30 The hydrological model for the Slea catchment (including Ruskington Beck and Leasingham Drain) was previously developed and described in the Final Hydrology Report[2]. In summary, two FEPs were identified on the River Slea itself – one at the upstream limit and one immediately upstream of Cranwell Line Sluice – and on the Ruskington Beck and Leasingham Beck watercourses (both at the downstream limits of these reaches).

2.31 The Slea catchment was subdivided into five subcatchments – one each for Ruskington Beck and Leasingham Beck, and an upper, middle and lower subcatchment for the River Slea. The Ruskington Beck, Leasingham Beck and upper Slea subcatchments were expected to be scaled directly and the middle Slea subcatchment scaled in order to achieve the correct peak flow at Cranwell Line Sluice. All four of these subcatchments would have flow-time boundaries to reflect their permeable nature. The lower Slea subcatchment was expected to include an unscaled FEH/FSR rainfall-runoff boundary.

2.32 During the development of the hydraulic model in conjunction with the hydrological model, it was found that using an unscaled inflow for the lower Slea subcatchment resulted in an unrealistically high peak flow which caused unrealistically precautionary flood extents. To address this issue, an FEH statistical analysis was conducted to achieve a more realistic idea of the peak flow that would be generated from the lower Slea subcatchment. The catchment descriptors were compared with gauged sites in the Lower Witham catchment and it was concluded that Stainfield Beck at Creampoke Farm was the most applicable site from which a *QMED* transfer could be made. This site was also used as the donor for a pooling group and hence growth curve.

2.33 The calculated peak flows for the lower Slea subcatchment are given in Table 2.4. Note that the 0.1% AEP flows were derived using the hybrid method as described in the Final Hydrology Report[2].

AEP (%)	50	20	10	5	4	2	1.33	1	0.5	0.1
Flow (m^3s^{-1})	0.5	0.8	1.0	1.2	1.3	1.5	1.6	1.7	1.9	5.2

Table 2.4: Statistical peak flows from the lower Slea subcatchment (FEP 31)

Choice of storm duration

2.34 It was originally proposed that the design event model runs would need to be undertaken for a maximum of three storm durations, reflecting how peak flows generated by a given return period rainfall event are larger in the tributary subcatchments and upper reaches of the Lower Witham catchment for short duration storms, and larger in the lower reaches of the Lower Witham catchment for longer duration storms.

2.35 Investigations into the effect on peak flow of varying the duration of the storm concluded that the 20.5 hour storm could be applied in all tributary subcatchments and still be within 5% of the (unscaled) peak flow, as calculated for the critical duration of that particular subcatchment.

2.36 The critical duration on the River Witham within the Lower Witham catchment is highly dependent on the critical durations of the subcatchments feeding it. The approximation formula cited in FEH Volume 3 for determining the critical duration is less applicable for large, complex catchments. This is particularly the case for the Lower Witham, where inputs from pumped subcatchments, groundwater dominated subcatchments and direct runoff dominated subcatchments are combined.

2.37 The critical duration on the River Witham through the lowland areas of the Lower Witham catchment is more concerned with peak volume than peak flow. As the critical duration at Lincoln in the Upper Witham catchment was 50 hours, and the critical duration in the Bain catchment was 45.5 hours – both of which account for peak volumes in the system – it was considered appropriate to choose a similar duration to apply in the Lower Witham catchment.

2.38 In order to consider the impact at any other storm duration, it would be necessary to run models of the Upper Witham and Bain to obtain the appropriate inflows from these catchments. However, this was outside the scope of the project.

2.39 Therefore, an average value of 48.5 hours was determined to be appropriate to the Lower Witham catchment. This was only applicable to the FSR/FEH rainfall-runoff based inflows (i.e. the pumped subcatchments and the direct runoff dominated subcatchments). It is not possible to account for the storm duration within the method used to determine the inflows from the groundwater dominated subcatchments, although it could be argued that the additional rainfall volume generated from a longer event would result in greater groundwater recharge, with little impact on the resulting flows to the main channel.

Upper Witham Inflows

2.40 It should be noted that there is a known inconsistency in the 50% AEP flow boundary inputs into the Witham and South Delph at the upstream end of the model. The issue is that the 50% AEP boundary has a slightly higher peak flow than most of the other modelled events of higher magnitude. This issue is illustrated graphically in Figure 2.8. It can be seen that the flow from for the 50% AEP event has a higher initial peak than much rarer events, although the peak is relatively short-lived. The reason why this occurs is that for the Upper Witham FMI model runs, the Witham washlands do not operate in the 50% AEP event, but do operate for rarer events.

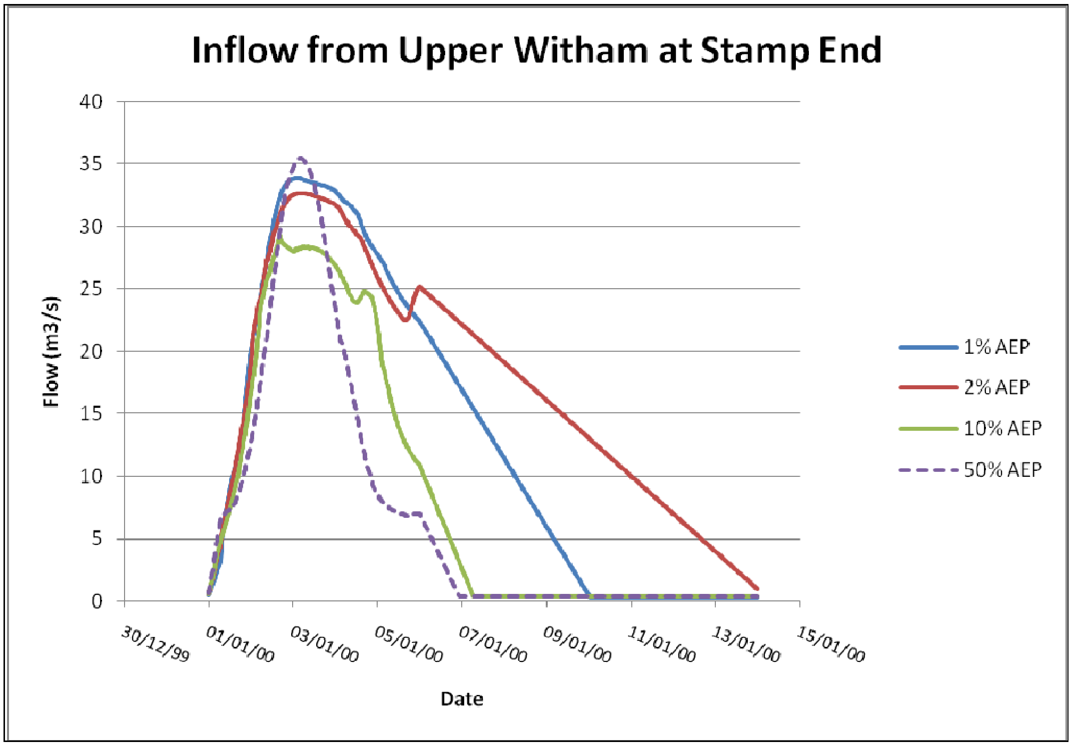

Figure 2.8: Inflow hydrograph from Upper Witham model at Stamp End

2.41 The discontinuity in peak flows at Stamp End is not reflected in the peak water levels; the 50% AEP water level is the lowest of all modelled return periods as would be expected. This discontinuity was discussed with the Environment Agency and it was concluded that this is a possible and realistic scenario that could occur for an event in which the washlands were not operable.

IWRS INFLOW BOUNDARIES

2.42 Following the development of the peak flow estimates and the decisions regarding the most appropriate method to develop the design event hydrographs, the hydrological boundaries were set up in InfoWorks RS.

2.43 For the groundwater dominated subcatchments, the inflow hydrographs were incorporated within the model as Flow-Time boundaries. Application of a Flow-Time boundary in InfoWorks RS combines a network based boundary data with the event based flow-time data. The majority of Flow-Time boundaries had been scaled to the required statistical peak separately from the InfoWorks RS model. The only exception was for the Slea – Middle subcatchment (LW_S_SM). Firstly, this inflow boundary was divided equally into three components, representing spring inflows to the main river (namely Cobbler's, Boiling Wells and Guildhall). Secondly, as the flow estimation point (site 29 – Slea at Cranwell Line Sluice) is on the main river, it was necessary to perform trials with these inflows scaled until the required peak flow at the estimation point was achieved.

2.44 For the direct runoff dominated and pumped subcatchments, the inflows were generated by FSR/FEH Rainfall-Runoff boundaries. Application of the FSR/FEH Rainfall-Runoff model in InfoWorks RS combines a network based boundary node (containing catchment descriptor and unit hydrograph details) with an event based rainfall boundary (containing rainfall depth, duration and profile details, return period specification, catchment wetness and baseflow values, and the boundary scaling factor).

2.45 As the network based boundary node data is fixed and does not alter for different design events, only the event based rainfall boundary information requires adjustment depending on the design event being simulated.

2.46 For each design event return period, a set of rainfall boundaries was developed (one for each subcatchment) with the required design event parameters. All the pumped subcatchments had fixed scaling factors irrespective of the event return period. The majority of the direct runoff subcatchments were scaled for each return period separately from the InfoWorks RS model. This was because they were either direct inputs to the model, or the modelled reach within the subcatchment was of negligible length.

2.47 The remaining direct runoff subcatchments were scaled using the hydraulic model and an iterative approach was required. This was due to the targeted statistical peak flow being a product of multiple upstream subcatchment inflows, and also due to the hydraulic impacts in flow upstream of the flow estimation point. A summary of this is provided in Table 2.5.

Site code	Subcatchments	Comments
Site 2 (Barlings Eau @ Langworth)	LW_BE_1, LW_BE_2, LW_BE_3, LW_WEB	- FEP (Langworth gauge) is on modelled reach - Combination of flows from LW_WEB, LW_BE_1 and LW_BE_2, plus hydraulic impact of modelled reach, determines peak - Inflow from LW_WEB is fixed - Three subcatchments of Barlings Eau scaled equally
Site 6 (Stainfield Beck @ Creampoke Farm)	LW_SB_1, LW_SB_2	- FEP (Creampoke Farm gauge) is on modelled reach - Inflow from LW_SB_1, plus hydraulic impact of modelled reach, determines peak - Both subcatchments of Stainfield Beck scaled equally
Site 7 (Snakeholme Drain)	LW_SD	- FEP is at downstream limit of modelled reach - Inflow from LW_SD, plus hydraulic impact of modelled reach, determines peak
Site 8 (Duckpool Drain)	LW_DD, LW_DD_1	- FEP is at downstream limit of LW_DD subcatchment - Inflow from LW_DD, plus hydraulic impact of modelled reach, determines peak - Both subcatchments of Duckpool Drain scaled equally

Table 2.5: Summary of scaling process for flow estimation sites where an iterative approach using the hydraulic model was required

2.48 Following iterations, a finalised set of scaled rainfall boundaries was achieved for each design event and saved as a stand alone InfoWorks RS event file, which could later be imported into the design model.

2.49 In addition to the event based inflows (from direct runoff dominated subcatchments, groundwater dominated subcatchments and pumped subcatchments) the model also includes ‘baseflow’ inflows at the upstream limit of each modelled reach. These are a low constant flow at which it is known the model simulation is stable. The term ‘baseflow’ is loosely applied here, as in hydrological terms, baseflow varies throughout an event, and for the design method should be calculated from flood event analysis or catchment descriptors. Often however, these values are too small to allow the model simulations to complete, which is why alternative fixed baseflow values are specified.

2.50 At each flow estimation point, the combined baseflow inputs are subtracted from the target peak flow, so that the design event inflows are not over-scaled. The difference between the baseflow as calculated from catchment descriptors, and the value specified in the model are too small to have a significant impact on the volume of the FSR/FEH rainfall-runoff inflow hydrographs.

2.51 The model also includes a series of ‘sweetening’ inflows. These are also required to maintain model stability at low flows, but are different to baseflow in that they are invariably not added at upstream limits of modelled reaches, and are also removed downstream. An example is the sweetening flow through Branston Delph, which is set-up so that there is always a small flow in the channel, even when both ends of the Delph are closed off. It is added in at the upstream section of the Delph (downstream of the penstock) and removed at the downstream section (upstream of the flapped outfall).

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AECOM

Defended Model Development

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3 Defended Model Development

INTRODUCTION

3.1 The defended hydraulic model of the Lower Witham was developed from the parent InfoWorks RS (IWRS) model constructed and calibrated by AECOM under the Witham Catchment Strategic Model (WiCSM) commission[6]. This model was in turn based on an ISIS model of the Lower Witham developed previously for the Lower Witham Strategy Study. One benefit of this model lineage was that the WiCSM parent model already contained basic 1D representations of the main flood plains and fens, and these were carried forward to the Lower Witham FMI model. However, significant lengths of modelled tributaries were also added during construction of the WiCSM model, and improvements were made to the schematisation of the 1D storage areas representing the lowland fens. During this development of the WICSM parent model, the additional modelling of floodplains was limited to the levels/extents observed during the calibration events. Based on the hydrological analysis carried out for this study, the two flood events against which the parent model was calibrated and verified were relatively small in magnitude (for example, the largest calibration event flow was close to a 20% AEP on the Barlings Eau and Stainfield Beck tributaries and a 5% event on Dunston Beck).

3.2 Due to the combination of the extension of the model prior to recalibration, the relatively small magnitude of the calibration/verification events, and the requirement for the Lower Witham FMI model to accurately model/map all flows up to and including the newly derived 0.1% AEP event plus an allowance for climate change, significant development of the WiCSM parent model was required to make it fit for flood map improvements modelling. This primarily involved extending cross-sections, expanding storage areas and adding new storage areas and lateral spills/floodplain sections. Additional refinements were also made to the model to add detail, improve the schematisation, and to represent design scenarios for hydraulic control structures where necessary.

3.3 This section describes development of hydraulic representation of the model. A description of the flood mapping components of the model is provided in Section 4.

MODELLING SOFTWARE

3.4 The modelling software used was InfoWorks RS, developed by Wallingford Software. InfoWorks RS is a modelling software package that combines the ISIS 1D flow simulation engine[7], GIS functionality and database storage within a single software package. This offers the capability to undertake model building, data management, flood mapping and damage assessment within one software package. The software includes full solutions of open channels, floodplains, embankments and hydraulic structures for steady and unsteady flow conditions. Rainfall-runoff simulation uses both event based and conceptual hydrological methods known as design events. Full interactive views of data are available using geographical plan views, sectional views, long sections, spreadsheet and time varying graphical data. At the time of selection of the IWRS software for the WiCSM project, the main advantages of InfoWorks RS over modelling software such as ISIS were that it can produce flood extent maps by combining the model results with a Digital Terrain Model (DTM) and the data base storage facility. However, ISIS has caught up somewhat in the interim with the introduction of the 'ISIS Mapper' add-on.

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[6] Witham Catchment Strategic Model, Hydraulic Modelling Report – Volume 4: Lower Witham sub-model. September 2007 (DRAFT).

[7] With the release of InfoWorks RS v9.0 in April 2008, 2D capability was introduced into the software subject to the purchase of an additional 2D licence.

3.5 The GIS functionality and the database storage within InfoWorks RS have been very useful for the Witham Flood Map Improvements project due to the large volumes of data involved. The software has allowed the creation of hyperlinks to photographs of the structures, survey drawings, structure data sheets and any other useful information to the relevant nodes within the model. GIS data and model structure has been overlaid within the software package, which has greatly aided the modelling process. Extended cross-sections and level area relationships for storage areas have been derived from the LiDAR/SAR DTM data within InfoWorks RS.

MODELLING APPROACH

3.6 The Lower Witham WiCSM calibration model was originally conceived and developed as a purely one-dimensional (1D) hydraulic model. However, the release of InfoWorks RS 2D software[7] presented the opportunity to incorporate two-dimensional (2D) representations of some of the floodplains for the defended flood map improvements model. It was agreed at the proposal stage that AECOM would include a limited amount of 2D modelling in the Lower Witham where this offered an obvious benefit, and this is described in more detail in section 3.36.

3.7 Following development of the Lower Witham defended flood map improvements model, unsteady simulations were undertaken to generate water levels within the river channel and associated floodplains using flows derived from the hydrological analysis.

3.8 The hydraulic model simulates the attenuation and the lag effects of a river channel, floodplain and artificial influences on flow and level as water moves through the river system. A variety of structures and controls can be modelled within hydraulic models such as IWRS.

3.9 The representation of any complex system by a model requires a number of assumptions to be made. The key assumptions of a computational hydraulic model are:

- The hydrological analysis based on the gauged data (where available) can be extrapolated to other parts of the system;
- The design flows are an accurate representation of flows of a given return period;
- The survey data accurately represent the geometry of the river channel and banks;
- The physical processes in the fluvial system are accurately described by the underlying equations;
- The fluvial system has been adequately schematised by the modeller;
- In the case of a 1D hydraulic model it is assumed that the hydraulic processes operating within the catchment are essentially 1-dimensional and do not vary within a cross section either horizontally or vertically.

3.10 These assumptions are generic for all computational hydraulic models and therefore apply to all Witham Flood Map Improvements studies. However, a number of assumptions specific to the Lower Witham study have also been made and these are detailed in the remainder of this section as appropriate.

3.11 If recorded data is available calibration and validation of hydraulic models should be undertaken to ensure comparison between simulated and observed data. This has previously been carried out as part of the WiCSM – Lower Witham calibration study using two discrete flood events: November 2000 and February 2001.

TOPOGRAPHIC DATA

3.12 In the WiCSM Lower Witham parent model, the in-bank model was constructed using channel survey data from a variety of sources:

- ESL survey 1992;

- Redland survey 1998/2000;
- Surveyline 2006;
- Halcrow 1992;
- EDI 2006;
- CSL 2006;
- Ratcliff 1998;
- Bullen 2001;
- Atkins 2002.
- Lower Witham Phase 2/3 as-built drawings, Bullen 2004.

3.13 During development of the defended Lower Witham flood map improvements model, it was necessary to extend significant numbers of cross-sections and add significant numbers of lateral spills and storage areas to model the floodplains. Lateral spills were defined using long section bank top data from the associated channel surveys where these were available. In other cases, it was necessary to base lateral spills and storage areas on the best available digital terrain data.

3.14 Two forms of digital terrain data were made available by the Environment Agency for this study: LiDAR and SAR data. The resolution of the LiDAR data is on a 2m grid and was provided in filtered (v_ascii) format, whilst the SAR data is based on a coarser 5km grid. Generally speaking, LiDAR is considered the more accurate of the two data products and was used in preference to SAR where available. LiDAR coverage was virtually complete for all main river and floodplains in the Lower Witham catchment (Figure 3.1).

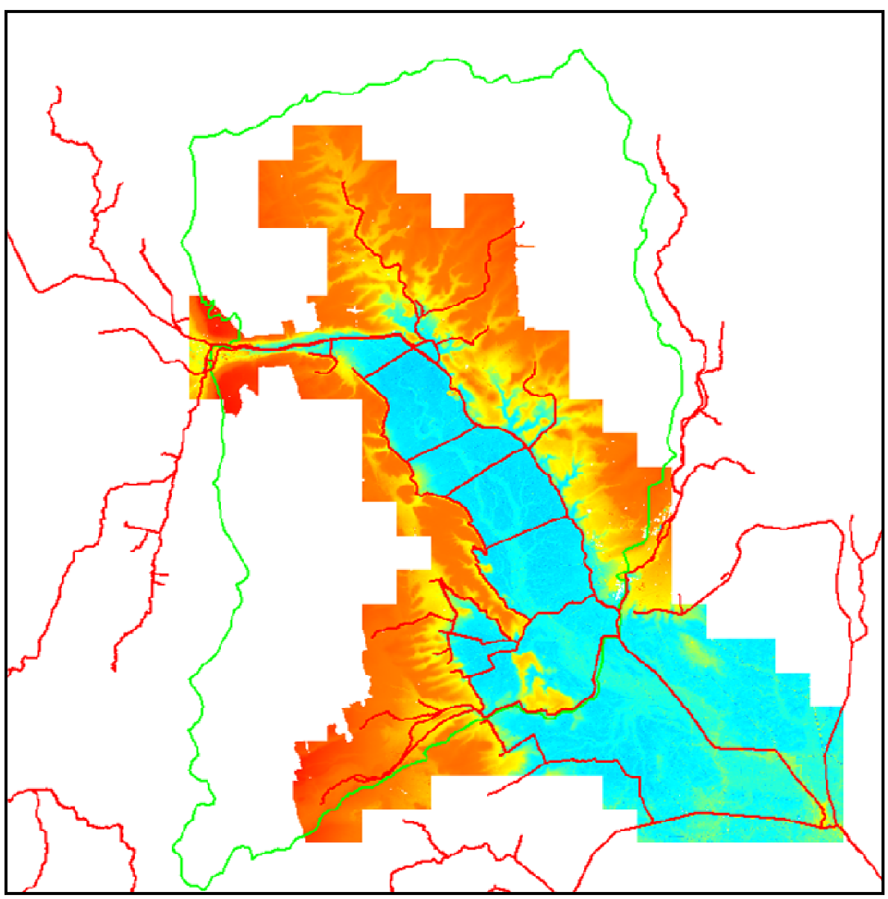

Figure 3.1: Extent of LiDAR coverage (shaded) in the Lower Witham catchment (green outline).

3.15 It should be noted that since the construction of the WiCSM parent model, the LiDAR data provided by the Environment Agency for most of the area off the left bank of the Witham between the River Bain and Duckpool Drain, has been deemed as being of poor quality by the Environment Agency's National Centre for Environmental Data and Surveillance (NCEDS) and has therefore been removed from the national LiDAR dataset. Following consultation with the Environment Agency, it was agreed that the available LiDAR would be used for this study despite these concerns, in the absence of more accurate data. It is noted that the LiDAR level data used in the Lower Witham FMI model development broadly matches levels from the various surveys and the SAR data, and therefore, whilst it may not be accurate enough to satisfy NCEDS strict quality assurance checks, it is considered to be of sufficient accuracy to produce reasonably reliable flood mapping.

3.16 In order to use the digital terrain data to extend the model in InfoWorks RS, it was necessary to import the data into the IWRS software to produce a DTM. Prior to doing this, a composite set of ascii data was produced by 'stamping' the LiDAR data on top of the SAR data using the Vertical Mapper software. This produces a single ascii dataset giving complete coverage of the catchment, with the more accurate LiDAR data taking precedence where available. This technique has the added benefit of filling in gaps in the LiDAR, such as those left where ponds exist, and aids in the flood mapping process.

REPRESENTATION OF FLOODPLAINS

3.17 This section details the approach taken in development of the Lower Witham defended FMI model to model floodplain flow. This involved the addition of schematic representations of the floodplain to the Lower Witham WiCSM calibration model. The main four types of hydraulic units used to represent the floodplain were: extended cross-sections, spill units, floodplain sections, and storage areas.

3.18 Specific details of where floodplain representations were added in comparison to the WiCSM calibrated parent model are not provided since this is best demonstrated within the IWRS software, either using a visual side-by-side comparison, or by using the *compare networks* functionality.

3.19 Development of the flood mapping model, which sits alongside the hydraulic model and enables InfoWorks RS to produce the flood mapping, is described in Section 4.

Extending cross-sections

3.20 Where there are no formalised embankments on the channel and the adjacent floodplain does not fall away significantly from the bank top level, the floodplain may be modelled by extending cross-sections onto the valley sides. This is the simplest method of modelling the floodplains and is relatively straightforward using InfoWorks RS, which can pick up levels from the DTM using a designated spacing and an extension line drawn onto the Geoplan. This method of floodplain representation was used wherever possible in developing the Lower Witham defended FMI model and was appropriate to large parts of the Barlings Eau and small tributaries of the Witham where the river is undefended and falls in the lowest point of a well defined valley.

3.21 In extending cross-sections, it is important to ensure that cross-section lines do not cross each other as this goes against the assumptions of 1D floodplain flow and makes flood mapping problematic. Instead, the required approach is to ensure that cross-section extensions follow the general direction of flow along the valley, even though this can require a 'dog-leg' in the cross-section line.

3.22 Figure 3.2 shows an example of a cross-section extended using the DTM and its location in the upper reaches of the Barlings Eau. Building on the experience of undertaking the River Bain FMI study, cross-sections were generally extended using a resolution of 5m minimum spacing as a spacing of 2m was considered to be unnecessarily small in most cases.

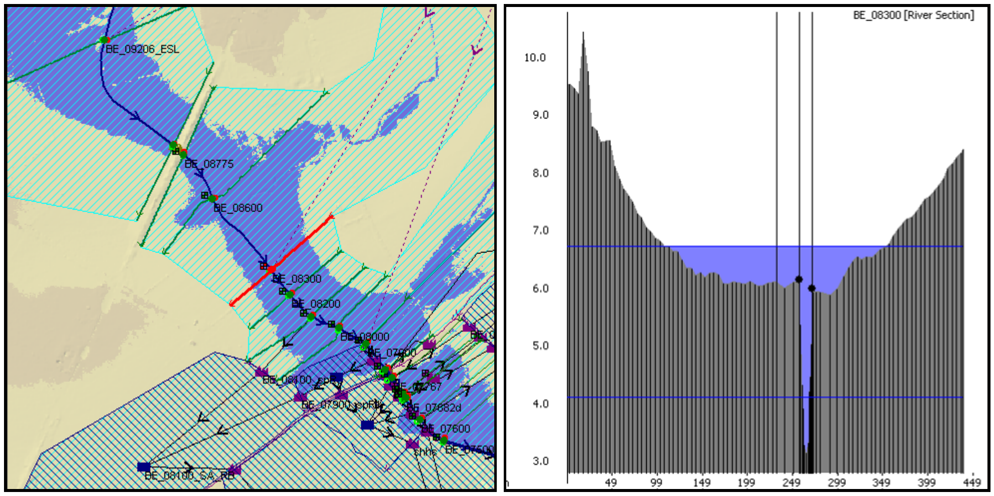

Figure 3.2: Example of a cross-section extended using DTM data (right) and the location illustrating the general approach to the orientation of cross-sections (left)

Spill and Floodplain Units

3.23 Where the channel is embanked, or the adjacent floodplain falls away significantly from the bank top level, it is better to model flow overtopping the banks by means of an overbank (lateral) spill unit in IWRS. Such a spill unit is typically connected to an adjacent storage area in a 1D model, although it can also be connected to a parallel channel representing the floodplain. In the case of a linked 1D-2D model, a spill can also be connected to a 2D simulation polygon.

3.24 Spill units can also be used to allow flow to spill between adjacent storage areas on the floodplain during periods of high flow. This is done where the floodplain is modelled as a series of interconnecting storage areas, having boundaries along topographic boundaries such as man-made embankments or natural ridges of higher land. In the Lower Witham defended FMI model, *Floodplain* sections were typically used in preference to spill units for the connection of floodplain storage areas. Floodplain sections are considered to be more physically realistic for this application due to the fact that they can calculate flow between connected flood storage areas using the Manning equation, thereby taking into account friction in the floodplain.

3.25 Spill units can also be used to model flow overtopping a structure, such as the bypassing of a surcharged bridge by means of flow over the deck or around the parapet. In this application the spill unit usually connects flow back into the channel downstream.

3.26 Spill and Floodplain units are identical in terms of their topographic data requirement within IWRS: both require a series of chainage and elevation data points along the high point of the embankment. For lateral spills representing flow over the banks between successive river sections, there are three potential sources of data available to define the spill:

- Long section bank top survey data;
- Bank elevations from cross-sections;
- LiDAR/SAR DTM data.

3.27 In theory the long section bank top survey should be the most accurate data source for definition of the spill. However investigations showed that in many cases the long section survey picked up the main break of slope of the embankment rather than the true crest. In addition, incoming drainage ditches were often included in long section bank top datasets when in reality these do not form the actual line of defence. Following consultation with the Environment Agency it was agreed that as a general approach the bank elevations from cross-section surveys would be used to define spills with additional low spots from the bank top long-section in-between. Where the low spot represents an incoming drain it was agreed to ignore the low spot if it appeared that the drain is embanked on the OS 10,000 scale basemap. Where this banktop survey data was not available it was necessary to use the bank elevations from the upstream and downstream cross-sections and assume a linear interpolation in between. Any significant low spots in the spills were queried with the Environment Agency as part of the review of the draft flood mapping. Where appropriate unrealistic low spots were raised to a more appropriate defence level.

3.28 In most cases the LiDAR/SAR DTM was found to be unsuitable for defining bank top spills because the filtering technique has left unrealistic gaps in the embankments, and in some cases filtered the embankments out completely.

3.29 For spills modelling flow bypassing a structure, elevation data was obtained from the upstream structure cross-section, although in a number of cases it was necessary to extend these spills beyond the lateral extents of the surveyed section. The data for extending these spill units was taken from the LiDAR/SAR DTM within InfoWorks RS. Similarly, the DTM was the main source of data for definition of spills and floodplain sections between storage areas. In both cases a typical spacing of 5 to 10m between points was used. An example of a floodplain section on the River Slea is shown in Figure 3.3.

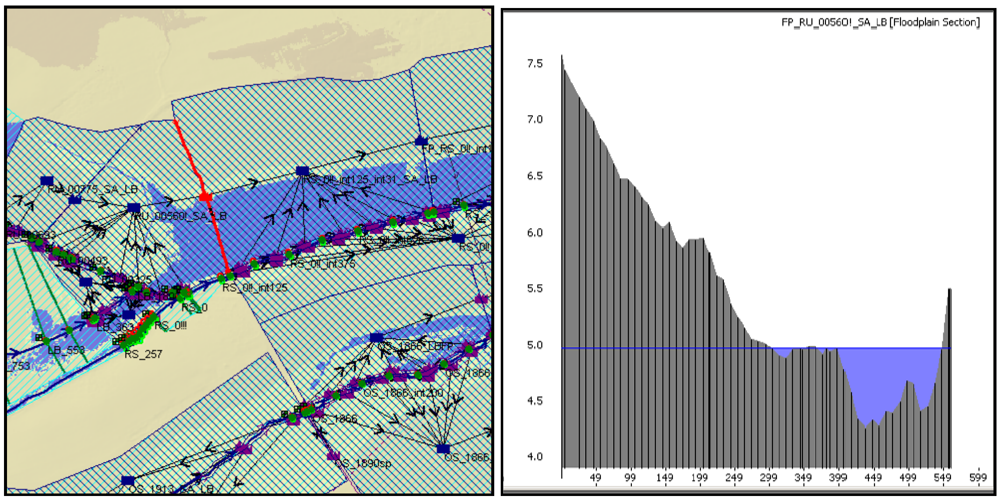

Figure 3.3: Example of a Floodplain spill unit (right), and its use in connecting two floodplain storage areas (left) adjacent to the River Slea

3.30 Wherever spills were generated using the LiDAR/SAR DTM, visual checks were made using 3D views in IWRS to check that the spill followed the line of maximum elevation. In some cases the route of the spill picked up anomalies in the DTM such as unrealistic 'holes' or 'spikes' in the data. Where it was considered that such anomalies would have had a significant effect on the flood mapping results, the data anomalies were manually removed from the spill units.

Between 2002 and 2007 a programme of bank works, referred to by the Environment Agency as 'Lower Witham Phase 2/3 works' was undertaken. This involved mainly structural remediation works but included some bank raising, thereby superseding parts of the last full topographic survey of the banks. It is understood that the Lower Witham strategy ISIS model has previously been updated to include this work. However, as a precaution, all relevant spill units were checked against the Phase 2/3 as-built drawings and updated if necessary.

Storage Areas

3.31 Storage areas were used to model areas of floodplain where over bank flow would collect. Water from the main channel can flow into these areas via spill units and may drain back into the main channel as the flood subsides. Water may also pass downstream along the floodplain to an adjacent storage area, connected by a spill or floodplain section unit.

3.32 The storage areas representing the floodplain were manually defined within the Lower Witham FMI defended model taking into account topographic boundaries such as roads, embankments and natural high ground. In a 1D model such as this, the storage areas model floodplain inundation using a 'spilling bucket' principle. Each storage area will fill, gradually inundating an area, until it reaches a threshold spill level where it spills into another unit such as a storage area or river channel (Figure 3.4). The inundated area ignores the location of where any water enters the storage area and fills the lowest point on the storage area first. Therefore, the resultant flood mapping does not show the flow path and this must either be added manually in post-processing or modelled more accurately using a 2D approach to determine the flow path.

3.33 The use of 1D storage areas to model the floodplains in this way can therefore be a source of inaccuracy in the flood mapping, especially if there is a significant gradient on the floodplain, and a lack of topographical barriers that lead to ponding of flood water in reality. Such inaccuracies can be minimised by carefully selecting the boundaries of the storage areas representing the floodplain, and by ensuring that each floodplain is broken down into an appropriate number of storage areas to ensure that the mapping is not overly dependant on the manual definition of flow paths. In some cases the 1D storage area approach may be considered too inaccurate to represent floodplain flow and these cases a linked 1D-2D model or a simplified 2D model to define the flow path is the better alternative. In the Lower Wiitham study, the main hydraulic model was set up using a 1D approach, with a series of child sub models using 2D modelling developed to improve the mapping where required. Further details of the 2D approach adopted are provided in Section 3.36.

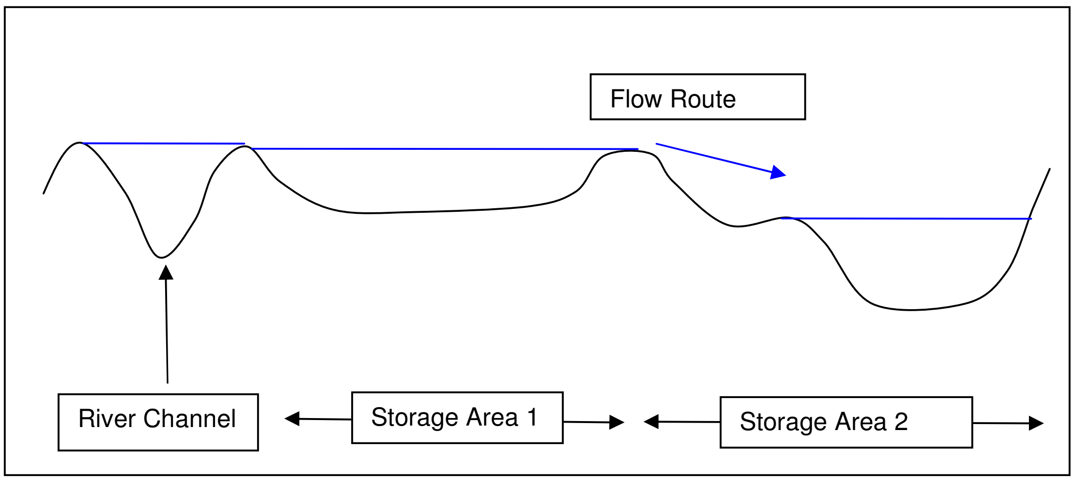

Figure 3.4: Example of a 1D ('spilling bucket') representation floodplain inundation using connected floodplain storage areas

3.34 Generally, the more problematic areas to model using a 1D storage area approach in the Lower Witham FMI defended model, tended to be in the lowland fen areas, such as those bordered by the Delphs to the west of the River Witham (Figure 3.5). These fens are so vast and flat that it is very difficult to accurately map inundation from overbank spills, even if the Fen is sub-divided into a series of connected storage areas.

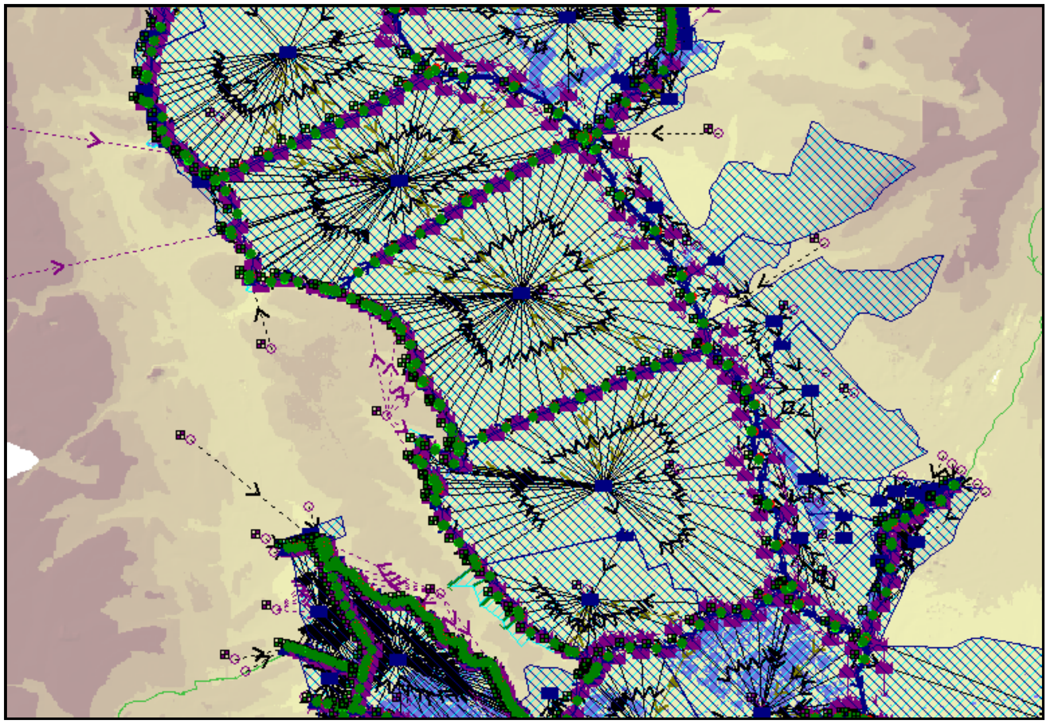

Figure 3.5: Lowland Fens bordered by 'The Delphs' on the Lower Witham in which a 1D approach to mapping proved to be inaccurate

3.35 For each 1D storage area that was defined, a level-area relationship was generated automatically using the LiDAR/SAR DTM within InfoWorks RS. This level-area relationship enables the model to calculate the volume at a particular elevation, and therefore to calculate the flood level from the balance of flows in and out of each storage area. The level-area relationship can be updated quickly and easily within the software should a more accurate DTM be obtained in the future.

2D FLOW MODELLING

3.36 For areas where 1D modelling using the main model proved to be too inaccurate to map flow paths, a series of child sub-models using 2D modelling were developed in Infoworks RS to improve the mapping. In the case of the lower reaches of the River Bain adjacent to Coningsby and Tattershall, this was done using a hydraulically linked 1D-2D approach. For other locations such as in the Fens to the west of the Witham, a simplified 2D approach to define the flow path was considered to be adequate. Further details of these two approaches are provided below:

Lower Bain 1D-2D Linked Modelling

3.37 In the River Bain FMI study it was found that a 1D approach to floodplain modelling was inadequate to describe the potential flow path of water overtopping the right bank of the Bain around Wharfe Lane in Coningsby. This was because the floodplain falls away gradually in both a southerly direction (across Wharfe Lane and back towards the watercourse) and in a south-westerly direction towards the urban area of Tattershall. In addition, a dismantled railway embankment runs east to west across the floodplain in this area which provides a discontinuous barrier to flow. Given these issues, and the importance of obtaining accurate mapping for a large urban area such as this, it was concluded that this area should be modelled using a linked 1D-2D approach. Subsequent discussions with the Environment Agency concluded that the best approach would be to undertake this work as part of the Lower Witham FMI study.

3.38 For initial development of the Lower Bain linked 1D-2D model, a truncated version of the River Bain FMI model was extracted from the full River Bain 1D FMI model. This truncated model extended from the confluence with the Witham to a point just upstream of Wharfe Lane in Coningsby. Design inflow boundaries were extracted from the full River Bain model for use as upstream flow boundaries for the truncated model.

3.39 Unfiltered LiDAR was provided by the Environment Agency to get a better handle on the height and extent of the disused railway embankment, although ultimately the filtered LiDAR data was used to specify a 2D simulation polygon to model flow moving in a south-westerly direction towards the floodplain of the Witham. The default Infoworks RS resolution of $100m^2$ maximum triangle size, and $25m^2$ min element area was considered appropriate for the 2D simulation polygon. Within this 2D simulation polygon, the continuous length of the disused rail embankment was represented by an impermeable breakline. The 2D simulation polygon was linked to the 1D overbank spills from the right bank of the River Bain.

3.40 Following discussions with the Environment Agency, it was agreed that the linked 1D-2D model would be extended to also include the left bank floodplain of the Lower Bain as a linked 2D area, in order to provide more accurate modelling of flows over the left bank of the River Bain around Coningsby and subsequent flow paths in a southerly direction behind the existing defences. This was driven by a proposal to 'mothball' a large section of left bank defences on the Lower Bain with the provision of a crossbank to contain flooding within the natural topography of this essentially rural area. As part of this extension data, AECOM incorporated the two Environment Agency siphons (NGR 521922, 357955 and 521447,357788) passing beneath the River Bain to directly link the two sides of the floodplain. The Environment Agency confirmed that these siphons do not have penstocks that allow them to be closed off and would therefore be allowed to pass flow beneath the River Bain during a flood event.

3.41 The initial intention was to paste this linked 1D-2D sub-model back into the main model to run as part of the main model. However, attempting to link the 2D simulation polygons back into existing 1D storage areas bordering the Witham proved problematic, with much non-convergence caused by recirculation over the long connecting spills. Similar problems were encountered in linking the 1D representations of the IDB pumps into the 2D domain as this had to be done via a dummy 1D storage area. In addition it was found that the 2D domain required a much smaller timestep than the main 1D model and thus incorporating the 2D areas into the main model would severely affect the run time of the main model. It was subsequently decided to keep the Lower Bain 1D-2D linked model as a separate sub-model with the flow paths generated by the model superimposed onto the 1D mapping during post processing. A linked 1D-2D approach was also attempted on parts of the floodplain of the River Slea and Leasingham Beck, but similar problems with recirculation, non-convergence and small timesteps were encountered. Therefore it was decided it would be more appropriate to define these flow paths using the simplified 2D flow path methodology detailed below.

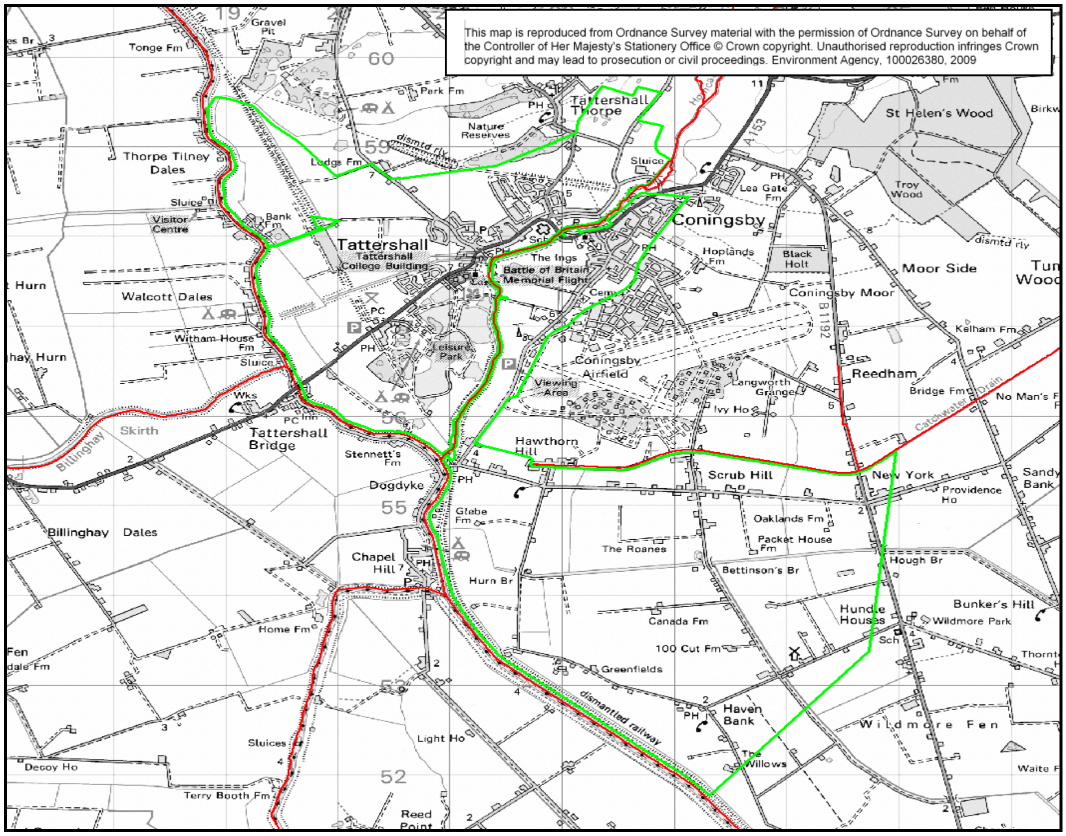

Figure 3.6: Extents of the Lower Bain linked 1D-2D sub-model (2D simulation polygons highlighted in green)

3.42 Figure 3.6 shows the extents of the Lower Bain linked 1D-2D model. It was found that flow over the right bank of the River Bain eventually ponded behind the defences on the Witham, whilst flow over the left bank could in theory continue flowing south-east towards Boston. However, in reality it was found that the flow did not progress much further south than Dogdyke due to its dissipation into the existing drainage network and/or topographic features. This was proved by completing a test run which extended the 0.1% AEP plus climate change model run-time.

3.43 The final Lower Bain1D-2D linked model was run for the 1% and 0.1% AEP events plus their climate change equivalents. It was not considered necessary to simulate lower magnitude events because there were no significant flow paths at these return periods and the existing 1D mapping from the main model was considered adequate.

Simplified 2D Flow Paths

3.44 In examining the draft mapping from the initial 1D simulations using the main Lower Witham FMI model, it became apparent that there were a large number of locations where flow was overtopping into 1D storage areas but the mapping was not showing any flooding in the immediate vicinity. This is because the 1D storage area approach assumes that flow fills the fen from the lowest elevation which in the case of large fens can be tens of kilometres away. A related problem is that the fen storage areas are so vast that even significant overbank flows can fail to show significant flooding at any concentrated location because the flows into the storage areas preferentially fill up the vast volume available in the network of IDB drains.

3.45 The option of manually generating flow paths was considered as this was successfully applied on the River Bain FMI project. However, with the areas in question being so flat, and the location of any ponded flooding being so distant, it was considered that this would be simply too subjective and inaccurate. The option to add fully linked 2D areas to the existing 1D model was also considered, but was considered to be too costly and time consuming based on the experience of using this approach on the lower reaches of the Bain. An alternative methodology based on constructing simplified 2D flow paths was devised as the best way to produce accurate flow path mapping at minimum cost.

3.46 The simplified 2D flow path method involved generation of a number of 2D simulation polygons in Infoworks RS, each one representing a particular fen or flow path route. A relatively coarse resolution was used for the 2D mesh with a maximum triangle size of $1500m^2$, with a minimum computational element area of $50m^2$. This was considered adequate to describe the flow paths with reasonable accuracy. Overbank spill hydrographs were then extracted from the 1D hydraulic model and applied to the relevant 2D simulation polygon as point sources. It was only necessary to run the 2D simulations for those return periods that resulted in overtopping of spills in the main 1D model.

3.47 Initial trials using the simplified 2D flow path methodology proved successful, providing a relatively quick and easy method of accurately determining the likely flow path to enhance the 1D flood mapping. As a result, the methodology was extended to more areas. A total of 13 separate 2D simulation polygons within 9 separate models were eventually constructed to determine flow paths. The areas covered by these models are shown in Figure 3.7 and are summarised in Table 3.1.

2D Area No.	Description	Primary Flow Sources*
1	Washingborough Fen	Sandhill Beck LB
2	Heighington Fen	Sandhill Beck RB, Heighington Beck RB
3	Langworth to Low Barlings	Barlings Eau RB
4	Branston Fen, Potterhanworth Fen & Nocton Fen	Nocton Delph LB
5	Bardney & Southrey	Duckpool Drain RB, Snakeholme Drain LB
6	Dunston Fen & Metheringham Fen	Nocton Delph RB and Car Dyke LB
7	Blankney Fen & Martin Fen	Timberland Delph LB and Car Dyke LB
8	Timberland Dales & Thorpe Tilney Fen	Timberland Delph RB and Car Dyke LB
9	North Kyme Fen & Damford Grounds	Billinghay Skirth RB and Kyme Eau LB
10	Leasingham Moor	Leasingham Beck RB
11	Upper Slea @ Bouncing Hill	River Slea RB
12	South Kyme Fen & Holland Fen	Kyme Eau RB
13	Dogdyke to Boston	River Witham LB

Table 3.1: Summary of 2D Areas covered by 2D Flow Path Modelling
***(RB = Right Bank, LB = Left Bank)**

3.48 The main limitation of the simplified 2D flow path approach is that there is no hydraulic feedback between the overbank spill and the adjacent ground. This feedback has the potential to cause a drowned flow condition, and thereby reduce the flow over the spill in comparison to a 1D spill unit flowing freely into a 1D storage area. However, in the majority of cases the height of the river embankments and the size of the adjacent fens is such that free weir flow is likely to be a reasonable assumption at most overtopping locations. In any case, the assumption of free flow in determining the flow paths would be the same as for a 1D model and is precautionary.

3.49 Once model runs were completed, a number of sensitivity runs were undertaken with a longer simulation period to check the spreading of the flow had reached its maximum. The final flow path outlines were superimposed on the 1D outlines from the main model to produce a composite flood extent. Examples of the improvement in the accuracy of the flood mapping resulting from this process are presented in Section 4.

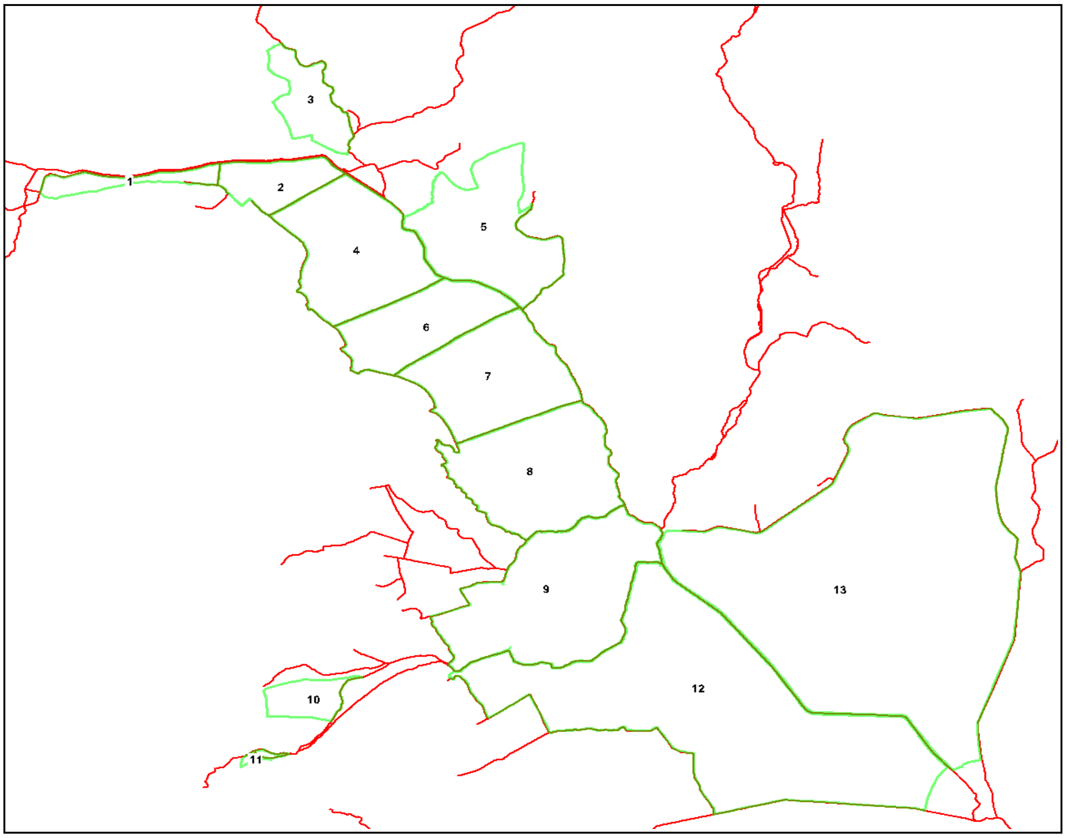

Figure 3.7: Extents of Simplified 2D Flow Path models with 2D simulation polygons highlighted in green (Numbers correspond to Table 3.1)

DOWNSTREAM BOUNDARY

3.50 The downstream boundary for the Lower Witham sub-model is at the mouth of the tidal River Haven near Tabs Head, to the south-east of Boston. Therefore, a design tidal boundary was applied as the downstream boundary of the model. This was based on a Mean High Water Spring tide for all runs with the exception of the climate change scenarios. The climate change tidal boundary was modified according to the latest DEFRA (October 2006) guidance to represent the 2115 scenario (shown in Figure 3.8 as presented in PPG25). Table 3.2 provides the peak levels for the MHWS spring tides for the present day and 2115.

Table B.1 Recommended contingency allowances for net sea level rise

Administrative Region	Net Sea Level Rise (mm/yr) Relative to 1990			
	1990 to 2025	2025 to 2055	2055 to 2085	2085 to 2115
East of England, East Midlands, London, SE England (south of Flamborough Head)	4.0	8.5	12.0	15.0
South West	3.5	8.0	11.5	14.5
NW England, NE England (north of Flamborough Head)	2.5	7.0	10.0	13.0

Notes:

1. For deriving sea levels up to 2025, the 4mm/yr, 3mm/yr and 2.5mm/yr rates (covering the three groups of administrative Regions respectively), should be applied back to the 1990 base sea level year. From 2026 to 2055, the increase in sea level in this period is derived by adding the number of years on from 2025 (to 2055), multiplied by the respective rate shown in the table. Subsequent time periods 2056-2085 and 2086-2115 are treated similarly.
2. Refer to Defra FCDPAG3 *Economic Appraisal Supplementary Note to Operating Authorities – Climate Change Impacts*, October 2006, for details of the derivation of this table. In particular, Annex A1 of this Note shows examples of how to calculate sea level rise.
3. Vertical movement of the land is incorporated in the table and does not need to be calculated separately.

Figure 3.8: Climate Change Guidance in Planning Policy Statement 25 Document

	MHWS-Present Day	MHWS-2115
Tabs Head Tidal Peak	3.860m AOD	5.001m AOD

Table 3.2: Tidal Boundary Peak Water Levels for the Haven in the Lower Witham Flood Map Improvements Model

3.51 In addition, inflows into to the Haven from the Black Sluice and Stonebridge catchments can have an impact on the water level immediately downstream of the Lower Witham outfall at Grand Sluice, with knock-on effects in terms of the rate of outflow through Grand Sluice affecting levels in the River Witham. The inflows from the Black Sluice model were obtained from the existing Black Sluice Strategy Study ISIS model from which the Black Sluice WiCSM model was originally developed. This was used as the South Forty Foot FMI model had not been completed prior to the Lower WItham FMI modelling. The Black Sluice Strategy Study ISIS model had results up to a 0.5% AEP event which were used in the Lower Witham FMI model. Additionally, it did not include the 5% AEP event, therefore the 4% AEP event was used as an appropriate substitute. The inflows from the Stonebridge catchment were taken from the Stonebridge Flood Map Improvements model[8]. Table 3.3 summarises the peak flows for each event for these two inflows The peaks for the Black Sluice do not necessarily follow a smooth increasing pattern with increasing return period due to ‘spikes’ in the peak flow caused when the Black Sluice pumps turn on and off at the start and end of each period of tide lock

AEP	Black Sluice Peak Inflow (m^3s^{-1})	Stonebridge Peak Inflow (m^3s^{-1})
50%	123.99	32.41
20%	119.33	38.76
10%	119.65	39.99
5%	4% Flow Used	41.91
4%	123.69	42.15
2%	126.96	43.75
1.30%	129.34	44.51

8 Faber Maunsell, (2008), Witham Flood Map Improvements Phase 2: Stonebridge Sub-Catchment Hydraulic Modelling Report

1%	131.72	45.33
0.50%	154.214	46.10
0.10%	0.5% Flow Used	47.06

Table 3.3: Peak inflows from the Black Sluice and Stonebridge models into the Lower Witham FMI model for various AEP events.

HYDRAULIC CONTROL STRUCTURES

3.52 Flow and levels in the Lower Witham catchment are influenced by a number of hydraulic control structures maintained and operated by the Environment Agency. Some of these are variable whilst others are fixed, and of the variable control structures, some are manually operated whilst others are automatically controlled. A number of the control structures in the Lower Witham FMI model were modified from those used in the calibration of the Lower Witham WiCSM parent model. These modifications were made either because the structure (or its current configuration) post-dates the calibration runs, the operating procedures have changed, or because it was necessary to use logical rules rather than observed manual settings to simulate a design condition. Details of the control mechanism for the major control structures in the Lower Witham FMI model are provided below. For structures not detailed below, it can be assumed that the operational settings from the Lower Witham WiCSM parent model were carried forward to FMI design runs.

Grand Sluice, Boston

3.53 Grand Sluice is the gravity outfall at the downstream end of the River Witham that allows discharge into the tidal River Haven when tidal conditions are favourable. Outflow is controlled by three vertical sluice gates that open clear of the water during flood conditions. During low flow conditions the gates seek to retain the water level within the Witham of +0.9m AOD during the winter season and levels of +1.4m AOD during the summer season. The gates are designed to operate automatically and adjust their opening based on remotely sensed water levels at Boston, Langrick and Bardney. In the Lower Witham WiCSM calibration model the control of the gates was undertaken using the 'remote water level' mode using levels at Bardney. This configuration was carried forward from the Lower Witham ISIS model and was found to produce satisfactory calibration results.

3.54 In the period between the completion of the Lower Witham WiCSM calibration work and the development of the Lower Witham FMI model, the Witham Flood Forecasting project used a copy of the WiCSM parent model to produce an ISIS flood forecasting model of the Witham catchment. As part of this work, the control of the Grand Sluice gates was updated to use logical control, utilising new data from the Swantel system to devise 28 logical control rules that sought to mimic the actual control rules used by the gates. The polling interval for the logical rules is 300 seconds. Whilst it was found that these new control rules behaved very similarly to the original control rules during a flood event, it was considered appropriate to update the Lower Witham FMI design model to include these latest updated rules. Following consultation with the Environment Agency, it was determined that the logical rules for the winter retained level would be used for the flood mapping design runs.

3.55 Similarly, the schematic representation of the tidal pointing doors at Grand Sluice was updated in the Lower Witham FMI model to match the latest Witham flood forecasting model. This involved replacing the Bernoulli loss units previously used to prevent backflow up the Witham with a flapped orifice unit of appropriate dimensions. Essentially, these two schematisations of the tidal pointing doors have the same effect, but the latter is considered to be current best practice.

Fiskerton Sluice

3.56 Fiskerton Sluice has 3 sluice gates which are designed to maintain the water level in the Lincoln to Bardney stretch of the River Witham between 2.92 and 3.0 mAOD. The model setup and logical rules devised for the Lower Witham WiCSM calibration model were carried forward to the Lower Witham FMI model as this produced satisfactory calibration results. The model representation includes three identical sluice gates which operate simultaneously based on the upstream water level. The logical rules open the gates as the upstream water level rises above 3.0m AOD, and close them if water levels drop below 2.92m AOD. Between 2.92m and 3.0m AOD the gate retains its previous position, so as to provide a 'dead band' to prevent hunting. The polling interval for the logical rules is 300 seconds.

Branston Island Sluice

The hydraulic structure used to fill Branston Island storage area is similar to Fiskerton Sluice in that it comprises of 3 sluice gates. As with Fiskerton sluice, the model setup and logical rules were carried forward from the Lower Witham WiCSM calibration model as this produced satisfactory calibration results. In the model, the three identical sluice gates operate simultaneously based on the upstream water level. The logical rules begin to open the gates when the upstream water level rises above 3.87m AOD. If water levels continue to rise, the three gates open incrementally, opening fully if the water level reaches 3.97m AOD. The polling interval for the logical rules is 300 seconds.

Branston Delph Overflow

3.57 An overflow culvert and penstock was recently constructed to allow overflow from Branston Delph to the Branston IDB system. The purpose of this is to allow the transfer of water from Car Dyke into the IDB system via Branston Delph, thereby relieving water levels on Car Dyke during a flood event. To enable this to occur, the penstock at the head of Branston Delph is opened simultaneously with the Branston Delph overflow penstock.

3.58 Details of the Branston Delph overflow penstock were provided by the Environment Agency with a view to including it in the Lower Witham FMI model. The overflow was represented in the model by a vertical sluice gate of appropriate dimensions and controlled by logical rules. Logical rules were also set up for the penstock controlling flow from Car Dyke into Branston Delph, such that it opens to attempt to limit the level to between 4.0 and 4.1m AOD upstream. Once the flow into Branston Delph raises the water levels in the Delph above 2m AOD, the logical rules on the overflow culvert begin to open the overflow penstock; the rules are set up to attempt to keep the level on Branston Delph below 2.2m AOD.

3.59 Following discussions with the Environment Agency, it was agreed that the Branston Delph penstock would be set to stay open at higher order events even if this overwhelms the IDB pumping station. This is a precautionary approach for flood mapping purposes.

Siphons and Penstocks

3.60 There are numerous siphons in the Lower Witham catchment that allow flow to be conveyed in low level drainage channels beneath embanked main river watercourses. Some of these are maintained and operated by the Environment Agency, but the majority are maintained and operated by the IDBs. In most cases, the purpose of the IDB siphons is to allow runoff to drain to the pumping stations from areas that would otherwise be hydraulically disconnected.

3.61 Most of the siphons in the Lower Witham have penstocks that allow flow through the siphon to be controlled. For example, it may be necessary to close the penstock to limit the extent of flooding occurring on one side of the siphon. Similarly, penstocks are also common on open channel watercourses that act as connections between IDB drainage sub-catchments. These penstocks are usually kept closed but may be opened in an emergency if the pumps in one sub-catchment fail and it is necessary to divert flow to the pumping station in an adjacent sub-catchment.

3.62 The Environment Agency provided GIS layers showing the location of siphons and penstocks in the Lower Witham catchment. AECOM determined that some of these were already represented in the Lower Witham FMI model whilst other were not. Following discussions with the Environment Agency, it was agreed that for the purpose of the design flood mapping it could be reasonably assumed that the majority of connecting penstocks would be closed by IDB or Environment Agency operations staff during a flood event. Where siphons/penstocks had not previously been added to the model, this meant that it was not necessary to model them. However, in the case of IDB penstocks connecting adjacent IDB sub-catchments it was agreed that floodplain sections should be included in the model to allow flow to pass between the sub-catchments in the event of flooding based on the natural topography.

3.63 There were a small number of exceptions to the general approach stated above, and the following configurations were agreed with the Environment Agency for the design condition:

- The siphon beneath Stainfield Beck (NGR 509727,372978) was not present on any of the GIS layers provided but was left in the Lower Witham FMI model for design runs;
- The penstock on the open channel connection at Shortferry Road (NGR 508590,371980) was modelled as permanently open for the design condition;
- The two Environment Agency penstocks conveying flow beneath the River Bain at Coningsby (See 3.37) do not have penstocks that allow them to be closed off and were therefore left in the main 1D Lower Witham FMI model, and the lower Bain 1D-2D linked sub-model.

DESIGN RUNS

3.64 Once finalised, the main Lower Witham FMI defended 1D model was run in combination with the appropriate design hydrology and downstream boundary for each return period to produce the required series of flood outlines.

3.65 A total of 24 design runs were undertaken for derivation of the defended flood mapping: the full suite of 12 return periods using the design hydrological boundaries critical for the upper parts of the tributaries, and a further set of 12 return periods using the design hydrological boundaries critical for the main Witham/lower tributaries.

Model Run Parameters

3.66 The InfoWorks RS model was run over a 312 hour period (13 days) which was sufficiently long to ensure that all river sections and storage areas had experienced their peak stage, thereby producing the maximum flood map extent. This was extended to a 744 hour period (31 days) for all events greater than the 1% AEP event (i.e. the 0.5% AEP event, 0.1% AEP event and both climate change scenarios).

3.67 The simulations were carried out on a desktop PC using a 2.13GHz Intel dual core processor with 2 GB of RAM. The default InfoWorks RS run parameters were used with the following exceptions:

- Adaptive timestepping, initial timestep = 10 seconds, min = 10 seconds, max = 300 seconds;
- Save interval = 900 seconds (Default is 300)
- Maximum iterations set to 20 (Default is 6)
- Flow convergence parameter (qtol) set to 0.05 (Default is 0.01)
- Level convergence parameter (htol) set to 0.02 (Default is 0.01)
- Spill threshold set to 0.005m (Default is 0.000001m)

3.68 Of these non-default parameters, it was the flow and level convergence parameter changes that produced the most benefit in terms of improving convergence and reducing run times to a reasonable level. These changed parameters slacken the tolerance of the model calculations slightly so that flow and level calculations do not have to be solved to as high a degree of accuracy before convergence is reached and the calculations move on to the next timestep. The revised values used in the model were those applied as standard practice for flood forecasting models. In theory this change has the potential to alter the modelling results slightly. In order to check that this change was not having an adverse effect on the flood mapping produced by the model, sensitivity tests were undertaken to compare flood mapping produced using the default parameters against those with the raised flow and level convergence parameters. It was found that for the 1% AEP event, in-channel maximum water levels varied by an average of just 1mm across all nodes, with a maximum increase of 0.034m when using the default parameters. For storage areas which mapped flooding, the average change in maximum water levels was 3mm with a maximum increase of 0.041m when using the default parameters. Most importantly, there was negligible visual difference in the mapped outlines in all cases.

3.69 Using the above run parameters, model simulation run times averaged approximately 12 hours for the 312 hour simulations (for events less than or equal to the 1% AEP), and between 29 and 180 hours for the 744 hour simulations (for events greater than the 1% AEP).

3.70 Aside from the increase in the simulation period, there was a general increase in model run time associated with increasing return period. This was mainly caused by periods of non-convergence of the model run as more of the floodplain representations were utilised by flow within the model. Such convergence issues are commonly experienced in complex floodplain modelling such as this. Efforts were made to minimise the non-convergence with the help of the diagnostic log, however, it was not possible to eradicate the non-convergence completely. In light of the remaining non-convergence, the results of the modelling were examined carefully to check for any obvious errors in the calculation of flows and water levels. Based on these checks it was considered that the model results were sufficiently accurate and realistic to be used for the production of flood mapping.

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Flood Mapping

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4 Flood Mapping

INTRODUCTION

4.1 The main outputs from this study are flood extent outlines for a range of return period design flood events. The flood extents have been derived for the defended scenario by combining hydrological models with the Lower Witham FMI hydraulic model, the development of which has been described in the previous sections.

4.2 InfoWorks RS is capable of producing the flood extent outlines by combining the hydraulic model results with a flood mapping model and a Digital Terrain Model (DTM). This integrated approach saves time compared to extrapolating river levels across a floodplain surface manually, especially if the models need to be re-run several times to iron out issues identified in the initial drafts of the flood mapping.

4.3 Despite the integrated production of flood extents offered by the InfoWorks RS software, it is still necessary to undertake some manual editing of the output extents. Such manual editing addresses issues such as flow path definition, LiDAR anomalies and 'dry islands'. Details of the approach to manual editing that has been undertaken on the Lower Witham FMI flood extents is given below.

4.4 It should be noted that the flood mapping produced by this study downstream of Coningsby is considered to be of greater accuracy than that which was produced by the Bain FMI model. This is because the Lower Witham FMI model includes the downstream reach of the River Bain, allowing the simulation of the interaction of flows and water levels in the channels and floodplains between the two watercourses. In addition, a linked 1D-2D sub-model of the Lower Bain was developed to more accurately map the flow paths as part of the Lower Witham FMI study.

INFOWORKS RS FLOOD MAPPING SET-UP

4.5 InfoWorks RS produces flood extents by applying the water surface calculated by the hydraulic model (from water levels at any given time of a model simulation) across the DTM. This requires the definition of a flood mapping model in InfoWorks RS. The flood mapping model is built up by specification of flood mapping objects within the same network as the hydraulic model.

4.6 Full details of how the flood mapping model is set-up was provided in the Bain FMI modelling report[1].

FLOOD EXTENT GENERATION

4.7 The flood extent is derived from the model simulation results by applying the calculated water surface against the ground model levels. A surface is then produced which outlines the boundary where the calculated water level and ground level are equal. 'In-filling' occurs where the calculated water level is above the ground level. This process is done only within the extents of the flood mapping compartments. InfoWorks RS can generate contours showing different depths within the flooded areas.

4.8 When developing a flood mapping model it is good practice to visually examine the flood mapping output to ensure the hydraulic/mapping model has sufficient coverage to contain the extents of the flooding. If the 'in-filled' flood extent is shown to coincide with the edge of the flood mapping compartment this usually indicates that flooding could extend further than is being shown. In such instances it is necessary to expand the hydraulic model and flood mapping compartments to ensure the flood extent outline is contained.

4.9 Once the flood extent has been finalised, it can be exported from InfoWorks RS to MapInfo (or other GIS software) for editing and formatting. It is usual to export a single contour at the maximum depth of flooding. This exports as a series of unformatted polygons into MapInfo.

MANUAL EDITING OF FLOOD EXTENTS

4.10 Following the export of the raw flood extents to MapInfo, a process of manual checking and editing was undertaken to produce the final flood outlines and ensure these were accurate. The first task was to merge the flood mapping for the 20.5hr duration (critical for the tributaries) with the flood mapping for the 48.5hr duration (critical for the lower reaches).

4.11 The merged flood extent polygons were cleaned in MapInfo to remove any isolated polygons and holes <250m^2. Each cleaned polygon extent was then trimmed to the NFCDD defence polyline dataset to ensure that the mapped extent did not encroach into the channel where there is a formal defence. After the cleaning process was complete the outlines were manually checked and edited. The IWRS model results were used to verify the accuracy of the flood extents and to define any flow paths.

Derivation of flow paths

4.12 One of the main purposes of the manual editing process was to add appropriate flow paths to the 1D flood extents generated from the main Lower Witham FMI model. These flow paths were defined either by 2D flow modelling (as described in Section 3.36), or by manual definition based on the topography.

4.13 In the main Lower Witham FMI 1D model, Infoworks RS maps the modelled water levels in each 1D floodplain storage area by filling it from the lowest elevation. This mapping methodology gives no indication of where the flow generating the mapped extents has originated from. Where areas of flooding appeared to have little connectivity to the channel, the surrounding spill units were checked to establish the source of the flow into the storage area. A decision was then taken on whether it would be most appropriate to generate flow paths using 2D flow modelling, or whether manual definition would be adequate.

4.14 2D flow modelling was considered to be most appropriate in areas such as the lowland Fens, where the topography is flat and overtopping flows tended to dissipate into the IDB drainage network and/or the flooding ponded a significant distance from the point of overtopping. In these situations the manual definition of flow paths was considered to be too inaccurate and subjective. Manual definition of flow paths was considered to be adequate where there was a short distance between the point of overtopping and the ponded area, with an obvious flow route based on the DTM.

4.15 Figures 5.1 and 5.2 show examples of the benefit of the derivation of flow paths in the lowland Fens using 2D modelling. The flow paths generated by 2D modelling were superimposed on the base 1D mapping to produce the final flood extents.

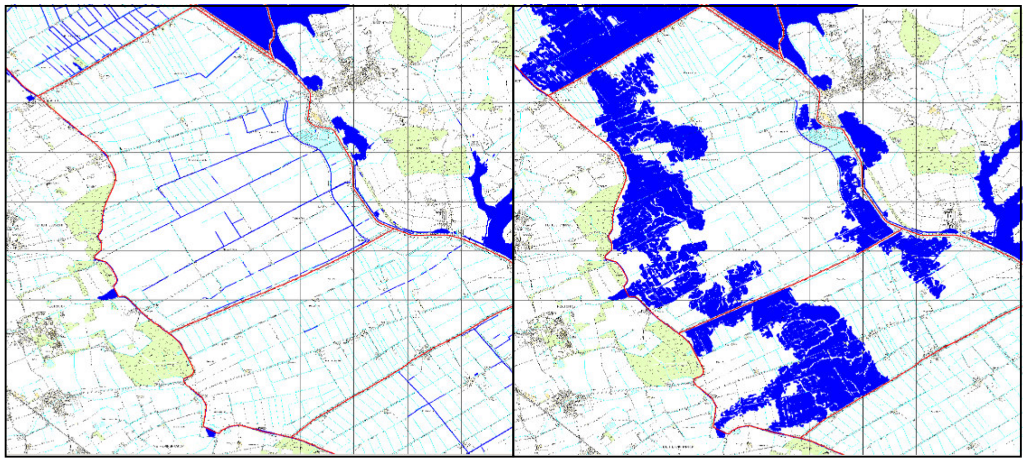

Figure 5.1: Branston Fen to Metheringham Fen (2D Area No.s 4 & 6). 1% AEP Flood Extents showing 1D output (left) vs. Final mapping with 2D flow paths added (right).

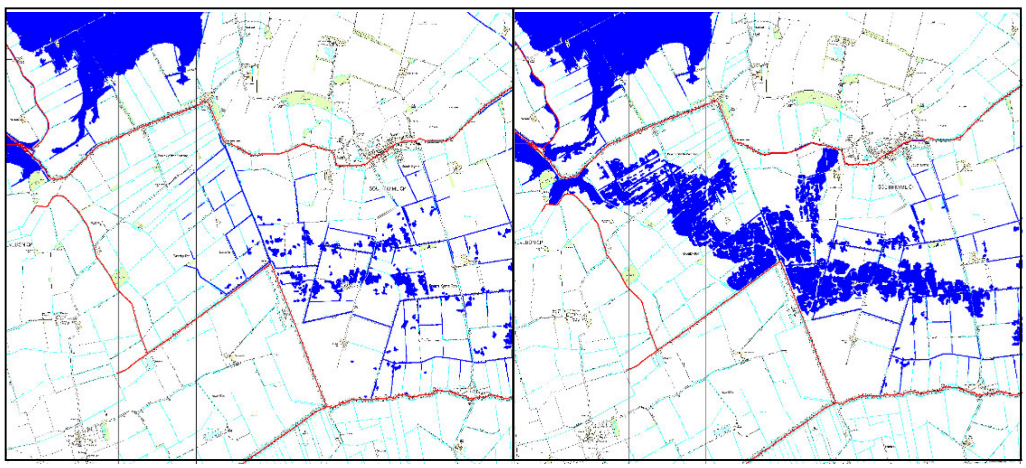

Figure 5.2: North Kyme Fen & Damford Grounds (2D Area No. 9). 2% AEP Flood Extents showing 1D output (left) vs. Final mapping with 2D flow paths added (right).

4.16 Figure 5.3 shows an example of the manual addition of a flow path. In this example the spills indicated overtopping some distance from the ponded flooding and a flow path was manually added in to connect the flooding to the channel. This illustrates the usefulness of the 3D view functionality in Infoworks RS when determining likely flow paths.

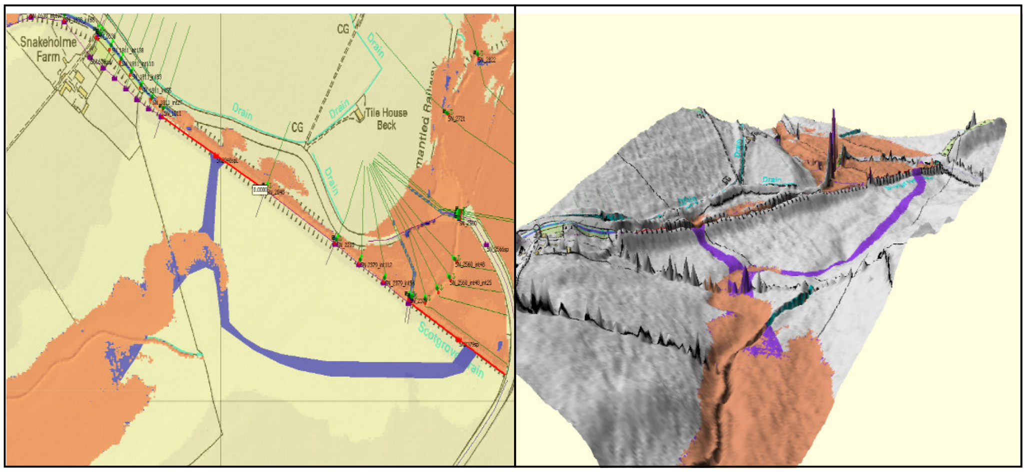

Figure 5.3: Spill units (highlighted in red) indicate overtopping on the right bank but the raw IWRS outline (orange) does not show any flow path. Flow path manually added using the DTM to ensure flow path followed the floodplain valley floor, the purple outline is the modified flood extent

Editing due to LiDAR anomalies

4.17 In several locations within the Lower Witham catchment the LiDAR filtering process had failed to remove large buildings. These were left as high mounds in the DTM and appeared as dry islands in the flood mapping. Where such anomalies were identified the buildings were filled in manually for the final flood map. Other anomalies in the DTM were also identified and manually corrected: The example in Figure 5.4 represents the tile boundary between two LiDAR tiles. This boundary caused a discontinuity in the DTM leading to a distinct edge in the flood extent; where possible the outlines were smoothed to minimise the edge. Such discontinuities are also possible at the LiDAR/SAR interface.

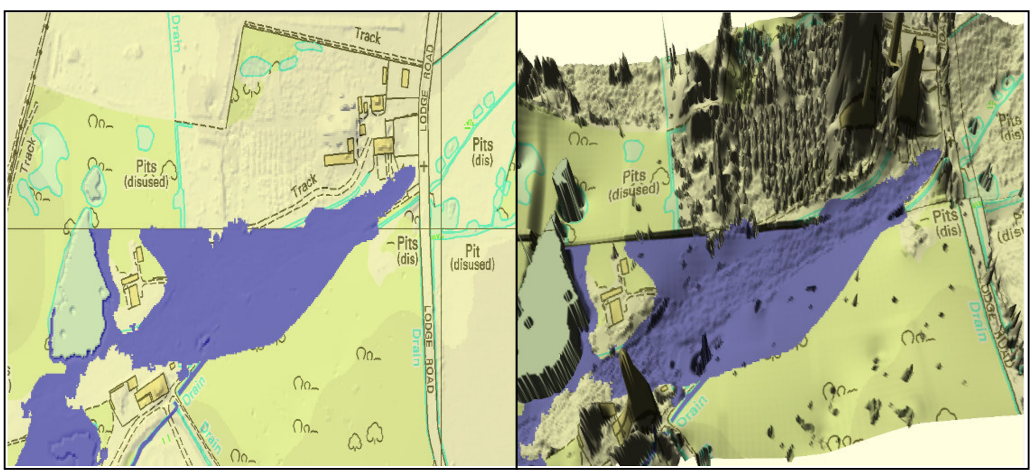

Figure 5.4: DTM anomaly caused by the boundary between two LiDAR tiles resulted in an edge in the modelled outlines in this area.

Removal of IDB Drains

4.18 At the request of the Environment Agency, IDB drains shown as inundated within the flood extents were removed. This was done within the MapInfo GIS software. The technique for removal was semi-automated whereby drains were removed manually from the 0.1%+CC flood extent which was then subsequently used as a mask for the other AEP flood extents. A final check and manual clean up was also required for each AEP flood extent. In some areas the inundated drain was attached to an area of flooding and was the principal flow path and in these instances the drain was left in the flood extents.

Other Manual Edits

4.19 At the request of the Environment Agency, the river channel between defences was removed from the final flood extents. Where no formal defences are present the channel was included in the mapped extents. Where formal defences exist on only one bank, the channel was left in the mapped extents, but with a gap between the channel and the defence to the defended side.

4.20 Any other minor errors in the flood mapping that were picked up during the QA checking were manually amended in production of the final outlines as appropriate. Further details are provided in the 'Known Issues' section below.

QUALITY ASSURANCE CHECKING

4.21 Quality Assurance checking of the final flood outlines was undertaken, primarily to check for the following:

- Errors in the flood mapping model which cause inappropriate levels to be mapped;
- Ensuring that flood extents become sequentially more extensive with increasing flood event magnitude;
- Flow paths which do not fall along the lowest elevations when compared to the DTM.

4.22 The processed flood outlines were also reviewed by the Environment Agency staff at draft stage to check that they match their perceptions of the areas flooded in historical events of known return periods. This checking process resulted in amendments to some spill unit levels in the model where it was considered that the draft model included an unrealistically low low-spot. Where this was done a comment was inserted into the model.

4.23 When checking the mapped flood extent for a given design event, it should be borne in mind that due to the hydraulic coefficients and control structure settings adopted for the 'design' scenario, it is possible that flooding in historical events may have been more extensive than that shown for the same approximate return period in the flood mapping produced by this study. It is also important to remember that some historical flood extents are exacerbated by the incorrect operation or failure of control structures, and by blockages neither of which are represented in the design hydraulic model.

4.24 The finalised defended flood outlines for the defended flood mapping were converted from MapInfo .TAB format into ArcView shapefile format and delivered to the Environment Agency in electronic format.

KNOWN ISSUES WITH THE FLOOD MAPPING

4.25 A number of issues with the flood mapping produced by this study were identified, some of which were dealt with by additional manual editing of the flood mapping output, whilst others offer opportunities for further development of the model in the future.

Washingborough Roadside North

4.26 Following review of the draft flood mapping outputs by the Environment Agency, the flood mapping at Washingborough Roadside North was specifically queried. The upstream end of the main river is culverted beneath Fen Road, and there have been recent flood events affecting properties along the culverted section close to the confluence of Fen Road and Keeble Drive (NGR 502969, 370595). The topographic survey undertaken for the model did not include internal CCTV survey of culverts and therefore the data only commences at the outlet into the open channel. It was also acknowledged by the Environment Agency that the flooding issues at this location are potentially complex with possible surface water contributions and siltation of the culvert. It was therefore agreed that it is beyond the scope of the current model/study to replicate the flooding in this area, and that this issue would be best investigated using a detailed localised model, possibly using Infoworks CS software.

Langworth Gauging Station

4.27 The Environment Agency's review of the draft flood extents also highlighted potential concerns with the accuracy of water levels and flood mapping in the vicinity of Langworth Gauging Station. Both in the model and in reality, flow overtops both the left and right banks immediately upstream of the gauging station and floods the adjacent land. The flood extent produced by the model on the left bank was consistent with the observed pattern of flooding from recent events. The flooding on the right bank was less straightforward to map as it consists of a shallow flow path travelling south towards Manor Farm. Initially this flow path was manually derived based on the DTM and the observed flooding pattern from the July 2007 event. A simplified 2D flow path model was also produced for comparison. It was noted that the 2D flow path was less extensive and this lead to investigations into the design water level produced by the model at the gauging station for various return periods.

4.28 It was found that the model appeared to underestimate the water level upstream of Langworth gauging station despite the design flows being consistent with the historical flow record and the selected design flood frequency curve. This was found to be attributable to the schematisation of the left bank floodplain, which was modelled as extended cross-sections. This schematisation is able to produce the expected flood extent in the left bank floodplain, but underestimates the water level within the channel as it allows water to be conveyed in the floodplain prior to overtopping of the left bank. As this issue was highlighted after production of the final draft of the flood extents, it was agreed that the model would not be amended and re-run to improve the results, as this would be costly and time consuming. Instead the right bank flood extents were taken as the original 1D extents with manual editing of flow paths based on recent flood events.

4.29 As a result of the issue highlighted at Langworth Gauging Station, the water levels between Langworth Bridge and the gauging station as tabulated in Appendix C are underestimated by up to 300mm. This will also affect the calculated AEP/SoP in Appendix F. The reported peak flows and the mapped extents in this area are considered to be accurate. It is intended that the schematisation of the model will be updated to address this issue as part of the next project to utilise the model. This is likely to be the Lower Witham Strategy Review modelling.

Kyme Eau at Chapel Hill

4.30 The Environment Agency also queried the mapping from flows overtopping the left bank of Kyme Eau in the model close to Chapel Hill IDB pumping station at low return periods. The low spot which overtops in the model is slightly lower than the minimum level quoted in the as-built drawings from bank works carried out in 2006. However, following investigations it was found that this as-built minimum level is also lower than the latest 1 in 2 year design water level. There is an outstanding query as to why the standard of protection at this location is so low despite recent bank works, and this will be picked up in the Lower Witham Strategy Review project. Despite this, it was agreed with the Environment Agency that the mapped overtopping of the left bank at low return periods is appropriate and that the associated flood extents would be retained in the final deliverables.

4.31 A GIS layer detailing all known issues with the flood mapping and locations of manual edits was provided along with the electronic versions of the finalised flood outlines. This layer provides an auditable record of the decision processes in undertaking manual edits to the raw flood mapping output.

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Sensitivity Analysis

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5 Sensitivity Analysis

INTRODUCTION

5.1 As part of the flood map improvements work on the Lower Witham catchment, it was necessary to investigate the sensitivity to the seasonal changes in channel and floodplain roughness. As the model was built to represent a typical winter scenario, the analysis involved increasing the roughness values by a specified percentage so that they reflect the increased channel and floodplain vegetation of the summer season. Seasonal changes to the operation of Grand Sluice gravity outfall, as specified by the Environment Agency, were also included.

5.2 The summer sensitivity analysis was undertaken using the 50%, 10%, 4%, 2% &1% AEP design events as set out in the project proposal. As with the standard scenario design event runs, it was necessary to run the 20.5 hour and 48.5 hour storm durations in order to ensure the appropriate peak water levels were obtained at each model node.

MODEL SET-UP

5.3 A copy of the defended base model was made. This model was modified to represent the summer scenario, from which the sensitivity runs could be made.

5.4 The first modification was to the rules at the Grand Sluice outfall near Boston. The logical rules applied at Grand Sluice are set up to open the gates based on levels sensed at Boston, Langrick Bridge and Bardney. The objective of the gates is to maintain retained water levels of 0.9m AOD during the winter season and levels of 1.4m AOD during the summer season. The rules were modified to reflect the summer season scenario.

5.5 The second modification was to the channel and floodplain roughness throughout the model. A global change was applied which increased all the Manning's 'n' roughness values associated with river sections by 20%. This reflects the increased roughness that would be associated with increased vegetation growth during the summer months.

SIMULATIONS AND RESULTS

5.6 No issues arose in running the models with the modified conditions, and so a complete set of results for the required design events was obtained.

5.7 The peak water levels at each node were extracted for each return period design event (using the maximum of the results from the two durations that were run). These results are tabulated in Appendix E.

5.8 The impact of the seasonal change of roughness was analysed by comparing the peak water levels from the sensitivity runs against those from the base case. Table 6.1 summarises the maximum and average change in peak water levels for each modelled watercourse within the Lower Witham catchment (for the 10% and 1% AEP design events only).

5.9 The average change takes account of both increases and decreases in the peak water levels between the sensitivity runs and the equivalent design run undertaken for the base case scenario. Small decreases in peak water levels are counter-intuitive for an increase in roughness, but these can occur at some nodes due to redistribution of flows within the model, or more simply due to rounding errors in the model computations.

Watercourse	10% AEP event		1% AEP event	
	Max. diff. (m)	Average diff. (m)	Max diff. (m)	Average diff. (m)
Anwick Fen Drain	0.75	0.63	0.03	0.01
River Bain	0.17	0.07	0.17	0.07
Branston Delph	0.14	0.11	0.03	0.00
Barlings Eau	0.13	0.06	0.09	0.04
Billinghay Skirth	0.01	0.00	0.02	0.00
Car Dyke	0.09	0.03	0.14	0.01
Dorrington Catchwater	0.03	0.01	0.03	0.02
Duckpool Drain	0.13	0.05	0.14	0.04
Dorrington Dyke	0.01	0.01	0.02	0.01
Digby Beck	0.09	0.06	0.10	0.07
Farroway Drain	0.17	0.16	0.01	0.01
Grand Sluice/Haven	0.01	0.00	0.01	0.00
Heighington Catchwater	0.05	0.04	0.05	0.02
Heighington Beck	0.15	0.06	0.11	0.03
Kyme Eau	0.03	0.01	0.02	0.00
Leasingham Drain	0.10	0.07	0.11	0.07
Lower River Witham – Lincoln to Bardney	0.01	0.00	-0.01	-0.02
Lower River Witham – Bardney to Chapel Hill	0.03	0.02	0.02	0.00
Lower River Witham –Chapel Hill to Boston	0.09	0.06	0.11	0.06
Marsh Drain	0.03	0.03	0.01	0.01
Metheringham Delph	0.02	0.00	0.00	0.00
New Cut	0.10	0.06	0.10	0.05
Nine Foot River	0.02	0.02	0.01	0.01
Nocton Delph	0.10	0.08	0.14	0.11
North Hills Drain	0.02	0.01	0.02	0.02
Old River Slea	0.10	0.04	0.11	0.05
Old River Witham	0.03	0.03	0.03	0.02
Queen Dyke	0.06	0.02	0.08	0.01
Ruskington Catchwater	0.16	0.06	0.06	0.04
River Slea	0.13	0.06	0.16	0.06
Ruskington Beck	0.12	0.05	0.11	0.04
Sandhill Beck	0.06	0.02	0.07	0.00
Stainfield Beck	0.20	0.09	0.27	0.07
Snakeholme Drain	0.16	0.07	0.17	0.04
South Delph	0.10	0.06	0.10	0.06
Timberland Delph	0.02	0.01	0.00	0.00
Welton Beck	0.13	0.02	0.09	0.05

Table 6.1: Maximum and average change in peak water levels for each watercourse in the Lower Witham catchment, resulting from seasonal sensitivity analysis

5.10 In summary, the seasonal sensitivity of Manning's n for the summer scenario shows the following:

- A rise in the average maximum peak water level for all watercourses except Billinghay Skirth and Timberland Delph for the 10% and 1% AEP events
- A maximum increase of 110mm on the River Witham for the 1% AEP event; with a maximum increase of 90mm for the 10% AEP event
- A maximum increase of 270mm on the tributaries at Stainfield Beck for the 1% AEP event; with a maximum increase of 200mm for the 10% AEP event
- A typical average increase of less than 120mm for all watercourses at the two AEP events in Table 6.1

5.11 The peak water levels at each model node for each modelled return period are tabulated in Appendix E.

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Standard of Protection Analysis

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6 Standard of Protection Analysis

6.1 A key requirement of Phase 2 of the Witham Catchment Flood Map Improvements project was to determine the indicative and actual Standard of Protection (SoP) for each NFCDD asset within the Witham catchment. The indicative standard was set in consultation with the Environment Agency, being appropriate to the land use adjacent to the modelled reaches of the study. The first stage of the analysis was to calculate the actual Annual Exceedance Probability (AEP) at all nodes within the modelled reaches. The second stage involved calculating the freeboard allowance and hence effective height at each node, which in turn determined the actual SoP. This provided an accurate assessment of the current defence levels in comparison to the indicative SoP levels.

INDICATIVE STANDARD OF PROTECTION

6.2 The indicative SoPs were decided upon following guidance specified in section 6 of the Flood and Coastal Defence Project Appraisal Guidance: Economic Appraisal Document (PAG3). The relevant table from this document is reproduced in Table 6.1

Land Use Band	Indicative Range of Housing Units (or Equivalent) per km of Single River Bank	Description	Fluvial Return Period (SoP %)	Annual Probability of Failure
A	≥50	Intensive developed urban at risk from flooding.	50-200 (2-0.5%)	0.005-0.02
B	≥25 to<50	Less intensive urban areas with some high-grade agricultural land and/or environmental assets of international importance requiring protection.	25-100 (4-1%)	0.01-0.04
C	≥5 to <25	Large areas of high-grade agricultural land and/or environmental assets of national significance requiring protection with some properties at risk.	5-50 (20-2%)	0.02-0.2
D	≥1.25 to<5	Mixed agricultural land with occasional, often agriculturally related, properties at risk. Agricultural land may be prone to flooding. May also apply to environmental assets of local significance.	1.25-10 (80-10%)	0.1-0.8
E	>0 to <1.25	Low-grade agricultural land, often grass, at risk from flooding with isolated agricultural or seasonally occupied properties at risk, or environmental assets at little risk from frequent inundation.	<2.5 (40%)	>0.4

Table 6.1: Summary of Indicative SoP from the Flood and Coastal Defence Project Appraisal Guidance: Economic Appraisal Document

6.3 The land use band set for the Lower Witham catchment ranged from land use band A to land use band C. This was confirmed by the Environment Agency. Based on the chosen land use band, the indicative SoPs for the Lower Witham catchment were used in the freeboard analysis (see below).

CURRENT BANK TOP ANNUAL EXCEEDANCE PROBABILITY

6.4 In order to assess the actual AEP at which the existing banks overtop on the modelled watercourses within the Lower Witham catchment, the results of the defended model runs were required. As stated previously, model runs were completed for the 50%, 20%, 10%, 5%, 4%, 2%, 1.33%, 1%, 0.5% and 0.1% as well as the 1% and 0.1% AEP climate change events. The modelled peak water levels were extracted for all model runs, with the maximum between the two durations being carried forward to the Freeboard and SoP analysis. The bed level, left and right banks were also extracted from the InfoWorks RS model network.

6.5 The extracted model results were compared against the bank levels to establish the actual AEP at each river section on all modelled watercourses. The results from this analysis are tabulated in Appendix F.

FREEBOARD ANALYSIS

6.6 As part of the SoP analysis it was necessary to assess the required freeboard for the existing defence assets within the Lower Witham catchment. Freeboard is defined as "an allowance to take account of the physical processes that affect the defence level and the uncertainty in the prediction of physical processes that affect the defence level"[9].

6.7 In order to determine the required freeboard the following methodology, originally developed for the Stonebridge catchment and based on guidance from the Fluvial Freeboard Guidance Note[9] was applied to the Lower Witham catchment. Firstly the Lower Witham defences within the National Fluvial and Coastal Defence Database (NFCDD) were identified, and the freeboard for these defences was calculated. Once the freeboard allowance was determined for the defences it was possible to apply the measure of freeboard to the river sections within the length of each defence. This enabled a detailed freeboard assessment of the asset whilst also identifying areas of low bank height as shown in the survey data.

Freeboard Analysis

6.8 The freeboard allowance was determined in two stages following the Fluvial Freeboard Guidance Note[9]. This involves the assessment of first the physical processes and secondly the uncertainty.

6.9 The first stage looks at the physical processes to determine the freeboard allowance. This requires the use of section 4 in the Fluvial Freeboard Guidance Note to ascertain the freeboard allowance of a particular defence based on physical process such as:

- Wave Overtopping;
- Settlement;
- Degradation;
- The act of cracking and vermin;
- Super-elevation at bends;
- Boatwash;
- Wind Set up;
- Sedimentation.

6.10 Note that with regard to the allowance for wave overtopping, the calculation undertaken was dependent on the fetch (i.e. distance from bank top/defence to defence perpendicular to flow channel). If the fetch was greater than 50m, the calculation associated with the Floods and Reservoir Safety guide[10] was applied rather than the standard relationship described in the Fluvial Freeboard Guidance Note[9]. In both cases, the calculated wave surcharge allowance was rounded to the nearest 0.05m. This is explicitly stated as a requirement for all values less than 0.125m (i.e. those calculated by the standard relationship), but was applied to those values calculated from the method in the Floods and Reservoir Safety guide for consistency.

[9] Fluvial Freeboard Guidance Note: Technical Report W187, Environment Agency, 2000.
[10] Floods and Reservoir Safety guide, 1996, Institute of Civil Engineers, Thomas Telford Ltd.

6.11 Each process was assessed for each defence type and the sum of all the allowances was used as a freeboard allowance for physical processes. For all other physical processes, no allowance has been made for their impact. This is the default approach as described by the advice notes in the Fluvial Freeboard Guidance Note[9]. These assumptions are not unreasonable:

- the defences are likely to have been in place long enough for any settlement to have taken place.
- maintenance is assumed to render the impacts of degradation and cracking negligible.
- the majority of the watercourses are not navigable, so boatwash is not an issue.

The assumptions made are restated in an example pro-forma, which is presented in Appendix F.

6.12 Stage two requires the identification and assessments of the uncertainties relating to the:

- Accuracy of hydrological data;
- Accuracy of hydrological analysis;
- Accuracy of hydraulic data;
- Accuracy of a hydraulic model;
- Significance of physical parameters;
- Consequences of failure.

6.13 The quick method presented in section 5 of the Fluvial Freeboard Guidance Note[8] was used to calculate an uncertainty score based on scores from a number of criteria (total score out of 30). This is then used in conjunction with simulated water levels to provide a freeboard allowance to account for uncertainties. The water levels used were based on the indicative SoP. The indicative SoP is based on the Flood and Coastal Defence Project Appraisal Guidance: Economic Appraisal Document[11] and assigns a land use band to each area for which there is a range of indicative standard of protection to be provided. For example, land use band A has a standard of protection between 2 – 0.5% AEP. The uncertainty freeboard allowance analysis was completed for a high SoP (eg, 0.5%) which is considered the most precautionary.

6.14 Once both stages are complete, the freeboard allowance for physical processes and for uncertainties are summed to give a total freeboard allowance for each defence asset.

6.15 Following discussions with the Environment Agency regarding the freeboard method and presentation of results, a few refinements to the methodology were made in comparison to the results presented previously for the Stonebridge and River Bain catchments. Previously, a freeboard value was calculated for defended sections only, with this freeboard value accounting for both physical processes and uncertainty. In the revised method, a freeboard value is also calculated for undefended sections, and with a slightly different methodology as described below.

6.16 For the undefended sections, the contribution of the physical processes was assumed to be negligible. This is because physical processes such as settlement, degradation and wave overtopping will not be important. Therefore only the uncertainty measure is used in the freeboard calculation.

6.17 In the calculation of the uncertainty measure in the freeboard calculation for undefended nodes, the multi-attribute test described above will have the same scores for 5 of the 6 categories (tables 5.1-5.4 and 5.6 in Technical Report W187[9]). However, for the 'significance of physical parameters' element of the multi-attribute test (table 5.5 in Technical Report W187[9]), a score of 1 is assumed at undefended nodes which assumes that uncertainties in physical parameters are negligible.

6.18 The actual SoP provided by the existing defences within the Lower Witham catchment was also assessed. This was calculated by deducting the freeboard allowance from the defence level to produce an effective height (see Figure 6.1).

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[11] Flood and Coastal Defence Project Appraisal Guidance: Economic Appraisal Document, Ministry of Agriculture, Fisheries and Food, 2000, 104pp.

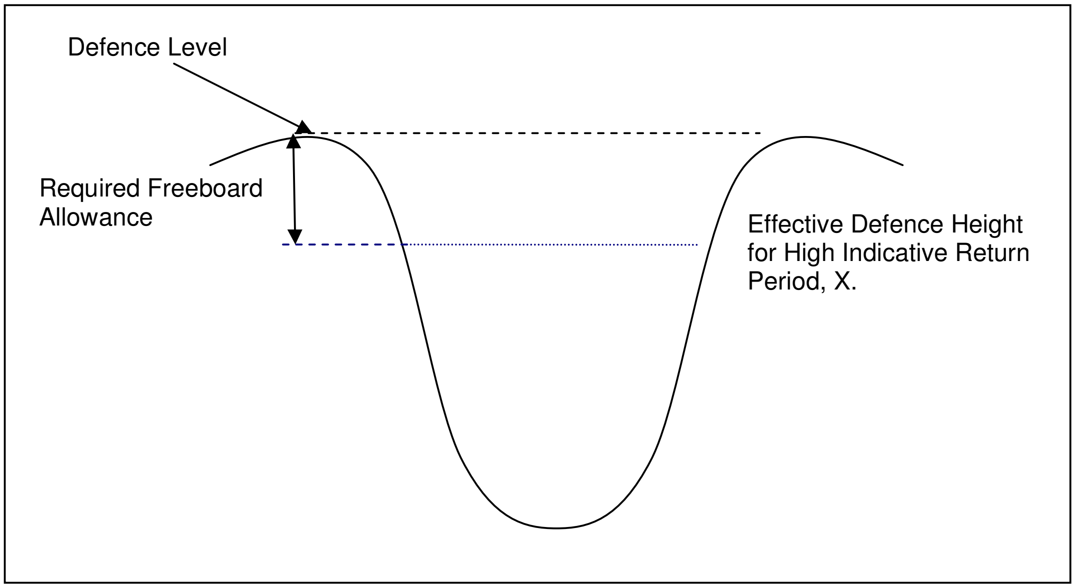

Figure 6.1: Standard of Protection and Required Freeboard

6.19 This effective height was then compared to all of the simulated water levels to determine the actual standard of protection for each node within the Lower Witham flood map improvements model. The actual SoPs are provided in Appendix F.

Conclusions

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7 Conclusions

SUMMARY

7.1 In order to produce the required outputs for the Lower Witham catchment, under Phase 2 of the Witham Catchment Flood Map Improvements commission, an InfoWorks RS hydraulic model was developed. This hydraulic model, referred to as the Lower Witham FMI model, was based upon the InfoWorks RS parent model developed and calibrated for the Lower Witham under the Witham Catchment Strategic Model commission.

7.2 The Lower Witham FMI 'with defences' model was refined and developed to include representations of all floodplains required to contain events up to and including the 0.1% AEP event with an allowance for climate change. An integrated flood mapping model was set up in InfoWorks RS to enable the software to map the model results.

7.3 Hydrological boundaries were developed using FEH methodologies for a suite of return periods: 50%, 20%, 10%, 5%, 4%, 2%, 1.33%, 1%, 0.5% and 0.1% AEP events, plus the 1% and 0.1% AEP events representing climate change (for 2115) based on current DEFRA guidance.

7.4 Two storm duration events (20.5 hours and 48.5 hours) were found to be adequate to represent the required peak flows and volumes at all points in the catchment following scaling of the rainfall-runoff boundaries to the peak flow estimates. The shorter duration reflected the faster and more peak dependent nature of the upper parts of the tributaries in the Lower Witham catchment, whilst the longer duration reflected the slower, volume dependent nature of the River Witham itself and the lower parts of the tributaries.

7.5 The western part of the Lower Witham catchment – known as the 'Heath' catchments – were represented by 'typical' hydrograph shapes (i.e. not dependent on storm duration) scaled to the required design event peak flow. This was due to the permeable nature of these subcatchments, where slow, prolonged response to rainfall input was prevalent.

7.6 Pumped subcatchments within the Lower Witham catchment were scaled based on analysis of previous events, in which a scaling factor/ratio between the volumetric input generated by the FSR/FEH rainfall-runoff method (based on known rainfall depths) and the pumped volume recorded by the IDBs was calculated. This scaling factor was assumed to be applicable to design rainfall inputs, and therefore applied to all design event models (with an increase of 20% to account for climate change where necessary).

7.7 The hydrological boundaries were added to the Lower Witham FMI 'with defences' model and run for the full suite of return periods to produce flood extents. These were exported to the MapInfo GIS package in order to add flow paths and undertake additional manual post processing.

7.8 For areas where 1D modelling using the main model proved to be too inaccurate to map flow paths, a series of child sub-models using 2D modelling were developed in Infoworks RS to improve the mapping. In the case of the lower reaches of the River Bain adjacent to Coningsby and Tattershall, this was done using a hydraulically linked 1D-2D approach. For other locations such as in the Fens to the west of the Witham, a simplified 2D approach to define the flow path was considered to be adequate. The flow paths were added to the 1D flood extents from the main Lower Witham FMI model during processing.

7.9 Quality Assurance checking of the final flood outlines was undertaken both by AECOM and the Environment Agency, to check for errors in the flood mapping process, and for consistency with observed historical patterns of flooding. Where issues were raised, these were addressed either making amendments to the model and re-running, undertaking manual edits to the flood map extents, or by making recommendations for further work. A full audit trail of all remaining issues and manual edits to the flood mapping was provided with the final output in the form of a GIS layer containing comments.

7.1 It should be noted that the flood mapping produced by this study downstream of Coningsby supersedes the mapping produced for this area by the River Bain Flood Map Improvements study. This is because the latter did not fully account for the interaction between the Lower Witham floodplains and the River Bain, and the fact that more accurate 2D modelling of flow paths was undertaken in this location for the Lower Witham study.

7.2 It should also be noted that the flood extents produced by this study may be less extensive than in a known historical event of the same approximate return period. This can occur because the hydraulic coefficients and control structure settings adopted for the 'design' scenario may result in lower levels than those observed during historical events. Similarly, the design condition assumes 'normal' operation of hydraulic structures and does not account for blockages.

7.3 An analysis of the standard of protection and freeboard was also undertaken as part of the study. This involved calculation of the indicative standard of protection, the required freeboard and the actual standard of protection at each river cross-section.

7.4 The final deliverables to the Environment Agency included all flood extent maps in ArcGIS .shp format as requested in the brief, plus a technical report including tables detailing the water level and flow at each model node, and long section plots for each watercourse for the 'with defences' model results.

7.5 Modelling of the undefended scenario and the generation of areas benefitting from defences is being undertaken separately for the three fenland sub-catchments (Lower Witham, South Forty Foot and Stonebridge) using a TUFLOW 2D model. The methodology and deliverables will be presented separately.

RECOMMENDATIONS

7.6 The following actions should be considered in any future work to improve the Lower Witham FMI model and accuracy of flood mapping:

- The model should be updated with new survey data as it becomes available;
- The LiDAR data on which part of the DTM used in the mapping is based, has been deemed as being of questionable accuracy by the Environment Agency, but was used in this study in the absence of more accurate data. The DTM, model and resultant mapping should be updated when more accurate LiDAR data becomes available in the future.
- There are many individual siphons and penstocks in the Lower Witham catchment which link individual drains with other parts of the floodplain and main river system. It was agreed with the Environment Agency that the majority of these should be assumed to be closed during the design event and as a result there was no requirement to add these to the model. Where siphons and penstocks have been modelled as open in the design condition it was necessary to make assumptions concerning dimensions and inverts in a number of cases as they are not included in the surveys. The collection of further survey data would be beneficial to improve the accuracy of the model, both for the existing siphons and penstocks in the model and those which are not currently modelled in case they need to be added in the future.
- The upper part of Ruskington Beck was not joined to the main 1D model for the final design runs due to some non-convergence issues and the requirement for a small timestep which would have significantly increased the run time of the main model. Further work to improve the convergence and run time would be beneficial to allow this part of the model to be permanently joined to the main model.
- The left bank floodplain at Langworth Gauging Station should be re-schematised to utilise overbank spills rather than extended cross-sections. This will enable the model to produce more accurate water levels at the gauging station.

CONCLUSIONS

7.7 This report has provided details of the hydraulic modelling and flood mapping undertaken in the course of the Lower Witham Flood Map Improvements study. The study has resulted in a hydraulic model that is considered 'fit for purpose' and flood extents that are of the required accuracy for use by the Environment Agency.

7.8 All deliverables specified in the Environment Agency's original brief have been supplied. This report has documented the methodology applied to produce these deliverables.

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AECOM

Appendix A – Barlings Eau Pre-Feasibility

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Appendix A – Barlings Eau Pre-Feasibility

Introduction

The Witham Flood Map Improvements (Phase 2) brief included a pre-feasibility analysis for floodplain optimisation on the Barlings Eau. The brief set out that this analysis should occur following the completion of the Standard of Protection Analysis, prior to the undefended and ABD elements of the project. It was agreed with the Environment Agency at a meeting on the 17th July 2008 that a full pre-feasibility study is not required. Instead it was agreed that Faber Maunsell would undertake a preliminary pre-feasibility analysis (PPFS) to look at the hydraulic effects of flood management options using the updated InfoWorks RS model, but not a cost-benefit analysis and environmental assessment. Consequently, our proposal included a fee estimate for the preliminary analysis not the full pre-feasibility study.

It was agreed with the Environment Agency that the analysis of flood management options would cover the Barlings Eau from the confluence with the Witham upstream as far as Langworth. The flood management options would focus on the lower reaches of the Barlings Eau and also on the village of Langworth, since these are the locations where overtopping of defences has caused significant flood risk to property during recent historical events. The purpose of the analysis is to investigate flood management options to increase the standard of protection at these locations without reducing the standard of protection at upstream or downstream locations.

Overview of Hydrology and Modelling Updates

Faber Maunsell updated and re-calibrated the Lower Witham catchment hydraulic model using InfoWorks RS software in 2007. This update of the modelling was undertaken as part of the larger Witham Catchment Strategic Model (WiCSM) project which commenced in 2005. The calibration process included the November 2000 and February 2001 events and a good match to the observed flood levels and extents was achieved.

Following calibration of the model, new design hydrology estimates were developed, and the model is currently undergoing development for the purposes of flood mapping. The development of the model has focussed on extending the model to accommodate the 1 in 1000 year flow plus an allowance for climate change, and on the scaling of inflow boundaries to match the design hydrology estimates. The Lower Witham ISIS model, from which the WiCSM calibration model originates, already included representations of the fens bordering the Lower Witham and Delphs. Therefore, the development of floodplains has primarily focussed on the tributaries of the Witham.

As part of the model development process, the schematisation of all floodplains is being reviewed to ensure it is appropriate for the purposes of flood mapping. In addition, any additional survey data which has become available since the WiCSM model calibration is being incorporated to improve the accuracy of the model.

Although the WiCSM calibration model included the floodplains of the Barlings Eau, these were represented in the model as extended cross-sections which is a legacy of the original ISIS model. This schematisation was considered acceptable during WICSM model calibration because there was not significant out of bank flow in the two calibration events. However, during model development for the flood map improvements (FMI) project, it was considered necessary to re-schematise large parts of the Barlings Eau floodplain using overbank spills to 1D storage areas. In the areas where it was applied, this schematisation is considered more representative of the real flooding mechanism for more extreme events; it will result in more accurate flood mapping than using extended cross-sections, and will facilitate linked 1D-2D modelling in the future.

The latest design hydrology estimates adopt an FEH statistical approach, in contrast to the original study which was based on a calibrated rainfall-runoff approach. The differences in hydrological approach have the potential to change the peak flows and volumes, and therefore the standards of protection which are produced by the updated model may not match those currently accepted based on the results of the previous Lower Witham ISIS model.

Because the results of the Barlings Eau PPFS were required prior to completion of the model to be used for the final flood mapping and SoP outputs, the model used in the PPFS is necessarily an interim version. However, efforts in model development have concentrated on producing accurate representations of the floodplain on the Barlings Eau, Marsh Drain and Lower reaches of Stainfield Beck. Similarly, efforts in scaling the hydrological inputs to match the design hydrological estimates have focussed on attaining the required flows for the Barlings Eau, Stainfield Beck, and all the major inputs to the Lower Witham which are important in determining water levels around the confluence of the Barlings Eau and the Old Witham. The water levels in this area are important as there is a significant backwater effect which contributes to the flood risk on the Barlings Eau.

Despite the fact that the PPFS is based on an interim version of the FMI model, it is considered that the model is sufficiently well developed to produce meaningful results for the study, at least in terms of the relative changes in water levels. Although it is not anticipated that the flood mapping extents and the standards of protection will vary significantly between the PPFS study and the final flood map improvements model, there is a risk that this could occur. Changes to the modelled water levels are most likely to occur as a result of ongoing work to refine the rules at the Grand Sluice. This has the potential to affect water levels on the Witham at Bardney, which in turn would alter the backwater effect on Barlings Eau.

Existing Flooding Pattern and Mechanism

In developing the model for use in the PPFS, it was necessary to check that the model was accurately simulating the pattern and mechanisms of flooding as observed during recent events. This allowed the model development to focus on correcting inaccuracies in overbank spill levels, and resulted in an 'existing' model which gives realistic standards of protection to provide a base case against which to judge the PPFS options.

Observed Flooding

Environment Agency staff provided a detailed account of the locations where flood defences were overtopped during recent extreme flood events, and details of any remedial works which have been carried out since. The two main locations where formal defences were overtopped in the Summer 2007 flood events were:

- On the right bank of the Barlings Eau, downstream of the road bridge in Langworth village;
- On the left bank of the Barlings Eau into Stainfield Fen, approximately 1km upstream of the confluence with the Old Witham.

Developing flood management options to increase the standard of protection at these locations, without reducing the standard elsewhere, is the main objective of this PPFS.

In addition there was flooding out of the main channel at other locations along the Barlings Eau, but these occurred in areas that are allowed to flood naturally where there are no formal defences. Further details of these locations are provided in Table A1 below.

Modelled Flooding

The FMI model was developed from the WiCSM calibration model to ensure that the simulated flooding pattern and mechanism is consistent with the observed. Of primary importance in this was the correct definition of overbank spill levels, and the following data sources were used to define these in the updated FMI model:

- Stainfield Fen flood bank survey, EDI 2008;
- Long section bank survey of Barlings Eau, Ratcliff 1998;
- LiDAR data;
- Discussions with Environment Agency Staff.

In a number of cases the bank top/defence levels provided by the survey and LiDAR were considered to be inappropriate for entry into the model, either because the survey did not pick up a realistic maximum line of defence, or the Environment Agency have undertaken works to raise defences since the summer 2007 events. Notable such locations are:

- On the right bank upstream of Langworth road bridge, Langworth. This area is not formally defended and water overtops to flood the adjacent field. The spill level was not accurately picked up by the survey data or LiDAR. EA staff advised that a spill level of 6.2m AOD is appropriate.
- For the setback defence at Scothern Lane, Langworth. EA staff advised that this defence was constructed to 7.2m AOD in 1994 but was subsequently raised to 7.3m AOD following the summer 2007 floods.
- At the low spot which overtops into Stainfield Fen from the left bank of the Barlings Eau at approximate chainage 1km, EA staff advised that the low spot was raised to 4.25m AOD following the summer 2007 floods. The elevation of the low spot prior to raising (from the 2008 survey) was 4.14m AOD.

Once all the spill levels in the model had been appropriately set to existing levels, the 1 in 5 and 1 in 10 year events were modelled. The preliminary flood mapping output for these two events can be seen in Figures A4 and A5 respectively.

The modelling shows that the standards of protection at the key locations are:

- Barlings Eau RB, downstream of the road bridge in Langworth Village – approximately 1 in 5 years;
- Barlings Eau LB chainage 1km – 1 in 5 to 1 in 10 years.

Further details on the relative order at which banks/defences are overtopped is provided in Table A1. This includes areas which are not formally defended and gives an idea of the relative standard at different locations during a catchment wide design flood event. During a localised or multi-peaked observed event the order of overtopping is likely to be different.

The locations of flooding in Table A1, their relative order and likely extents (Figure A5) were reviewed by Environment Agency staff and confirmed to be realistic. It was therefore concluded that the model was sufficiently accurate to use as an 'existing situation' (base case) for the investigation of flood management options.

Order	Channel	Bank	Chainage (m)	Location	Formal Defence	Comment
1	Barlings Eau	Left	4500	Gatecliff Wood	No	
2	Barlings Eau	Left	5200	Opposite Stainfield Grange	No	
3	Abbey Island	Right	150	Abbey Island	No	First spill to start filling Abbey Island between Barlings Eau and Marsh Drain (followed quickly by other spills from both watercourses)
5	Barlings Eau	Right	7900	U/s of Langworth Bridge	No	Embanked but not part of EA formal defence
6	Barlings Eau	Left	3500	U/s of Ferry House	No	
7	Barlings Eau	Right	6000	Newball Wood Cottages	No	Contained by higher ground behind bank
8	**Barlings Eau**	**Right**	**7767**	**D/s of Langworth Bridge**	**Yes**	**Fills area adjacent to gauging station**
9	Old Witham	n/a		Branston Island Sluice	n/a	Gates open when level in Old Witham reaches 3.87m AOD
10	**Stainfield Beck**	**Left**	**992**	**Opposite Stainfield Grange**	**Yes**	**Spills from low spot of 4.48m as advised by EA**
11	**Barlings Eau**	**Left**	**1000**	**Stainfield Fen**	**Yes**	**Spills from low spot of 4.25m AOD as advised by EA**

Table A1: Relative order of overtopping at 1 in 10 year return period (Defended areas in Bold

Options Considered

At the proposal stage, it was identified by Faber Maunsell that flooding over the left bank of the Barlings Eau into Stainfield Fen is likely to be more significantly influenced by high water levels in the River Witham than flows from the Barlings Eau. Therefore the hydraulic benefit of floodplain optimisation measures on the Barlings Eau is unlikely to be significant in increasing the standard of service at this location. This view was based on Faber Maunsell's previous knowledge and experience of modelling in the catchment. As a result of this it was proposed that the analysis of flood management options would be undertaken in two parts, to look at options for the standard of protection in Langworth village separately from options for the lower reaches of the Barlings Eau. It was agreed that the interaction between options for the two areas would be ignored at the preliminary pre-feasibility stage.

Lower Reaches of Barlings Eau

In order to test the hypothesis that the backwater effect from the Lower Witham is the most significant factor in overtopping of the Barlings Eau embankment into Stainfield Fen, it was considered appropriate to undertake a sensitivity analysis whereby the Lower Witham model is run with a baseflow on the Barlings Eau, but with full flood inflows on the Lower Witham and other tributaries. This would determine whether floodplain optimisation measures on the lower reaches of the Barlings Eau are appropriate. If the flooding mechanism is predominantly from the backwater effect from the Lower Witham then the options for alleviation would revert to the off-line storage options identified during the Lower Witham Strategy Review.

Langworth Village

It was agreed at a meeting between Faber Maunsell and the Environment Agency on 18th March 2009 that the following specific options would be investigated with a view to increasing the standard of protection in Langworth Village:

- Option 1 - Raising defences on the right bank of the Barlings Eau downstream of the road bridge
- Option 2 – Providing on-line flood storage upstream of the railway embankment
- Option 3 – Setting back the left embankment of the Barlings Eau, chainage 6km to 7km.

For all options in Langworth it was necessary to assess the impact on water levels at other critical locations to ensure there are no negative impacts.

The results were analysed to determine if there is an option which decreases flood levels in Langworth whilst not increasing flood levels downstream of Langworth.

Options Modelling

Details of the model set-up required to investigate each of the options is provided below:

Lower Reaches of Barlings Eau

The full Lower Witham model was set up with 1 in 10 year design hydrology for all inputs except Barlings Eau and Welton Beck which were set to baseflow. It was not considered necessary to model the 1 in 5 year event.

Langworth Village Option 1 – Bank Raising

The full Lower Witham model was set up with spills on the right bank between the road bridge (chainage 7.77km) and chainage 6.5km raised well above the 1 in 10 year flood level. The model was run with the 1 in 5 and 1 in 10 year hydrology.

Langworth Village Option 2 - Storage

Using the existing situation model it was determined that water would begin to overtop the defence on the right bank downstream of the road bridge at a flow of 31 cumecs. Between the railway embankment and the road bridge there are two significant incoming tributary inflows on the left bank as shown in Figure A1:

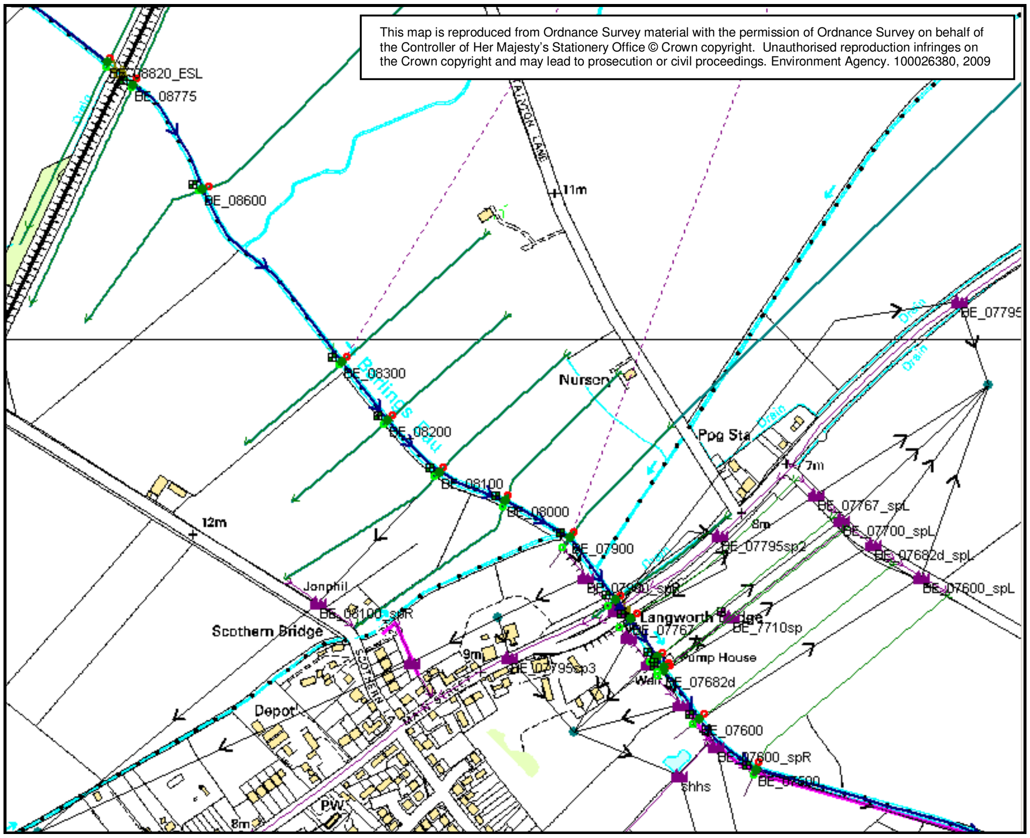

Figure A1: Significant tributary inflows between the railway embankment and Langworth Bridge

The downstream tributary is referred to as Bullington Beck, whilst the upstream tributary will be referred to as Stainton Beck. The inflow from these two tributaries is calculated in the model as part of a larger lumped hydrological catchment. These flows are input into the model as lateral inflows which weight the flow from the larger lumped catchment according to the catchment area of the two becks. Although this method will only approximately calculate the inflow of the two tributaries, it is sufficiently accurate to demonstrate that they contribute a significant proportion of the flows reaching Langworth Bridge. This matches the perceptions of Environment Agency staff.

Given that the inflows from Bullington and Stainton Becks are significant, and these enter the Barlings Eau downstream of the proposed storage site, a greater degree of throttling of flows is required upstream of the railway bridge than would otherwise be the case. Table A2 shows the peak flows that can be allowed to pass through the railway embankment at a 1 in 10 year return period to limit flows reaching Langworth Bridge to 31 cumecs

	Flows (cumecs)			
Return Period	**Existing @ Rail Embankment**	**Combined from Becks (A)**	**Max d/s Langworth Bridge (B)**	**Throttle Max (B-A)**
1 in 10	21.56	24.74	31.00	6.26

Table A2: Calculation of the maximum allowable pass forward flow at railway embankment

The throttle at Langworth railway embankment was modelled using a QH relationship, in which the existing QH relationship was used up to the maximum flow target, with the flow kept constant at the target value with increasing stage beyond this point. The QH relationship was entered into a copy of the full Lower Witham model and was run with the 1 in 10 year hydrology. It was not considered necessary to run this option with the 1 in 5 year hydrology.

Langworth Village Option 3 – Setting back the left bank

For option 3, a change to the set-up of the existing model was required. In the existing model, the floodplain on the left bank between chainages 6 and 7km was initially set up as extended cross-sections. The model was set up in this way because the embankment is relatively low in this area and is not a formal defence, therefore it overtops at a relatively low return period. Therefore, modelling this reach as extended cross-sections was considered appropriate in order to produce more accurate 1D flood mapping. The problem with this model schematisation for investigation of the benefits of setting back the banks at low return periods, is that flow will be conveyed behind the embankment before it is overtopped in the existing situation, and therefore the full benefit of setting back the embankment would not be demonstrated using the model.

To fully demonstrate the effects of setting back the banks at the 1 in 5 and 1 in 10 year return periods, the existing model was amended to represent the floodplain as spills into a storage area between chainages 6km and 7km. A second model was then set up to represent Option 3, in which the spill units were populated with approximate floodplain levels behind the embankment to represent a scenario without the embankment in place. Simulations for the 1 in 5 and 1 in 10 year return periods were run and the modelled water levels were assessed and compared to the existing Barlings Eau levels.

Model Results and Discussion

The full results of the PPFS modelling are presented in terms of the maximum water level at each cross-section on the Barlings Eau in Tables A3 to A6. The model results at key locations and their implications are discussed below.

Lower Reaches of Barlings Eau

The results in Table A4 show the effect on water levels of reducing flows on the Barlings Eau to baseflow levels during a 1 in 10 year event (See column 'Stainfield Sensitivity'). This shows that the maximum reduction in water levels achieved at the location of the low spot on the left bank of the Barlings Eau (Chainage ~1km) is 0.16m. This is the maximum reduction in water levels that could be achieved by floodplain optimisation on the Barlings Eau and this demonstrates that the water levels at this location are mainly a function of the backwater effect from the Lower Witham system.

In reality, any floodplain optimisation on the Barlings Eau would reduce peak flows by a much lesser amount than that modelled, and therefore any reduction in water levels in the lower reaches of the Barlings Eau is likely to be small or insignificant. This is backed up by the results from Options 1-3 for Langworth Village.

Langworth Village Option 1 - Bank raising

The results in Table A3 and A4 show that there is no significant change in water levels at any point along the Barlings Eau as a result of raising the right bank downstream of Langworth bridge. Initially this may seem surprising as logic would suggest that this option would push more flow downstream thereby raising water levels. Following a closer inspection of the results it appears that this effect does occur, but downstream water levels are only raised by a maximum of 4mm, which is not enough to register an increase to 2 decimal places in the results table.

Another contributing factor is that the existing overtopping flow is only 0.28 cumecs at the 1 in 10 year return period, and therefore the flow increase as a result of raising this low spot would be relatively insignificant in comparison to the 1 in 10 year existing flow of 36.5 cumecs at the gauge.

In the lower reaches of the Barlings Eau it should not be expected that levels would be significantly increased as a result of this option as the sensitivity analysis has shown that water levels in this area are primarily influenced by backwater effects from the Lower Witham.

If this option is progressed to a full pre-feasibility study then the effects of bank raising at higher return period events should be investigated as it is possible that increases in water level at some downstream locations may be more significant.

Langworth Village Option 2 - Storage

The results in Table A4 show that the 1 in 10 year in-line storage option would reduce water levels by 0.2m at Langworth gauging station. With this reduction in water levels the spill on the right bank is just on the verge of overtopping and so it can be concluded that the standard of protection has increased from approximately 1 in 5 years to 1 in 10 years in Langworth village. As would be expected the benefits of storage are reduced further downstream with a reduction in water levels of 0.05m at the confluence with the Lower Witham.

The trade off for this benefit is an increase in water levels upstream of the railway embankment of up to 1.00m. The increase in the flooded extent as a result of this is illustrated in Figure A2.

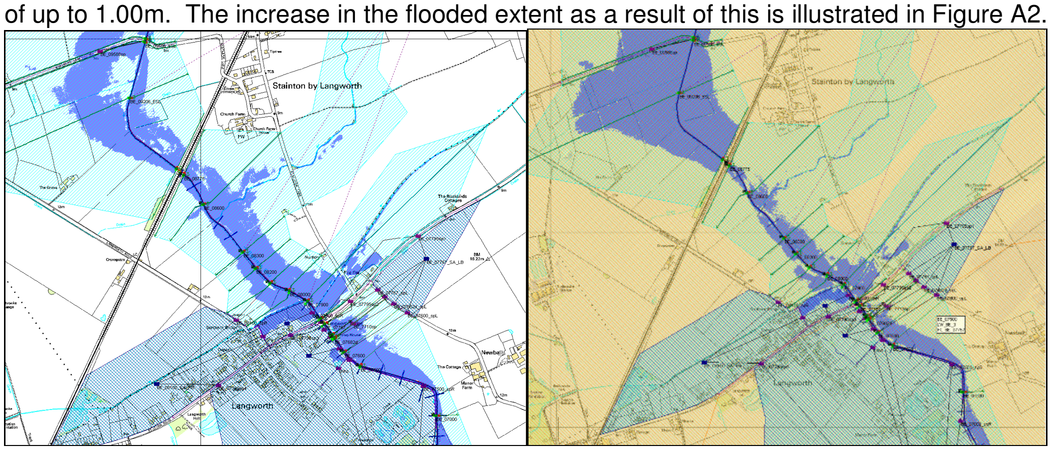

Figure A2: Option 2 Storage - Comparison of flooded extent upstream Of Langworth for existing situation (left) vs storage option (right)

A significant increase in standard of protection using on-line storage would require use of a second storage site since the flows produced by Bullington and Stainton Becks would exceed the 31 cumecs maximum ignoring the inflow from the upper reaches of the Barlings Eau. It would therefore be necessary to identify a second storage site on one of the two incoming becks. A preliminary review has identified the A158 road embankment which crosses Bullington Beck near Bullington as a potential site. Figure A3 shows a 3D view of this location where it may be possible to store water up to a level of 11m AOD.

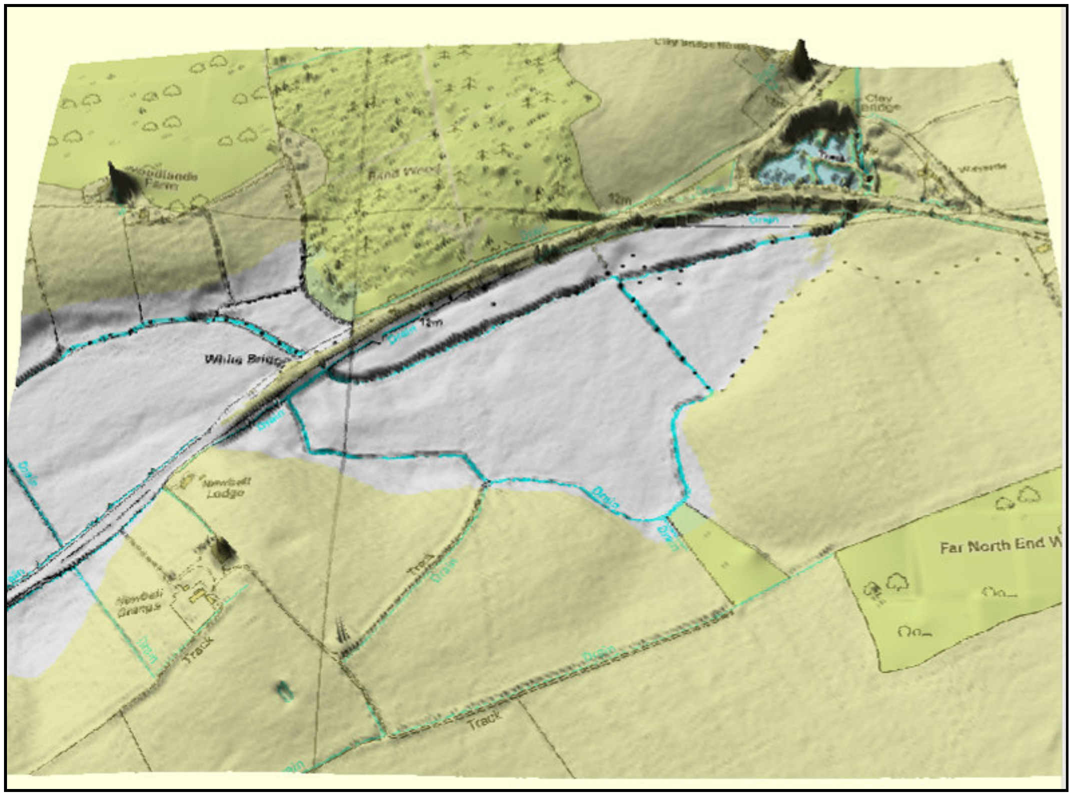

Figure A3: 3D view of potential secondary storage location on Bullington Beck showing 11m AOD contour

Langworth Village Option 3 – Setting back the left bank

The results in Tables A5 and A6 show that the change in the schematisation of the existing model undertaken in order to investigate Options 3 has resulted in a slight change in existing water levels. The increase in water levels is a maximum of 0.11m around Langworth village at the 1 in 5 year event. The change in water levels is much less marked for the 1 in 10 year return period with many locations actually showing a decrease in water levels. This is not particularly surprising as it is likely that the revised schematisation will have less impact on water levels with increasing return period as the storage behind the embankment becomes fully utilised well before the peak of the event. For the purpose of assessing the benefit of Options 3 the results were compared against the revised existing schematisation (spills into a storage area) to ensure consistency.

The results show that as a result of removing the left embankment between chainages 6 and 7km, water levels are reduced upstream by up to 0.2m for the 1 in 5 year event and 0.12m for the 1 in 10 year return period at Langworth gauging station. It is to be expected that there is a bigger reduction in water levels at a lower return period because the area behind the embankment will have a greater unused storage volume. As return periods increase it is likely that the benefit of this option will diminish to zero, because the storage behind the embankment would fill up prior to the peak of the event. It can also be seen that there is benefit from this option in the lower reaches of the Barlings Eau with water levels reduced by 0.05m at the 1 in 5 year event. At the 1 in 10 year event and above this benefit disappears as the influence of the backwater effect from the Lower Witham increases.

The observed hydraulic effects makes this an attractive hydraulic option for flood risk management on the Barlings Eau because the existing standard of service for Langworth and the Lower reaches of the Barlings Eau is between a 1 in 5 and 1 in 10 year return period. The option is most effective at reducing water levels all along the Barlings Eau at the 1 in 5 year event, and therefore it will achieve a small increase in the standard of service at both upstream and downstream locations. The maximum increase in standard that can be achieved in the lower reaches of the Barlings Eau appears to be to a 1 in 10 year event as by this return period the backwater effect from the Lower Witham is more dominant. However, the results suggest that at Langworth village this option can increase the standard of protection beyond the 1 in 10 year return period.

Conclusions and Recommendations

The conclusions that were drawn from the Barlings Eau PPFS modelling, and recommendations for future work are as follows:

Lower Reaches of Barlings Eau

The modelling has shown that water levels in the lower reaches of the Barlings Eau are mainly caused by the backwater effect from the Lower Witham, rather than the flows in the Barlings Eau. It was shown that the maximum reduction in water levels that can be achieved by reducing peak flows in the Barlings Eau is 0.16m at the 1 in 10 year event and in reality any reduction due to flood management options on the Barlings Eau is likely to be insignificant.

In light of the above, it is recommended that the Environment Agency refer to the off-line storage options identified during the Lower Witham Strategy Review in order to determine whether an increase in standards of protection can be achieved in this area.

Langworth Village Option 1 - Bank raising

The modelling has shown that raising the defences on the right bank downstream of Langworth Bridge has not significantly increased water levels at any location along the Barlings Eau during a 1 in 5 or 1 in 10 year return period. It is therefore concluded that this is a hydraulically feasible option to increase the standards of protection in Langworth Village, and it would be worth considering this option as part of a full pre-feasibility study.

It is recommended that if this option is investigated further, the hydraulic effects are investigated for all return periods, since there is the potential for events of greater return period than the 1 in 10 year event to produce a more significant downstream water level increase.

Langworth Village Option 2 - Storage

The modelling has shown that on-line storage upstream of the railway embankment at Langworth has the potential to reduce water levels in Langworth village by approximately 0.2m at the 1 in 10 year return period event. This would increase the standard of service of the existing defence on the right bank downstream of Langworth Bridge from the current 1 in 5 year to a 1 in 10 year standard. In order to achieve this, water needs to be stored upstream of the railway embankment to a level 1m higher than the existing 1 in 10 year water level. This results in a significant increase in the flood extents upstream of the railway embankment. If the Environment Agency wish to investigate storage to a standard greater than 1 in 10 years it will be necessary to investigate a secondary storage site on either Bullington or Stainton Becks. As part of this, it would be advisable to calculate the hydrological inputs from Stainton and Bullington Becks in more detail, rather than relying on the current (strategic level) lumped approach.

It is considered unlikely that upstream storage would turn out to be the preferred option if it was investigated as part of a full pre-feasibility study. This is because of the large volumes of water that need to be stored and the likely high costs of providing the required storage for a relatively modest increase in standards of protection. It is recommended that these points are considered by the Environment Agency in deciding whether to include a storage option as part of any full pre-feasibility study.

Langworth Village Option 3 – Setting back the left bank

The modelling has shown that there is a significant benefit to setting back the left bank between chainages 6 and 7km on the Barlings Eau. This option will increase the standard of protection at Langworth village beyond the 1 in 10 year event and also shows a benefit in the lower reaches of the Barlings Eau for events less than the 1 in 10 year return period. As the existing standard of protection from the left bank of the lower reaches of Barlings Eau is between 1 in 5 and 1 in 10 years, it is likely that this option will result in modest increase in the standard of protection at this location. At the 1 in 10 year event and above, the backwater effect from the Lower Witham becomes dominant, negating any benefits seen further upstream.

It is therefore concluded that this is a hydraulically feasible option to increase the standards of protection in Langworth Village and in the Lower reaches of the Barlings Eau, and it would be worth considering this option as part of a full pre-feasibility study. It is considered likely that this option would stand a good chance of becoming the preferred option when considered as part of a full pre-feasibility study, since the costs would be relatively low and benefits are seen at both upstream and downstream locations. If this option is progressed to full pre-feasibility study stage, then it is recommended that the effects at return periods greater than the 1 in 10 year event are investigated using the model.

Summary

This preliminary pre-feasibilty study has investigated the hydraulic performance of a number of flood risk management options for the Barlings Eau, with the objective of increasing the standards of protection in the Lower reaches of the Barlings Eau and at Langworth village.

It was found that water levels in the lower reaches of the Barlings Eau are dominated by the backwater effect from the Lower Witham at return periods of 1 in 10 years and above. Therefore, to increase standards of protection beyond the 1 in 10 year return period would require an option that achieved a reduction in water levels in the whole of the Lower Witham system, such as the off-line storage options identified during the Lower Witham Strategy Review.

At Langworth village, three options were investigated to increase the standard of protection and all three achieved a significant benefit without a reduction in standards elsewhere. The upstream storage option produced the most significant benefit in terms of reduction in water levels but the costs of this option are likely to be significant and further investigation into the most appropriate storage site(s) will be necessary.

The option to set back the left bank between chainages 6 and 7km on the Barlings Eau would appear to be the best option hydraulically since this results in an increase in standards of protection at both upstream and downstream locations. The costs involved with this option are likely to be relatively modest and it is therefore considered likely that it would be the preferred option in a full pre-feasibility study.

The Environment Agency may wish to consider investigating the benefits of combining Option 1 (bank raising) with Option 2 (setting back the left bank) as it is likely that this could achieve a further increase in standard of protection at Langworth without affecting standards downstream. Initially this could be done as a preliminary hydraulic feasibility as an extension of this report.

It is recommended that the Environment Agency give consideration to the findings of this preliminary pre-feasibility study to decide whether it would be worth progressing to a full pre-feasibility assessment. If this is the case, Faber Maunsell would need to review the outline costs submitted as part of the original Lower Witham FMI proposal.

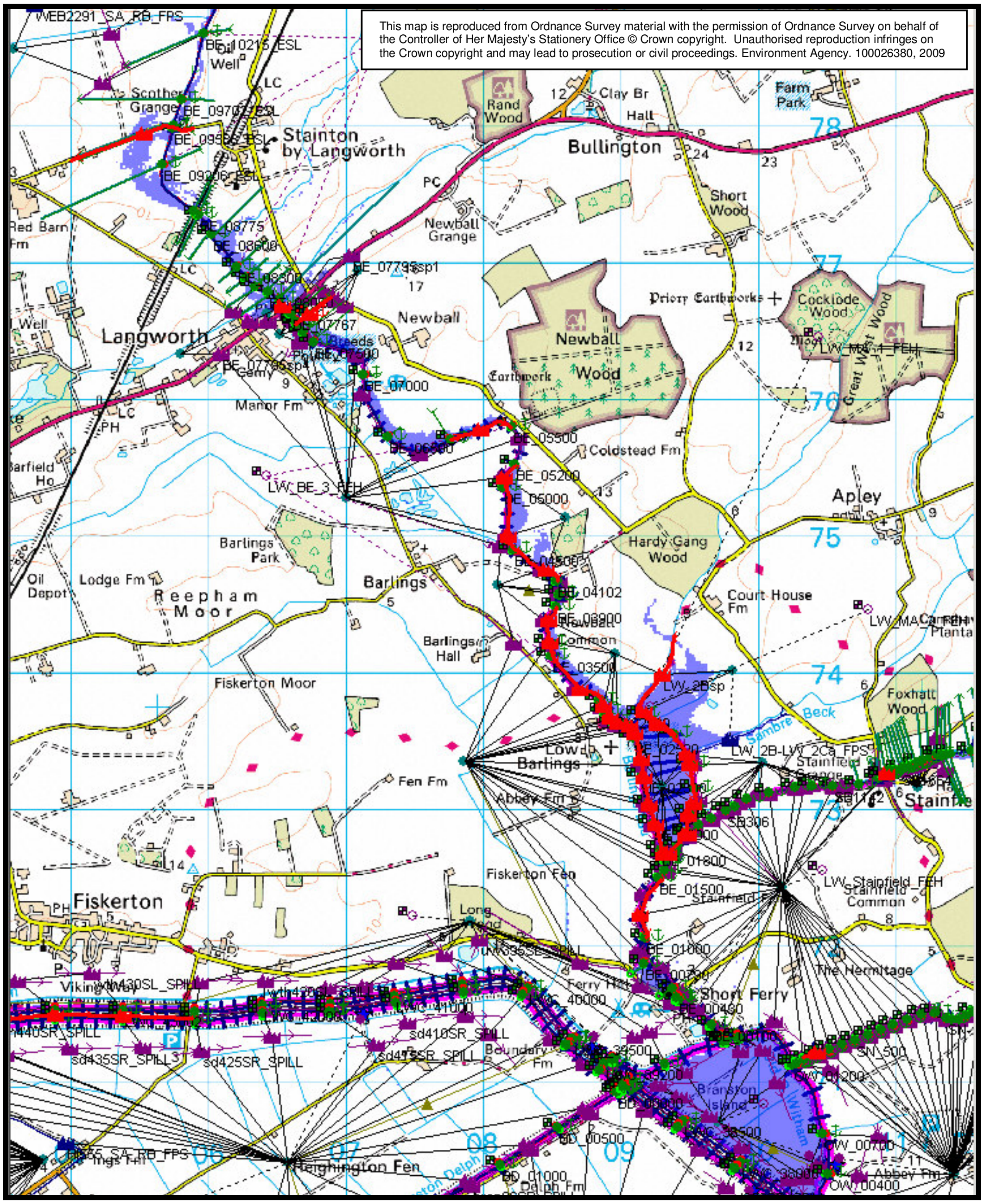

Figure A4: Existing 1 in 5 year Event – Preliminary Flood Mapping Extents
(Spills which are operating highlighted red)

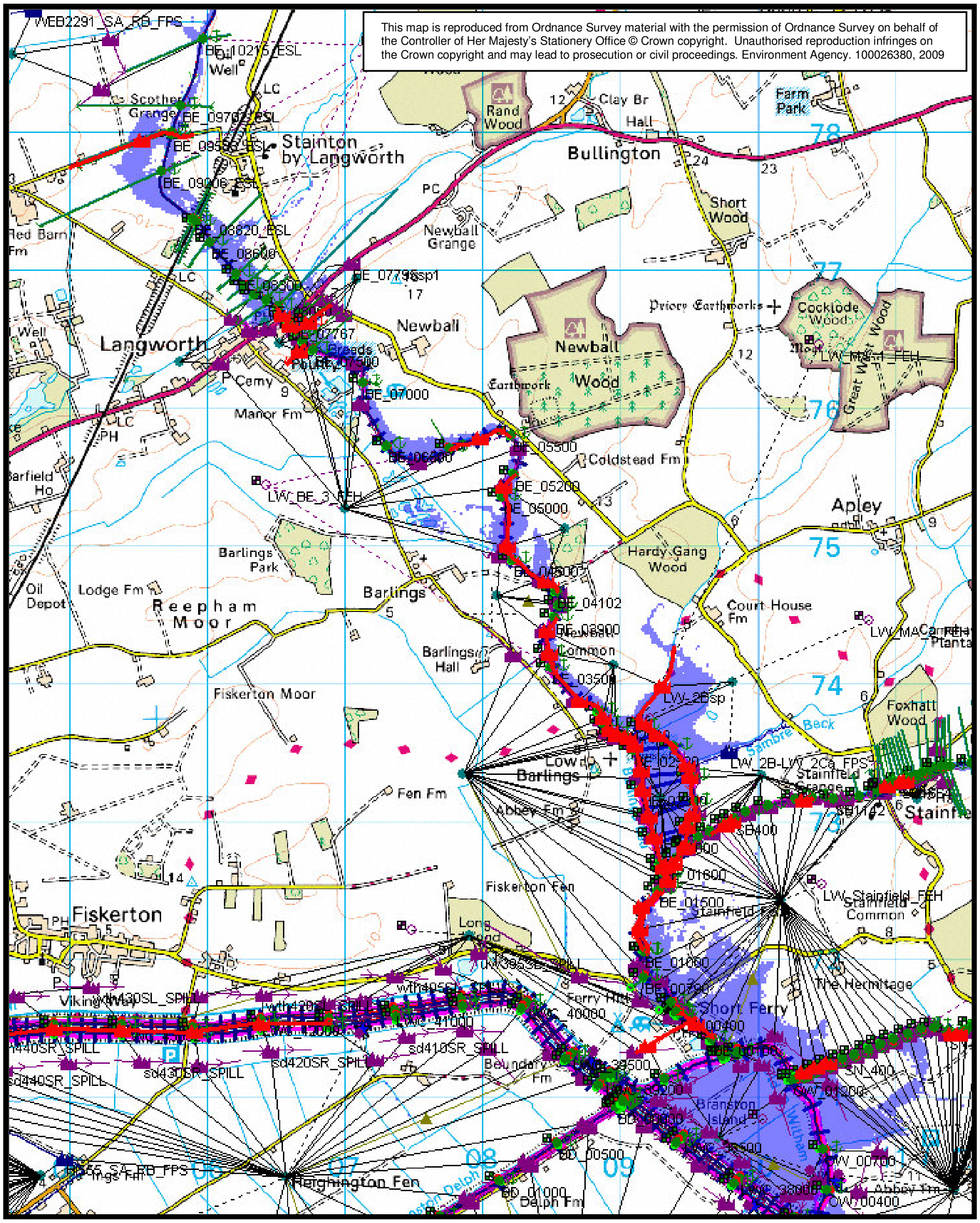

Figure A5: Existing 1 in 10 year Event – Preliminary Flood Mapping Extents
(Spills which are operating highlighted red)

Modelling Results

		1 in 5 year Water Levels (m AOD)		
Location	**Model Node**	**Existing**	**Langworth Option 1**	**Change vs Existing**
Welton Beck confluence	BE_11714!!_ESL	7.40	7.40	0.00
	BE_11224_ESL	7.26	7.26	0.00
	BE_10272_ESL	7.05	7.05	0.00
	BE_10215_ESL	6.89	6.89	0.00
	BE_09702_ESL	6.79	6.79	0.00
Stainton Lane bridge u/s	BE_09588_ESL!	6.77	6.77	0.00
Stainton Lane bridge d/s	BE_09588_ESL	6.75	6.75	0.00
	BE_09206_ESL	6.70	6.70	0.00
Railway Embankment u/s	BE_08820_ESL	6.61	6.61	0.00
Railway Embankment d/s	BE_08775	6.59	6.59	0.00
Stainton Beck confluence	BE_08600	6.57	6.57	0.00
	BE_08300	6.55	6.55	0.00
	BE_08200	6.53	6.53	0.00
	BE_08100	6.52	6.52	0.00
	BE_08000	6.52	6.52	0.00
Bullington Beck confluence	BE_07900	6.51	6.51	0.00
Langworth Bridge u/s	BE_07792	6.40	6.40	0.00
Langworth Bridge d/s	BE_07767	6.36	6.36	0.00
	BE_07700	6.33	6.33	0.00
Langworth gauging station	BE_07700!	6.32	6.32	0.00
	BE_07682d	6.29	6.29	0.00
	BE_07600	6.28	6.28	0.00
	BE_07500	6.25	6.25	0.00
	BE_07000	6.00	6.00	0.00
	BE_06500	5.77	5.78	0.00
Newball Wood House	BE_06000	5.51	5.51	0.00
	BE_05500	5.12	5.12	0.00
	BE_05200	4.91	4.91	0.00
	BE_05000	4.88	4.88	0.00
	BE_04500	4.84	4.84	0.00
	BE_04102	4.57	4.57	0.00
Gate Cliffe Farm bridge	BE_04102!	4.57	4.57	0.00
	BE_04000	4.54	4.54	0.00
	BE_03900	4.48	4.48	0.00
	BE_03600	4.35	4.35	0.00
	BE_03500	4.34	4.34	0.00
	BE_03000	4.34	4.34	0.00
	BE_02900	4.34	4.34	0.00
Barlings Abbey	BE_02800	4.34	4.34	0.00
	BE_02688	4.33	4.33	0.00
	BE_02677	4.33	4.33	0.00
	BE_02500	4.33	4.33	0.00
	BE_02300	4.33	4.33	0.00
	BE_02100	4.33	4.33	0.00
	BE_02000	4.33	4.33	0.00
	BE_01800	4.33	4.33	0.00
Stainfield Beck confluence	BE_01700	4.33	4.33	0.00
	BE_01700!	4.33	4.33	0.00
	BE_01590	4.33	4.33	0.00
	BE_01500	4.33	4.33	0.00
	BE_01000	4.33	4.33	0.00
	BE_00790	4.33	4.33	0.00
	BE_00500	4.33	4.33	0.00
Short Ferry Bridge	BE_00430!	4.33	4.33	0.00
	BE_00430	4.33	4.33	0.00
	BE_00400	4.33	4.33	0.00
	BE_00100	4.33	4.33	0.00
	BE_00100!	4.33	4.33	0.00

Table A3: Barlings Eau PPFS Modelling Results – 1 in 5 year Existing and Option 1

		1 in 10 year Water Levels (m AOD)						
Location	Model Node	Existing	Stainfield Sensitivity	Change vs Existing	Langworth Option 1	Change vs Existing	Langworth Option 2	Change vs Existing
Welton Beck confluence	BE_11714!!_ESL	7.51	5.67	-1.85	7.51	0.00	7.77	0.25
	BE_11224_ESL	7.40	5.44	-1.96	7.40	0.00	7.74	0.35
	BE_10272_ESL	7.17	5.07	-2.10	7.17	0.00	7.74	0.57
	BE_10215_ESL	7.00	4.78	-2.22	7.01	0.00	7.73	0.73
	BE_09702_ESL	6.89	4.63	-2.27	6.90	0.00	7.73	0.84
Stainton Lane bridge u/s	BE_09588_ESL!	6.88	4.60	-2.28	6.88	0.00	7.73	0.85
Stainton Lane bridge d/s	BE_09588_ESL	6.86	4.60	-2.26	6.86	0.00	7.73	0.87
	BE_09206_ESL	6.81	4.53	-2.28	6.81	0.00	7.73	0.93
Railway Embankment u/s	BE_08820_ESL	6.73	4.43	-2.30	6.74	0.00	7.73	1.00
Railway Embankment d/s	BE_08775	6.71	4.43	-2.28	6.71	0.00	6.47	-0.25
Stainton Beck confluence	BE_08600	6.69	4.41	-2.29	6.70	0.00	6.46	-0.23
	BE_08300	6.67	4.37	-2.30	6.67	0.00	6.46	-0.22
	BE_08200	6.66	4.36	-2.30	6.66	0.00	6.45	-0.21
	BE_08100	6.65	4.35	-2.30	6.66	0.00	6.44	-0.21
	BE_08000	6.65	4.35	-2.30	6.65	0.00	6.44	-0.21
Bullington Beck confluence	BE_07900	6.64	4.33	-2.31	6.64	0.00	6.43	-0.21
Langworth Bridge u/s	BE_07792	6.52	4.32	-2.20	6.52	0.00	6.33	-0.19
Langworth Bridge d/s	BE_07767	6.46	4.32	-2.14	6.46	0.00	6.29	-0.17
	BE_07700	6.45	4.32	-2.13	6.45	0.00	6.26	-0.18
Langworth gauging station	BE_07700!	6.44	4.32	-2.12	6.45	0.00	6.25	-0.19
	BE_07682d	6.41	4.32	-2.10	6.42	0.00	6.22	-0.20
	BE_07600	6.40	4.31	-2.08	6.40	0.00	6.21	-0.19
	BE_07500	6.37	4.31	-2.06	6.38	0.00	6.18	-0.19
	BE_07000	6.11	4.30	-1.81	6.12	0.00	5.93	-0.18
	BE_06500	5.88	4.29	-1.59	5.88	0.00	5.71	-0.17
Newball Wood House	BE_06000	5.61	4.29	-1.32	5.61	0.00	5.43	-0.17
	BE_05500	5.18	4.28	-0.90	5.18	0.00	5.08	-0.11
	BE_05200	4.94	4.28	-0.66	4.94	0.00	4.88	-0.05
	BE_05000	4.91	4.28	-0.63	4.91	0.00	4.85	-0.06
	BE_04500	4.89	4.28	-0.61	4.89	0.00	4.81	-0.08
	BE_04102	4.63	4.28	-0.35	4.63	0.00	4.55	-0.09
Gate Cliffe Farm bridge	BE_04102!	4.63	4.28	-0.35	4.63	0.00	4.54	-0.08
	BE_04000	4.60	4.28	-0.32	4.60	0.00	4.52	-0.08
	BE_03900	4.52	4.28	-0.24	4.52	0.00	4.46	-0.06
	BE_03600	4.47	4.28	-0.19	4.47	0.00	4.38	-0.09
	BE_03500	4.47	4.28	-0.19	4.47	0.00	4.38	-0.09
	BE_03000	4.47	4.28	-0.19	4.46	0.00	4.38	-0.08
	BE_02900	4.47	4.28	-0.19	4.46	0.00	4.38	-0.08
Barlings Abbey	BE_02800	4.47	4.28	-0.19	4.46	0.00	4.38	-0.08
	BE_02688	4.45	4.28	-0.18	4.45	0.00	4.38	-0.07
	BE_02677	4.46	4.28	-0.18	4.46	0.00	4.38	-0.08
	BE_02500	4.45	4.28	-0.18	4.46	0.00	4.38	-0.07
	BE_02300	4.45	4.28	-0.18	4.46	0.00	4.38	-0.07
	BE_02100	4.45	4.28	-0.18	4.46	0.00	4.38	-0.07
	BE_02000	4.46	4.28	-0.18	4.46	0.00	4.38	-0.08
	BE_01800	4.46	4.28	-0.18	4.46	0.00	4.38	-0.08
Stainfield Beck confluence	BE_01700	4.45	4.28	-0.17	4.45	0.00	4.38	-0.07
	BE_01700!	4.45	4.28	-0.17	4.45	0.00	4.38	-0.07
	BE_01590	4.45	4.28	-0.17	4.45	0.00	4.38	-0.07
	BE_01500	4.45	4.28	-0.17	4.45	0.00	4.38	-0.07
	BE_01000	4.44	4.28	-0.16	4.44	0.00	4.38	-0.06
	BE_00790	4.44	4.28	-0.16	4.44	0.00	4.38	-0.06
	BE_00500	4.44	4.28	-0.16	4.44	0.00	4.38	-0.06
Short Ferry Bridge	BE_00430!	4.44	4.28	-0.16	4.44	0.00	4.38	-0.06
	BE_00430	4.43	4.28	-0.16	4.43	0.00	4.38	-0.05
	BE_00400	4.43	4.28	-0.16	4.43	0.00	4.38	-0.05
	BE_00100	4.43	4.28	-0.15	4.43	0.00	4.38	-0.05
	BE_00100!	4.43	4.28	-0.15	4.43	0.00	4.38	-0.05

Table A4: Barlings Eau PPFS Modelling Results – 1 in 10 year Existing and Options 1 & 2

Location	Model Node	1 in 5 year Water Levels (m AOD) Existing	Existing (spills along embankment)	Change vs Existing	Langworth Option 3 (embankment removed)	Existing (with emb) vs Option 3 (emb removed)
Welton Beck confluence	BE_11714!!_ESL	7.40	7.48	0.09	7.47	-0.01
	BE_11224_ESL	7.26	7.36	0.10	7.35	-0.01
	BE_10272_ESL	7.05	7.13	0.09	7.14	0.01
	BE_10215_ESL	6.89	6.98	0.08	6.96	-0.02
	BE_09702_ESL	6.79	6.87	0.08	6.83	-0.04
Stainton Lane bridge u/s	BE_09588_ESL!	6.77	6.86	0.09	6.80	-0.05
Stainton Lane bridge d/s	BE_09588_ESL	6.75	6.84	0.08	6.78	-0.06
	BE_09206_ESL	6.70	6.78	0.09	6.70	-0.08
Railway Embankment u/s	BE_08820_ESL	6.61	6.71	0.10	6.58	-0.13
Railway Embankment d/s	BE_08775	6.59	6.69	0.10	6.55	-0.14
Stainton Beck confluence	BE_08600	6.57	6.67	0.10	6.53	-0.15
	BE_08300	6.55	6.65	0.10	6.49	-0.16
	BE_08200	6.53	6.64	0.11	6.47	-0.16
	BE_08100	6.52	6.63	0.11	6.46	-0.17
	BE_08000	6.52	6.62	0.11	6.45	-0.17
Bullington Beck confluence	BE_07900	6.51	6.62	0.11	6.44	-0.18
Langworth Bridge u/s	BE_07792	6.40	6.49	0.09	6.30	-0.19
Langworth Bridge d/s	BE_07767	6.36	6.43	0.08	6.26	-0.18
	BE_07700	6.33	6.41	0.09	6.22	-0.20
Langworth gauging station	BE_07700!	6.32	6.41	0.09	6.21	-0.20
	BE_07682d	6.29	6.37	0.09	6.16	-0.22
	BE_07600	6.28	6.36	0.08	6.13	-0.23
	BE_07500	6.25	6.33	0.08	6.09	-0.24
	BE_07000	6.00	6.02	0.02	5.63	-0.39
	BE_06500	5.77	5.87	0.09	5.58	-0.29
Newball Wood House	BE_06000	5.51	5.61	0.09	5.55	-0.06
	BE_05500	5.12	5.18	0.06	5.15	-0.04
	BE_05200	4.91	4.94	0.03	4.92	-0.02
	BE_05000	4.88	4.91	0.04	4.89	-0.02
	BE_04500	4.84	4.89	0.05	4.86	-0.03
	BE_04102	4.57	4.63	0.06	4.59	-0.04
Gate Cliffe Farm bridge	BE_04102!	4.57	4.63	0.06	4.59	-0.04
	BE_04000	4.54	4.60	0.05	4.56	-0.03
	BE_03900	4.48	4.52	0.04	4.50	-0.02
	BE_03600	4.35	4.46	0.11	4.37	-0.09
	BE_03500	4.34	4.46	0.12	4.37	-0.10
	BE_03000	4.34	4.46	0.12	4.36	-0.10
	BE_02900	4.34	4.46	0.12	4.36	-0.10
Barlings Abbey	BE_02800	4.34	4.46	0.12	4.36	-0.10
	BE_02688	4.33	4.45	0.12	4.34	-0.11
	BE_02677	4.33	4.45	0.12	4.35	-0.10
	BE_02500	4.33	4.45	0.11	4.34	-0.10
	BE_02300	4.33	4.45	0.11	4.34	-0.11
	BE_02100	4.33	4.45	0.11	4.34	-0.11
	BE_02000	4.33	4.45	0.12	4.34	-0.11
	BE_01800	4.33	4.45	0.12	4.34	-0.11
Stainfield Beck confluence	BE_01700	4.33	4.45	0.11	4.34	-0.11
	BE_01700!	4.33	4.45	0.11	4.34	-0.11
	BE_01590	4.33	4.44	0.11	4.34	-0.11
	BE_01500	4.33	4.44	0.11	4.34	-0.11
	BE_01000	4.33	4.44	0.11	4.33	-0.11
	BE_00790	4.33	4.44	0.11	4.33	-0.11
	BE_00500	4.33	4.43	0.11	4.33	-0.11
Short Ferry Bridge	BE_00430!	4.33	4.43	0.11	4.32	-0.11
	BE_00430	4.33	4.43	0.10	4.32	-0.11
	BE_00400	4.33	4.43	0.10	4.32	-0.11
	BE_00100	4.33	4.43	0.10	4.32	-0.11
	BE_00100!	4.33	4.43	0.10	4.32	-0.11

Table A5: Barlings Eau PPFS Modelling Results – 1 in 5 year Existing and Option 3

Location	Model Node	1 in 10 year Water Levels (m AOD)				
		Existing	Existing (spills along embankment)	Change vs Existing	Langworth Option 3 (embankment removed)	Existing (with emb) vs Option 3 (emb removed)
Welton Beck confluence	BE_11714!!_ESL	7.51	7.48	-0.03	7.47	-0.01
	BE_11224_ESL	7.40	7.36	-0.04	7.35	-0.01
	BE_10272_ESL	7.17	7.13	-0.03	7.12	-0.02
	BE_10215_ESL	7.00	6.98	-0.03	6.95	-0.03
	BE_09702_ESL	6.89	6.87	-0.02	6.84	-0.03
Stainton Lane bridge u/s	BE_09588_ESL!	6.88	6.86	-0.02	6.82	-0.04
Stainton Lane bridge d/s	BE_09588_ESL	6.86	6.84	-0.02	6.80	-0.04
	BE_09206_ESL	6.81	6.78	-0.02	6.74	-0.04
Railway Embankment u/s	BE_08820_ESL	6.73	6.71	-0.02	6.66	-0.06
Railway Embankment d/s	BE_08775	6.71	6.69	-0.02	6.63	-0.06
Stainton Beck confluence	BE_08600	6.69	6.67	-0.02	6.61	-0.06
	BE_08300	6.67	6.65	-0.02	6.59	-0.06
	BE_08200	6.66	6.64	-0.02	6.57	-0.07
	BE_08100	6.65	6.63	-0.02	6.56	-0.07
	BE_08000	6.65	6.62	-0.02	6.55	-0.07
Bullington Beck confluence	BE_07900	6.64	6.62	-0.02	6.54	-0.07
Langworth Bridge u/s	BE_07792	6.52	6.49	-0.03	6.40	-0.09
Langworth Bridge d/s	BE_07767	6.46	6.43	-0.02	6.34	-0.09
	BE_07700	6.45	6.41	-0.03	6.30	-0.11
Langworth gauging station	BE_07700!	6.44	6.41	-0.03	6.29	-0.12
	BE_07682d	6.41	6.37	-0.04	6.24	-0.14
	BE_07600	6.40	6.36	-0.04	6.22	-0.14
	BE_07500	6.37	6.33	-0.04	6.18	-0.15
	BE_07000	6.11	6.02	-0.09	5.67	-0.34
	BE_06500	5.88	5.87	-0.01	5.63	-0.24
Newball Wood House	BE_06000	5.61	5.61	0.00	5.61	0.01
	BE_05500	5.18	5.18	0.00	5.19	0.00
	BE_05200	4.94	4.94	0.00	4.94	0.00
	BE_05000	4.91	4.91	0.00	4.92	0.00
	BE_04500	4.89	4.89	0.00	4.90	0.00
	BE_04102	4.63	4.63	0.00	4.63	0.00
Gate Cliffe Farm bridge	BE_04102!	4.63	4.63	0.00	4.63	0.00
	BE_04000	4.60	4.60	0.00	4.60	0.00
	BE_03900	4.52	4.52	0.00	4.52	0.00
	BE_03600	4.47	4.46	-0.01	4.46	-0.01
	BE_03500	4.47	4.46	-0.01	4.46	-0.01
	BE_03000	4.47	4.46	-0.01	4.45	-0.01
	BE_02900	4.47	4.46	-0.01	4.45	-0.01
Barlings Abbey	BE_02800	4.47	4.46	-0.01	4.45	-0.01
	BE_02688	4.45	4.45	0.00	4.45	0.00
	BE_02677	4.46	4.45	-0.01	4.45	0.00
	BE_02500	4.45	4.45	-0.01	4.45	0.00
	BE_02300	4.45	4.45	-0.01	4.44	0.00
	BE_02100	4.45	4.45	-0.01	4.44	0.00
	BE_02000	4.46	4.45	-0.01	4.45	0.00
	BE_01800	4.46	4.45	-0.01	4.45	0.00
Stainfield Beck confluence	BE_01700	4.45	4.45	0.00	4.44	0.00
	BE_01700!	4.45	4.45	0.00	4.44	0.00
	BE_01590	4.45	4.44	0.00	4.44	0.00
	BE_01500	4.45	4.44	-0.01	4.44	0.00
	BE_01000	4.44	4.44	0.00	4.43	0.00
	BE_00790	4.44	4.44	0.00	4.43	0.00
	BE_00500	4.44	4.43	0.00	4.43	0.00
Short Ferry Bridge	BE_00430!	4.44	4.43	0.00	4.43	0.00
	BE_00430	4.43	4.43	0.00	4.43	0.00
	BE_00400	4.43	4.43	0.00	4.43	0.00
	BE_00100	4.43	4.43	0.00	4.43	0.00
	BE_00100!	4.43	4.43	0.00	4.43	0.00

Table A6: Barlings Eau PPFS Modelling Results – 1 in 10 year Existing and Option 3

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Appendix B - Lower Witham Peak Flows - Defended

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Appendix C - Lower Witham Peak Water Levels - Defended

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Appendix D - Lower Witham Long Sections - Defended

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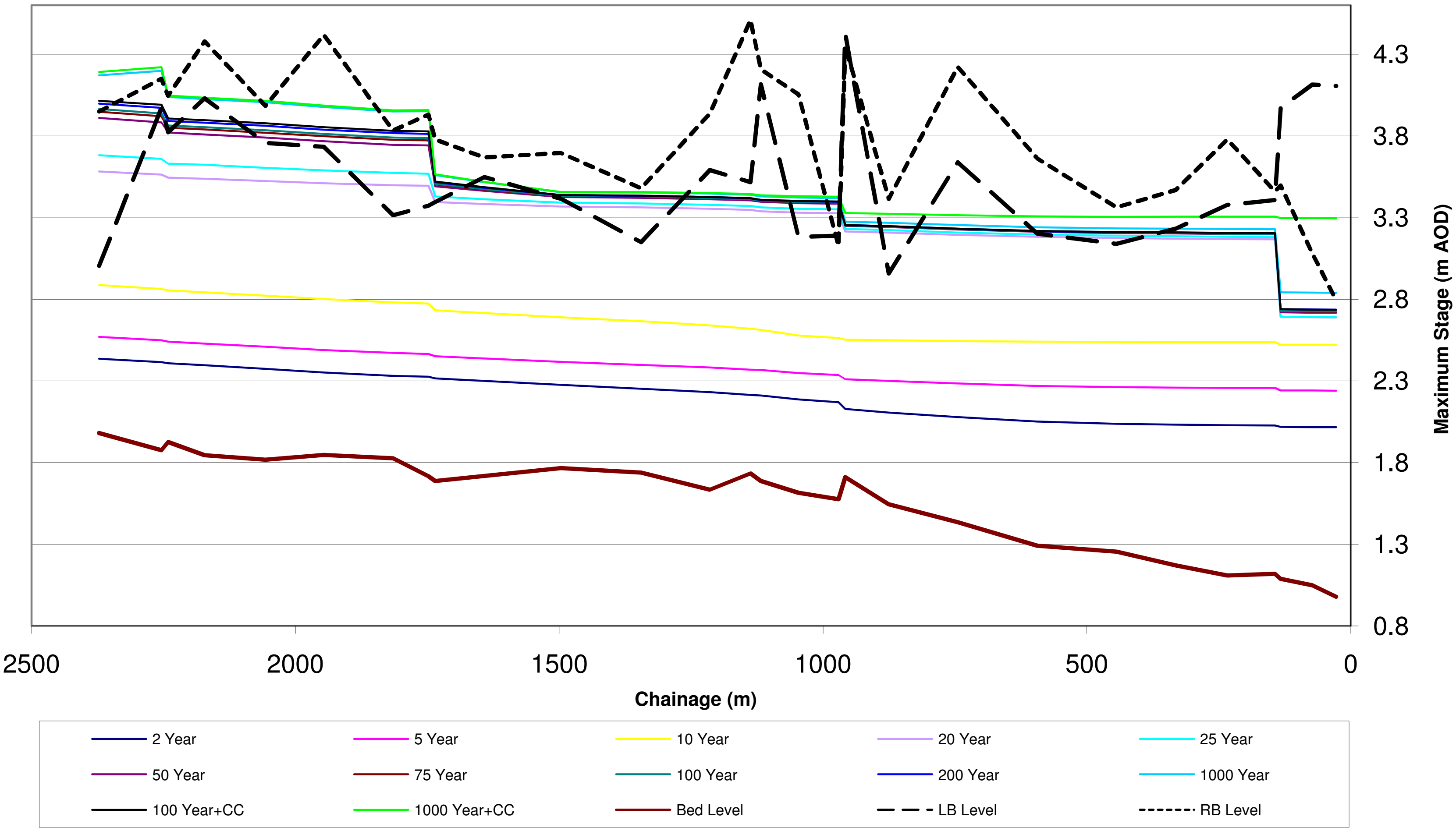
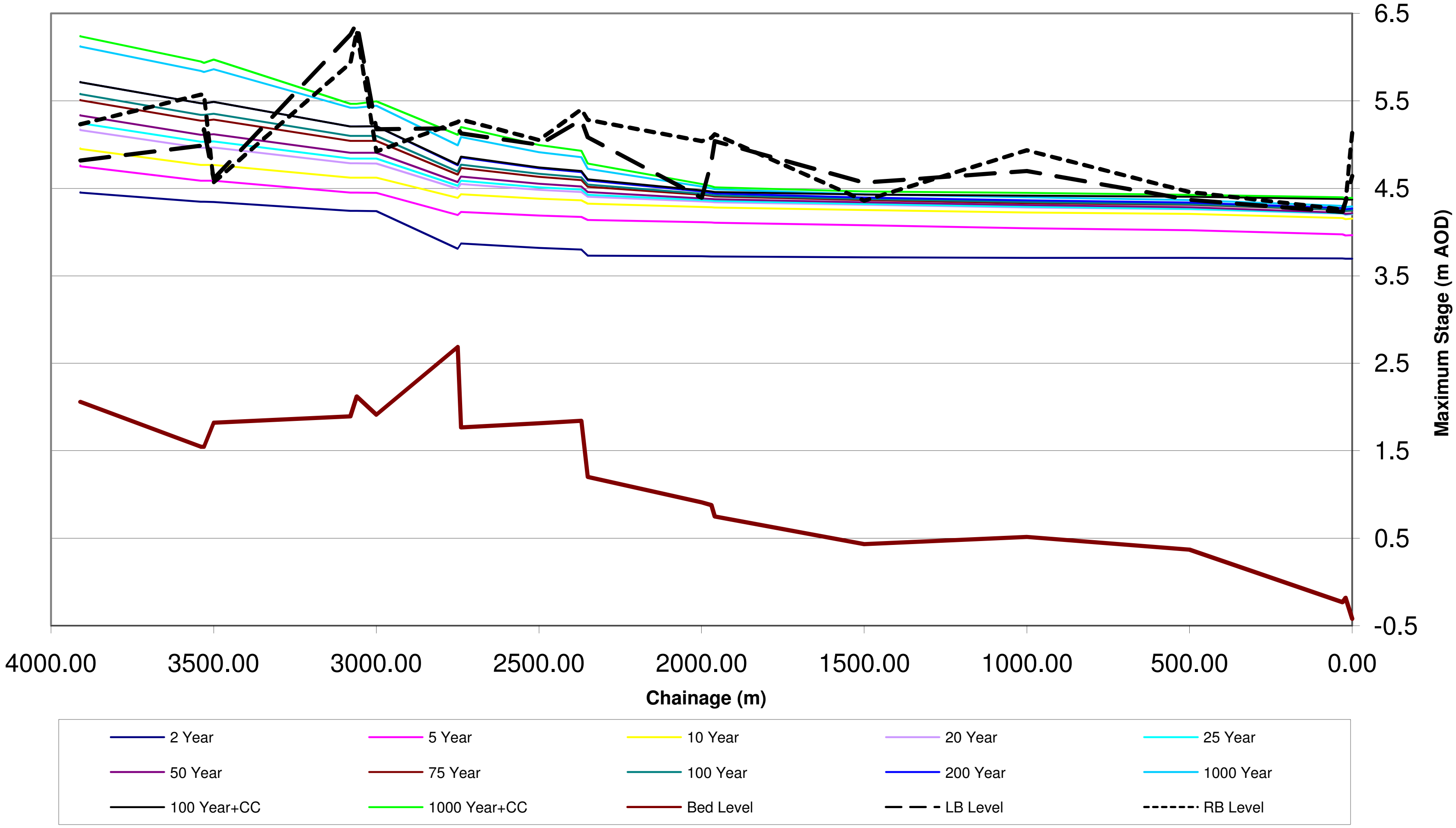
Lower Witham Catchment Long Sections

Figure D1:- Anwick Fen Drain- Max Stage

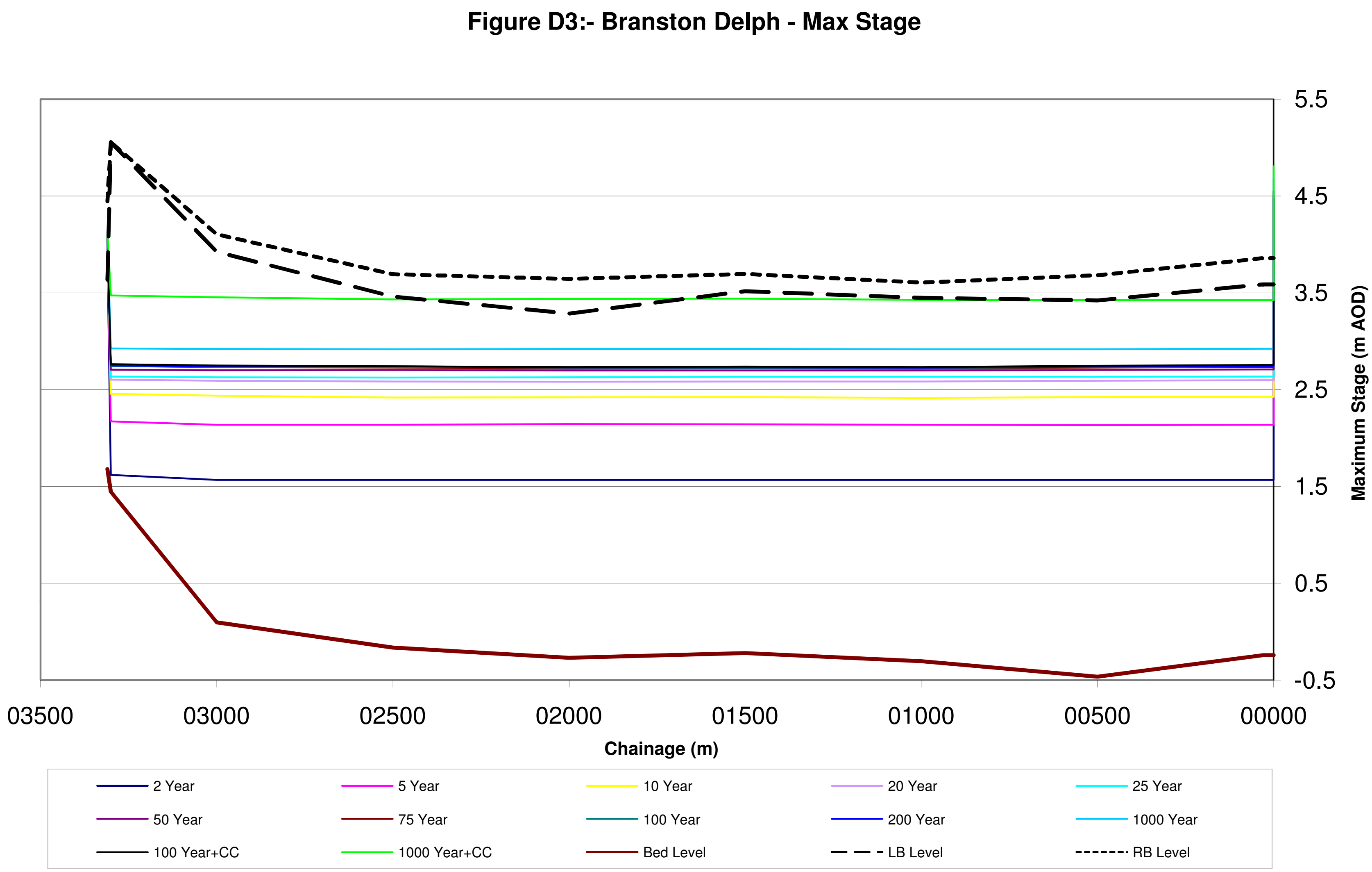


Lower Witham Catchment Long Sections

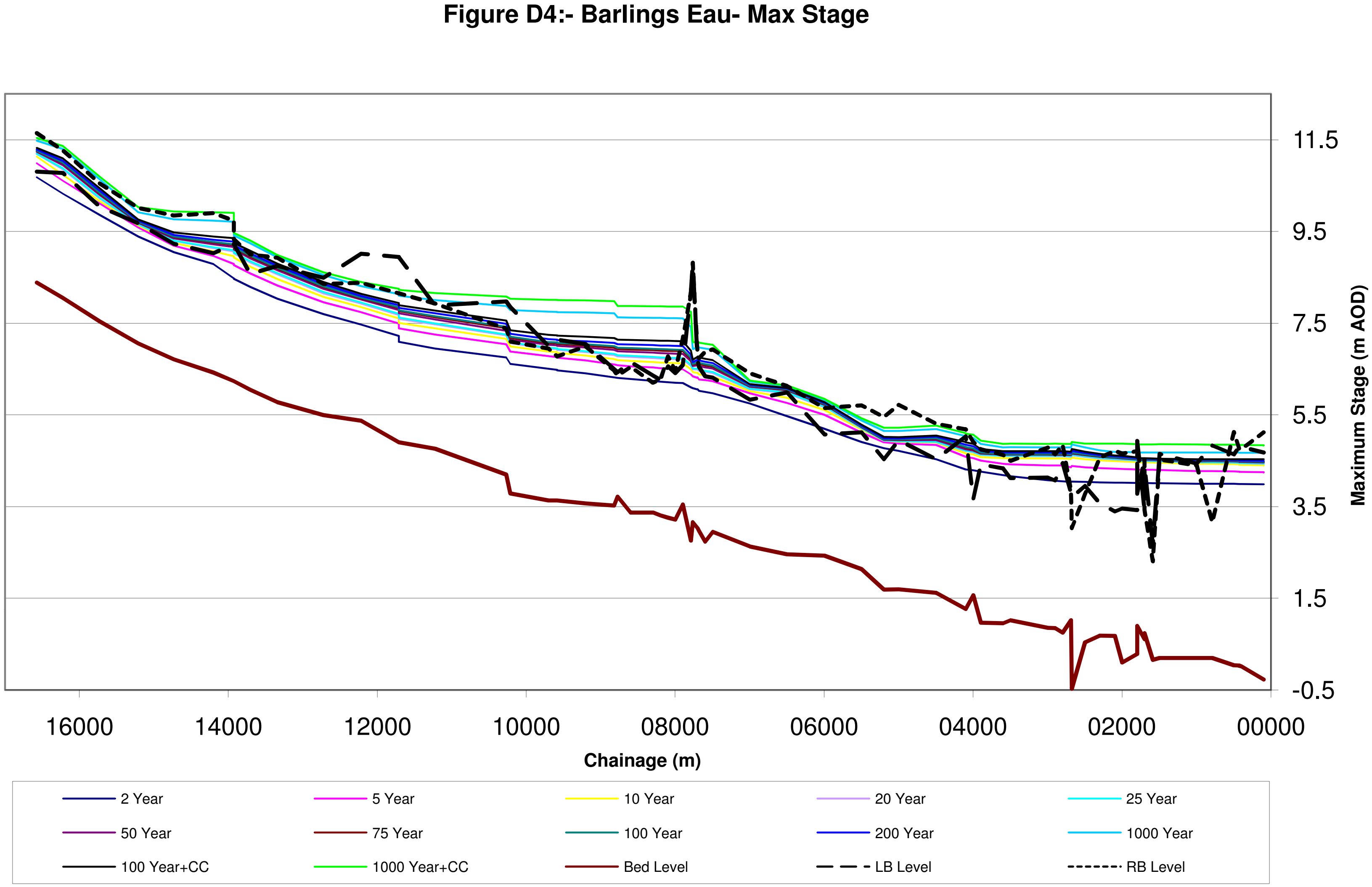
Figure D2:- River Bain - Max Stage



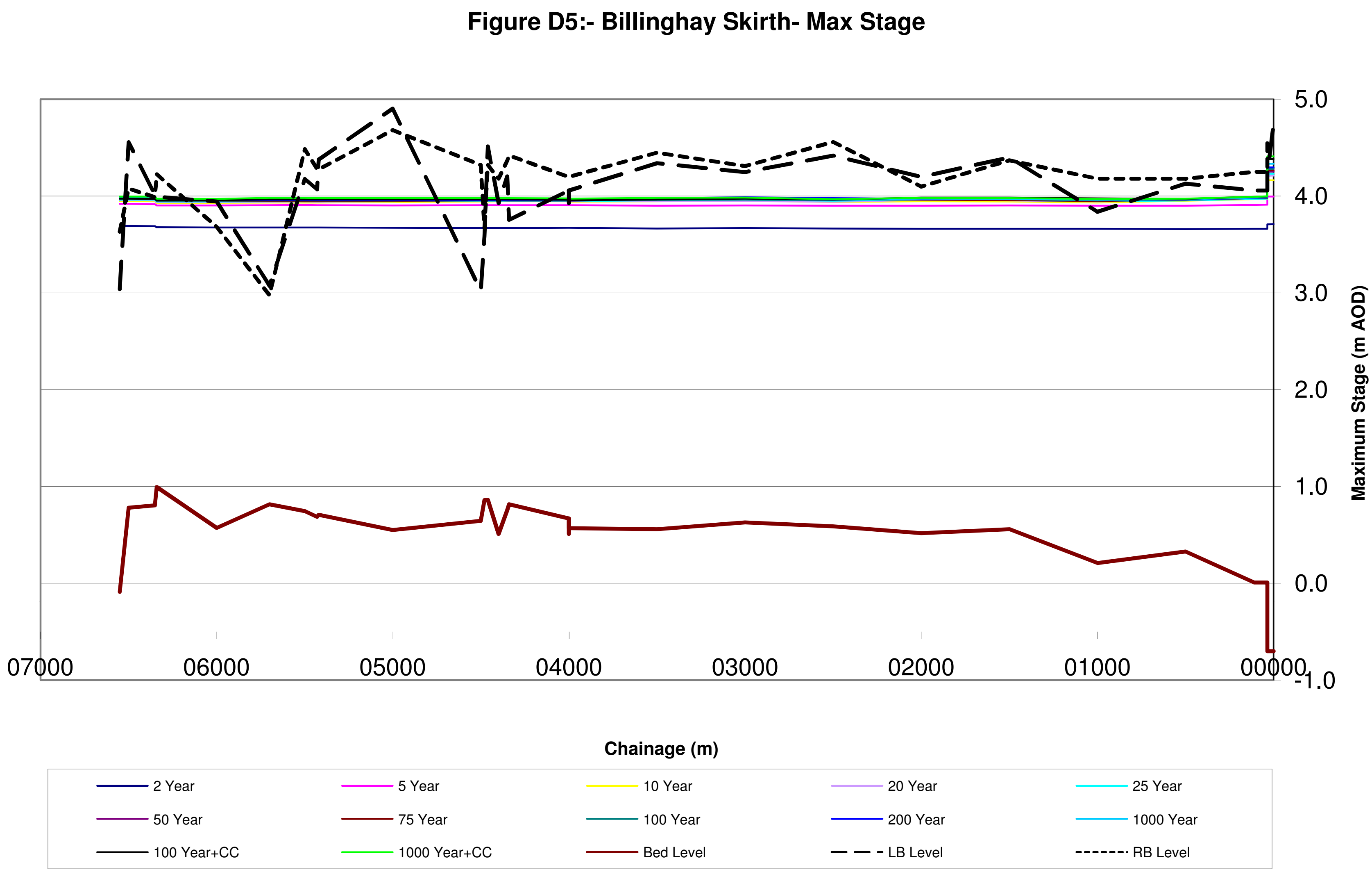
Lower Witham Catchment Long Sections



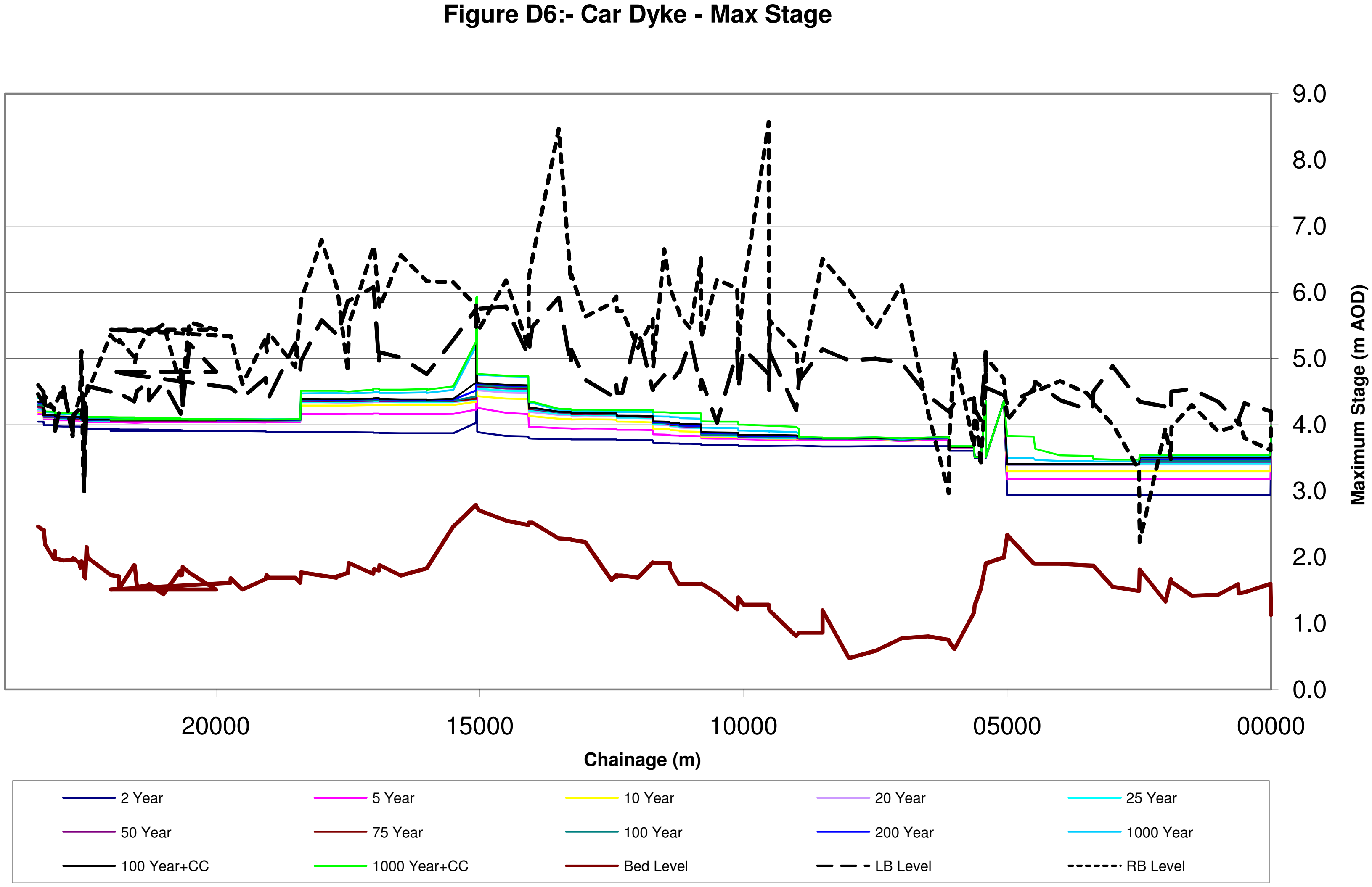
Lower Witham Catchment Long Sections



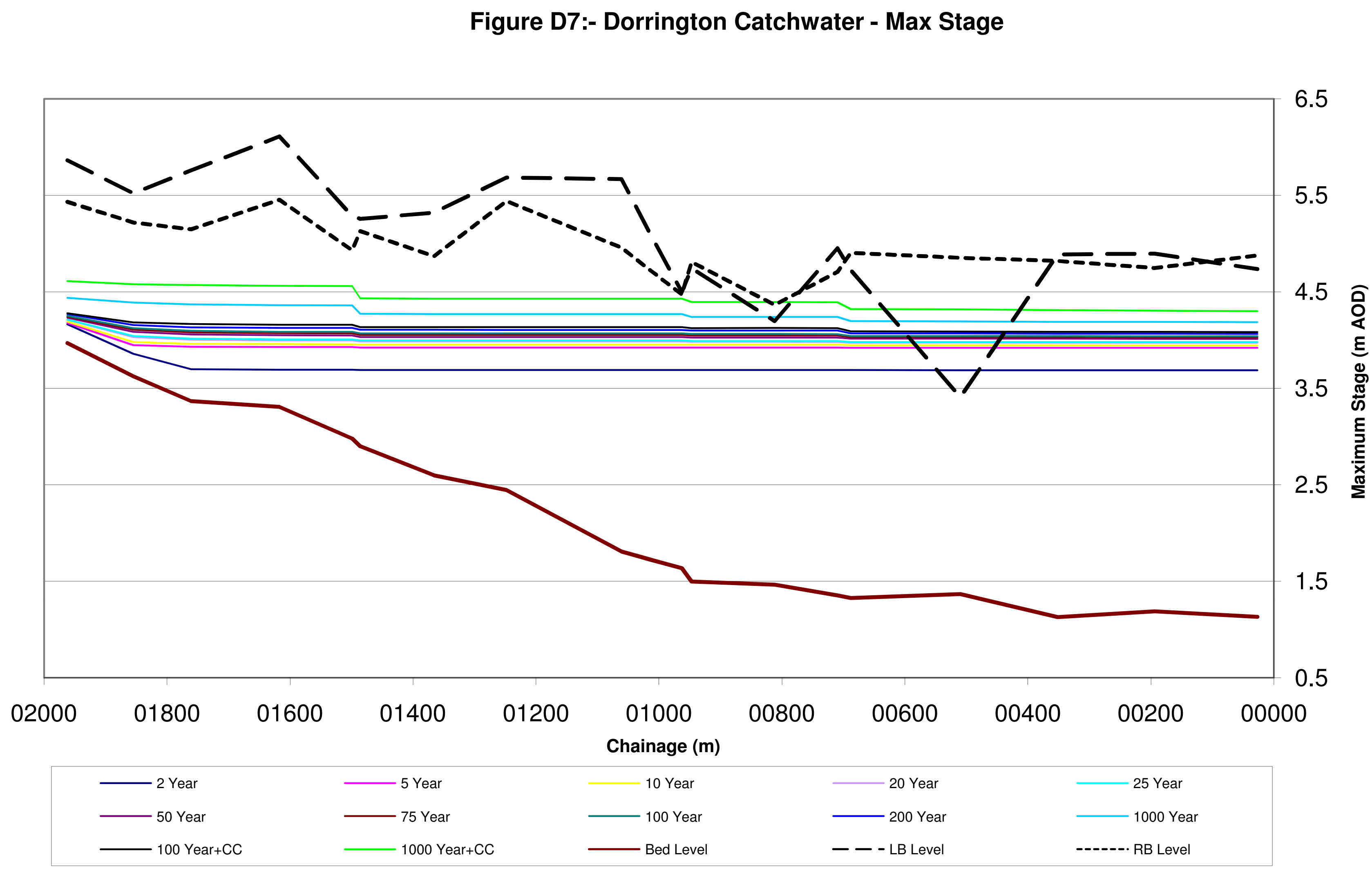
Lower Witham Catchment Long Sections



Lower Witham Catchment Long Sections

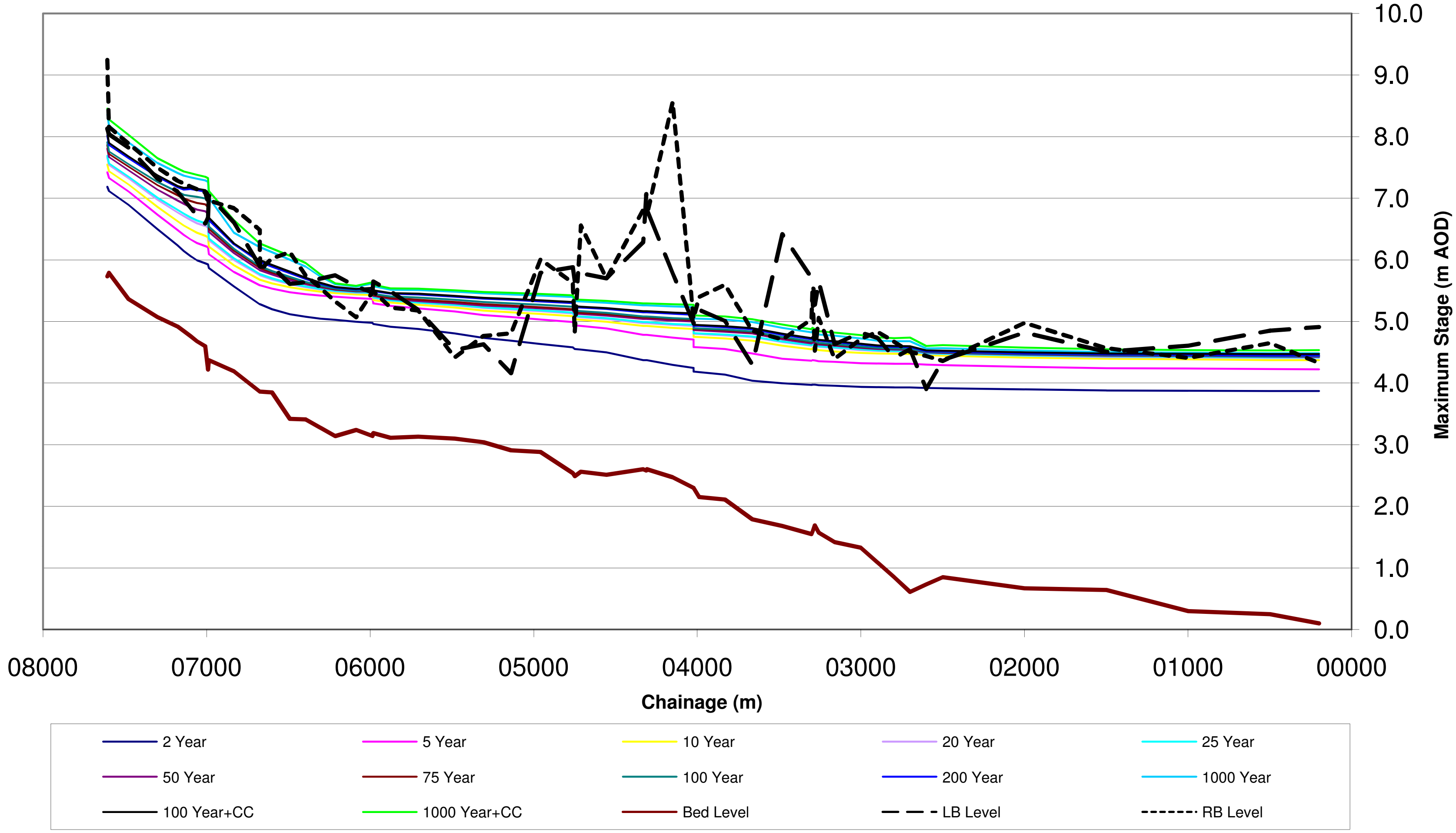


Lower Witham Catchment Long Sections

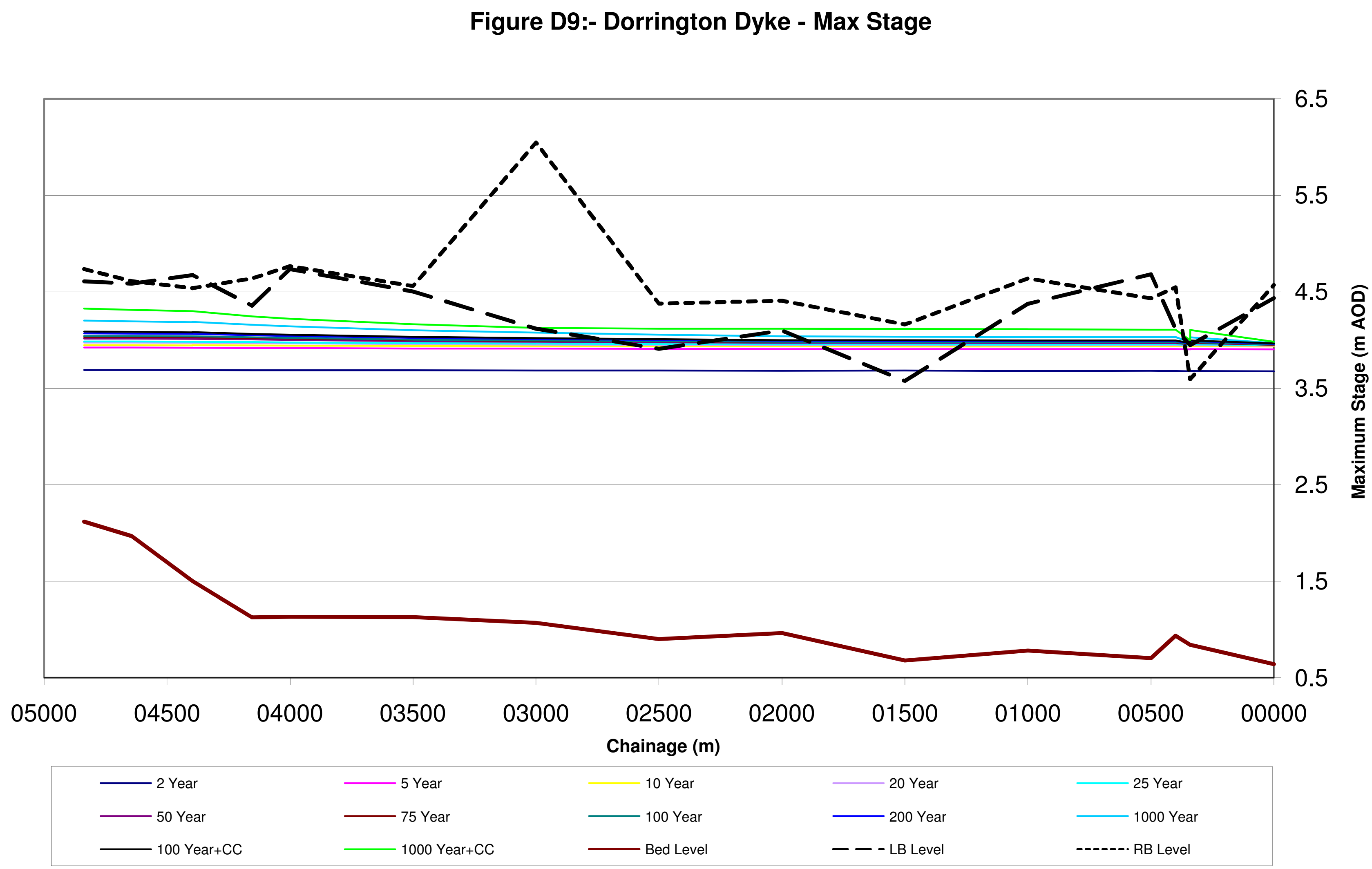


Lower Witham Catchment Long Sections

Figure D8:- Duckpool Drain - Max Stage

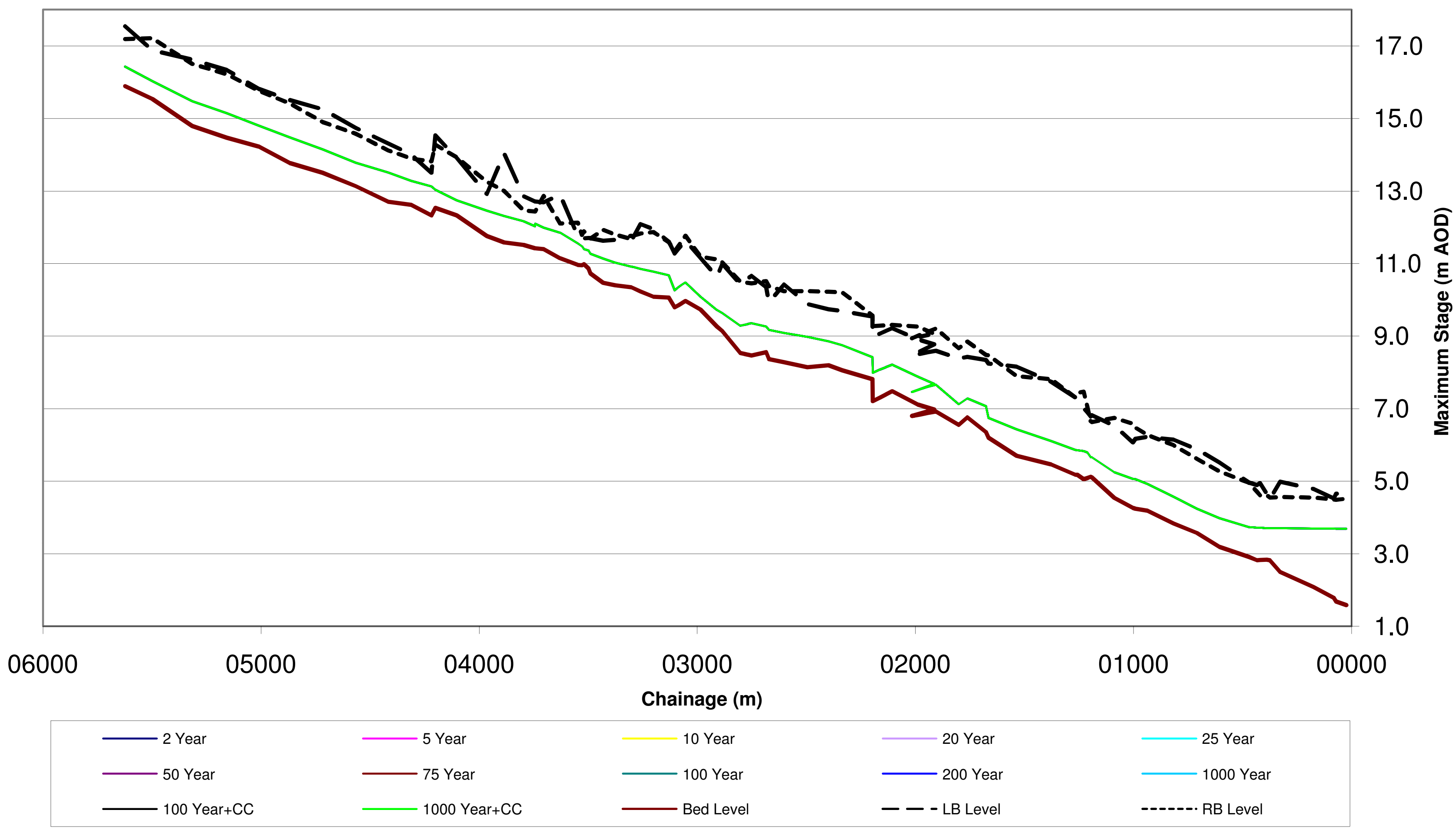


Lower Witham Catchment Long Sections



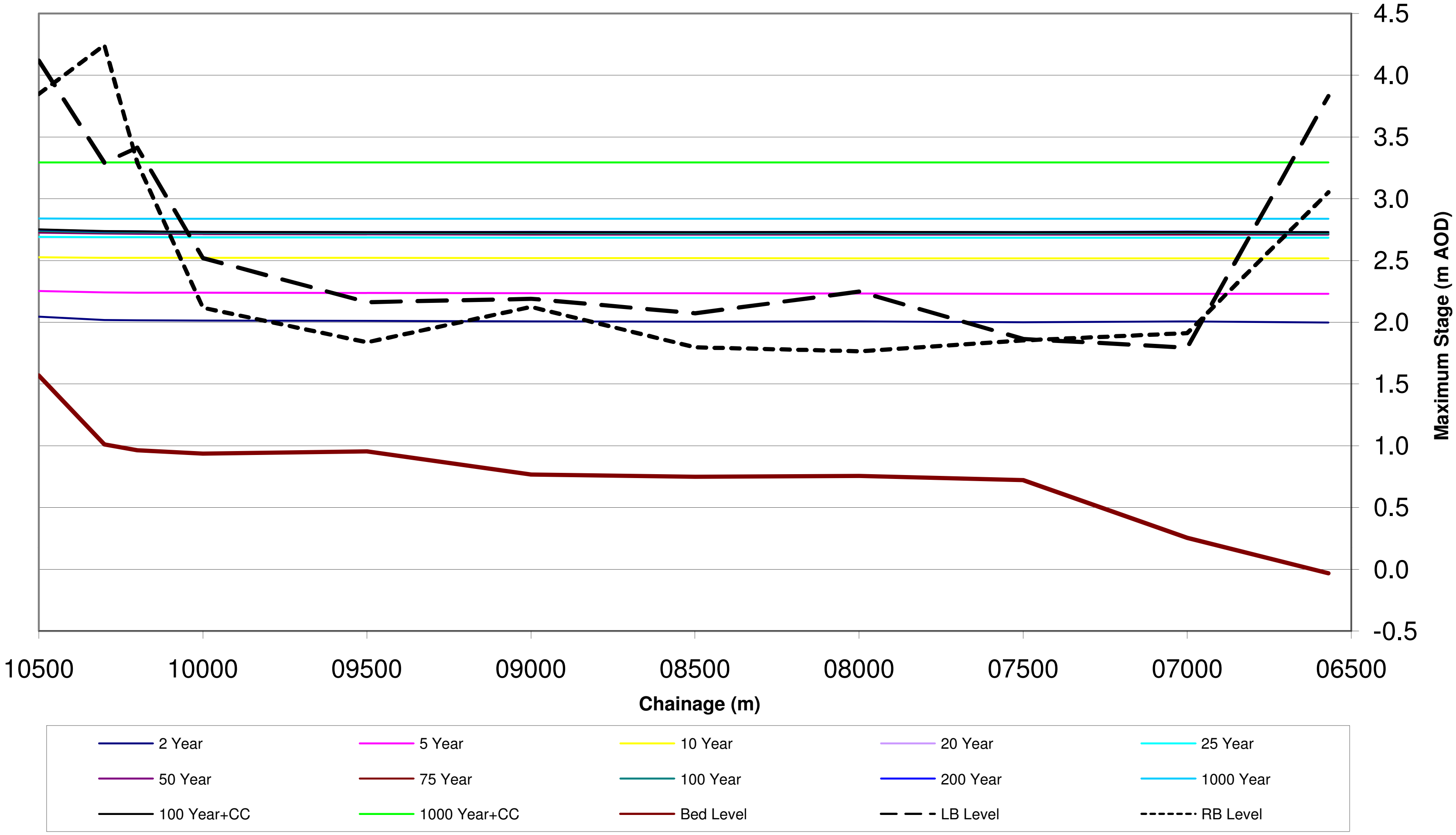
Lower Witham Catchment Long Sections

Figure D10:- Digby Beck - Max Stage

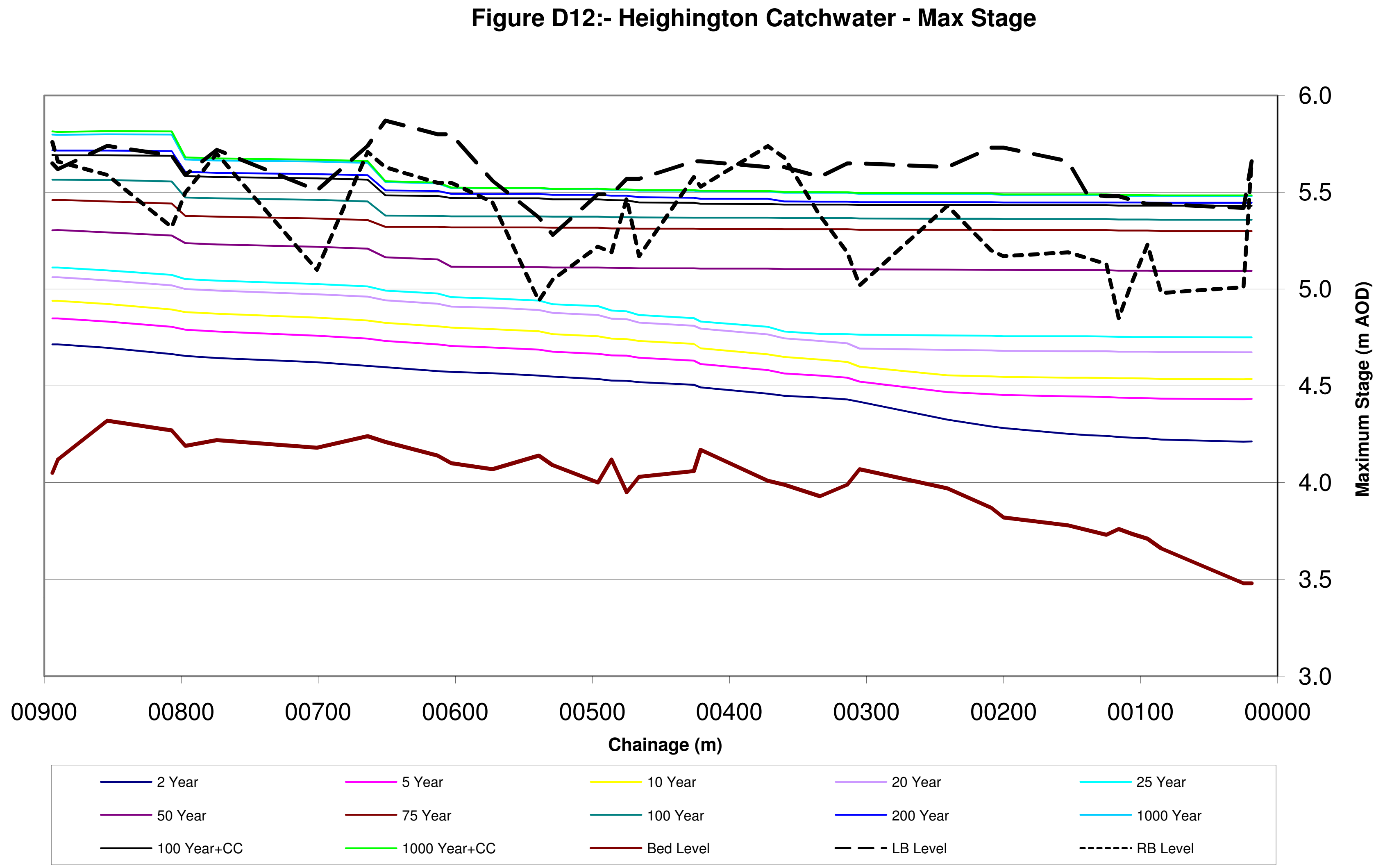


Lower Witham Catchment Long Sections

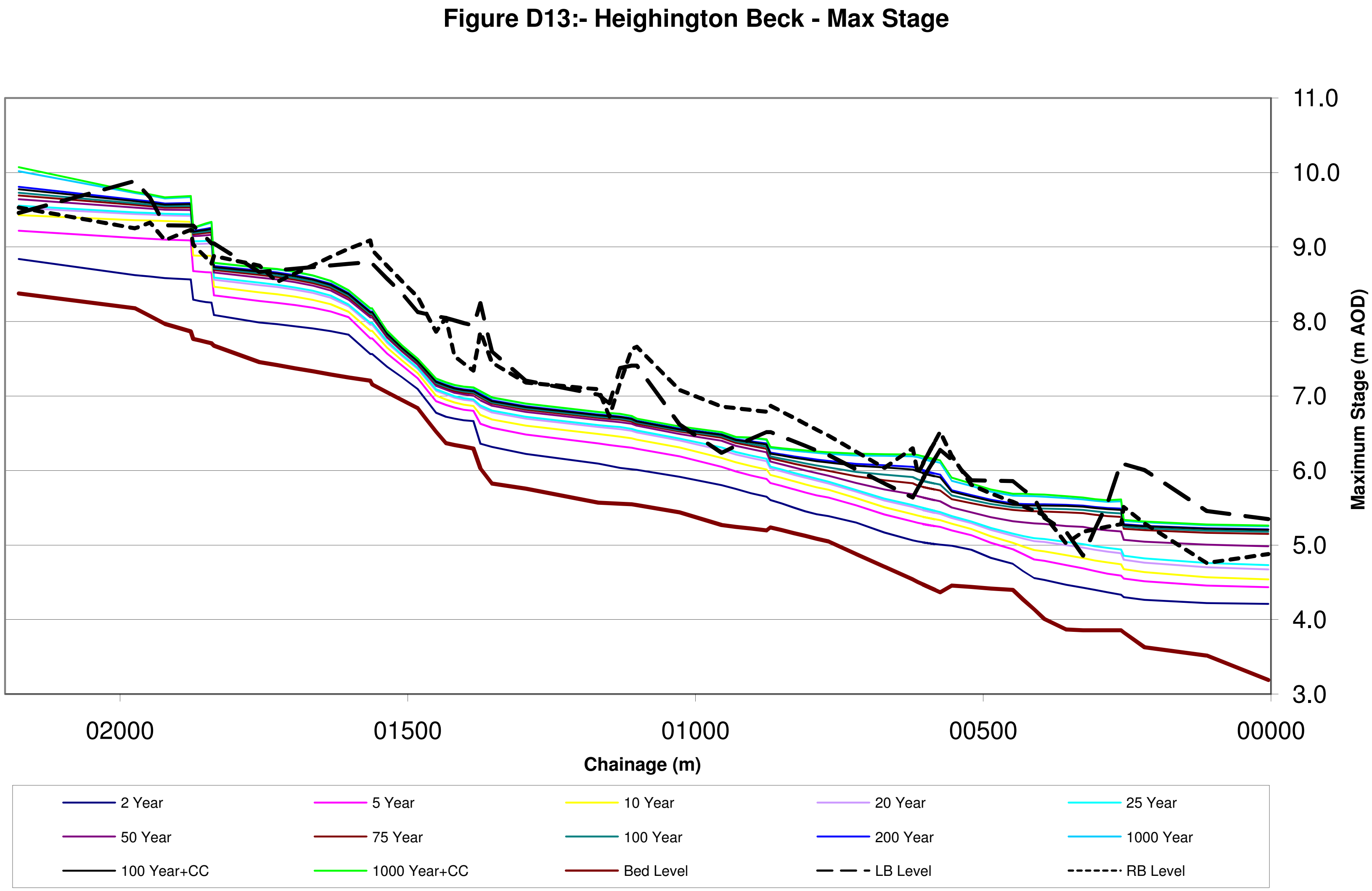
Figure D11:- Farroway Drain - Max Stage



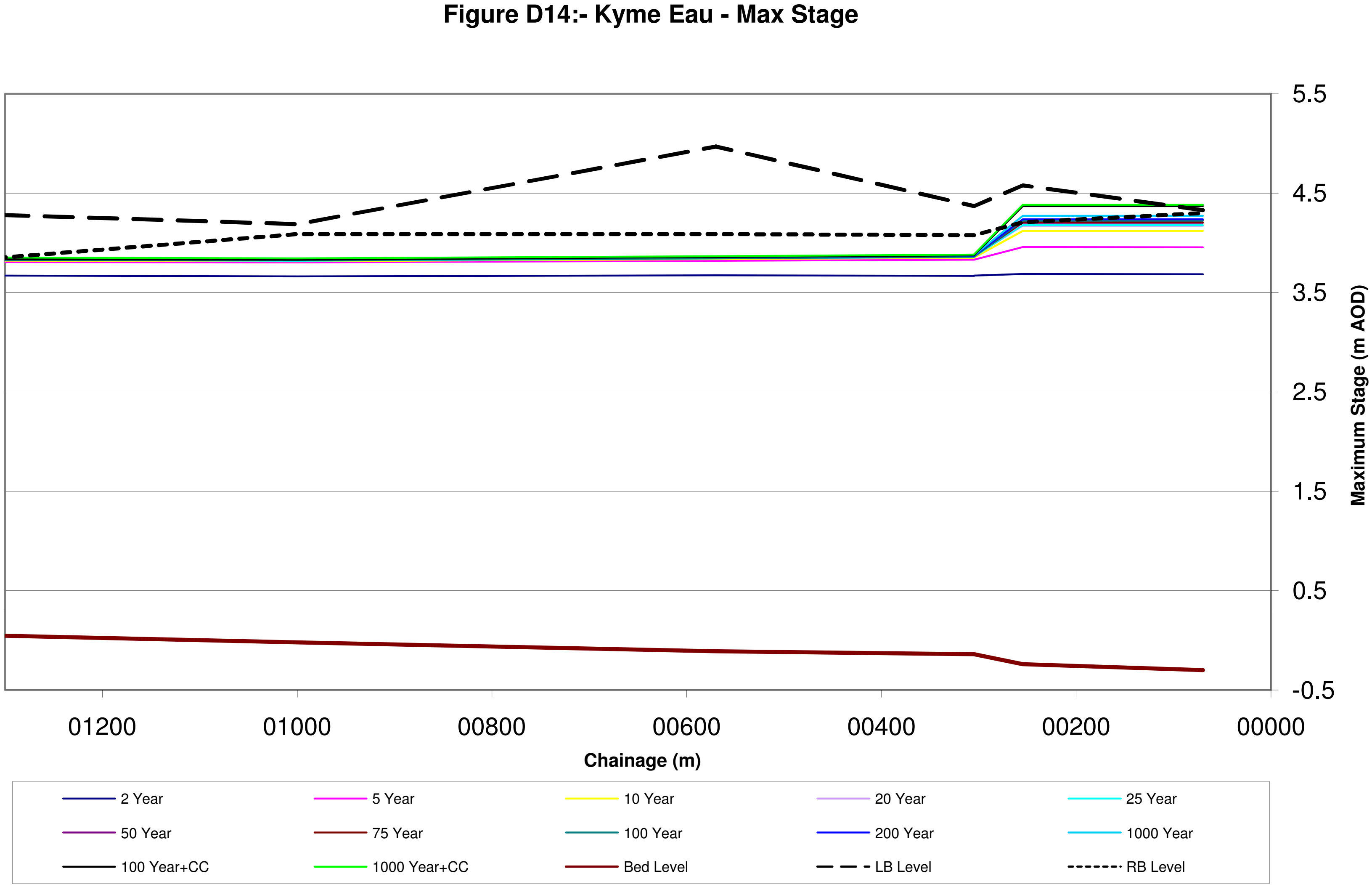
Lower Witham Catchment Long Sections



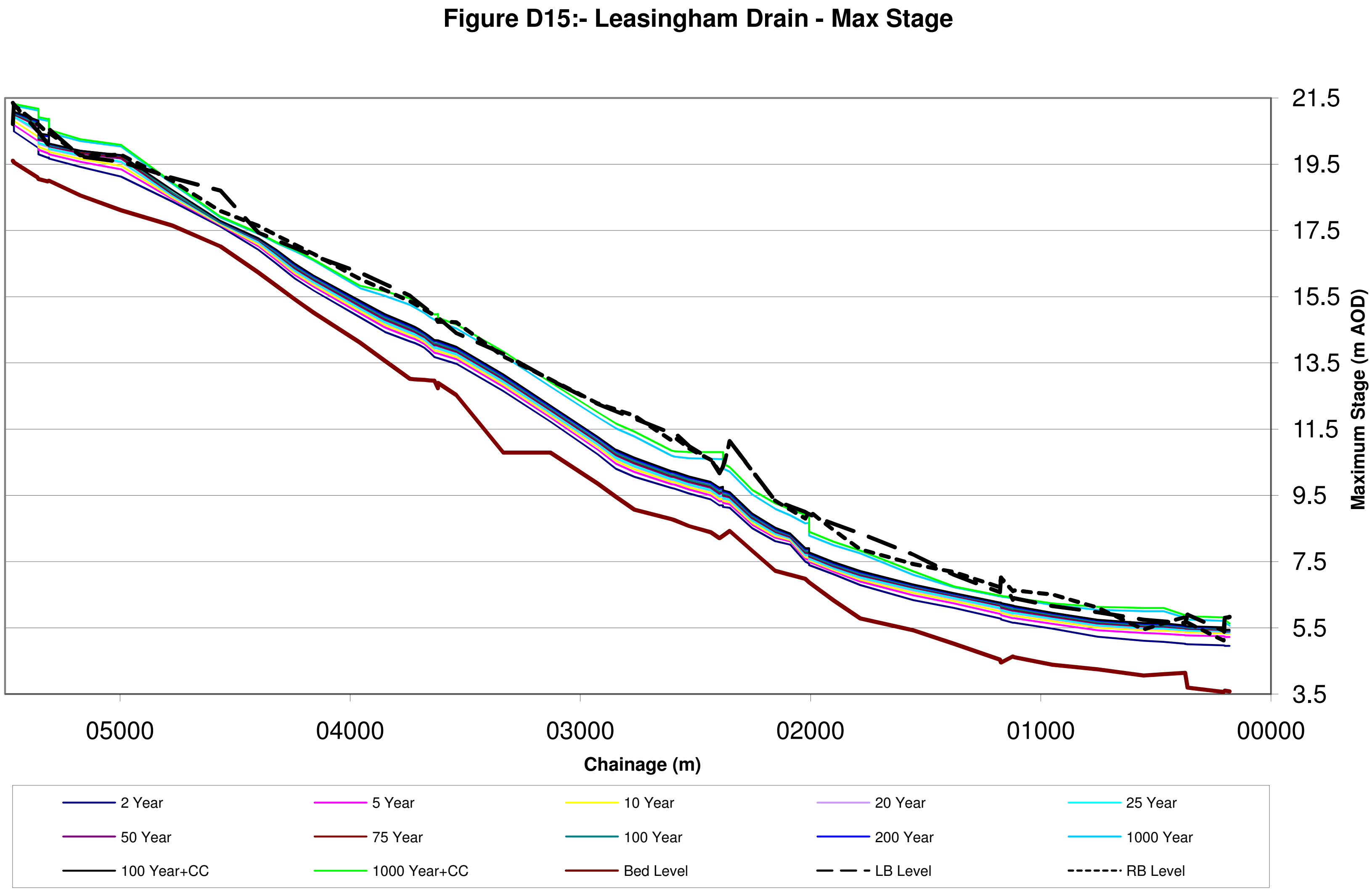
Lower Witham Catchment Long Sections



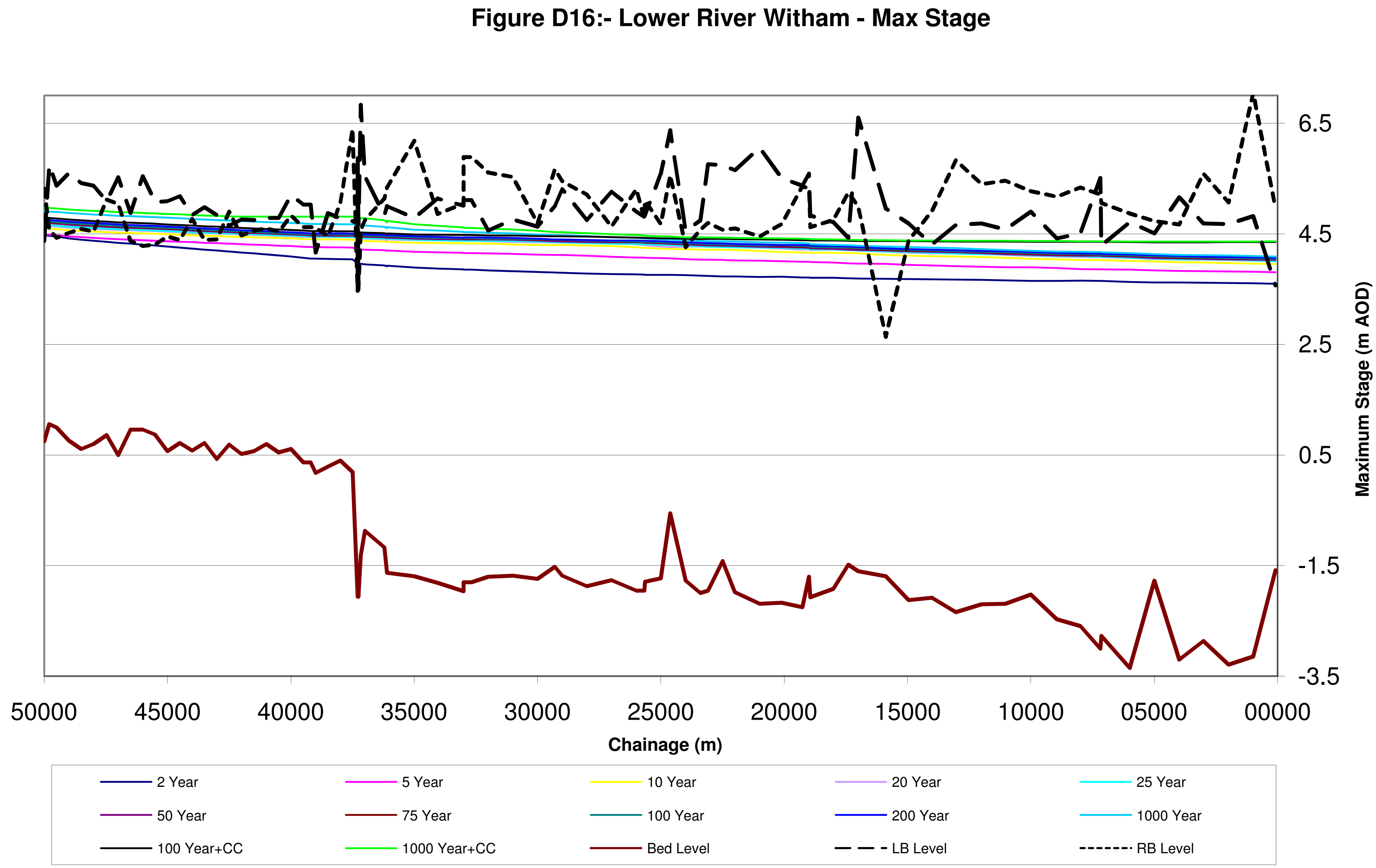
Lower Witham Catchment Long Sections



Lower Witham Catchment Long Sections

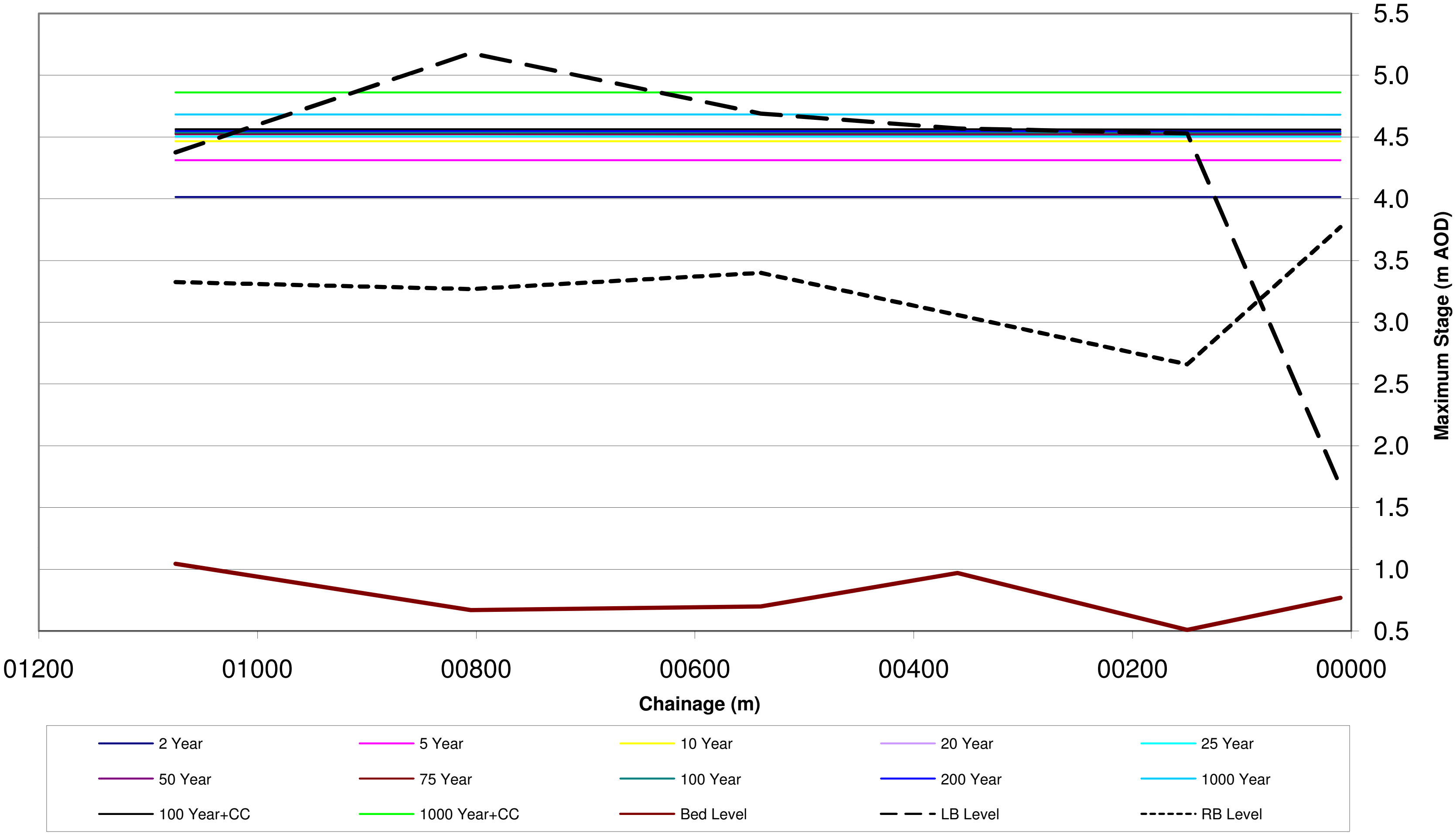


Lower Witham Catchment Long Sections

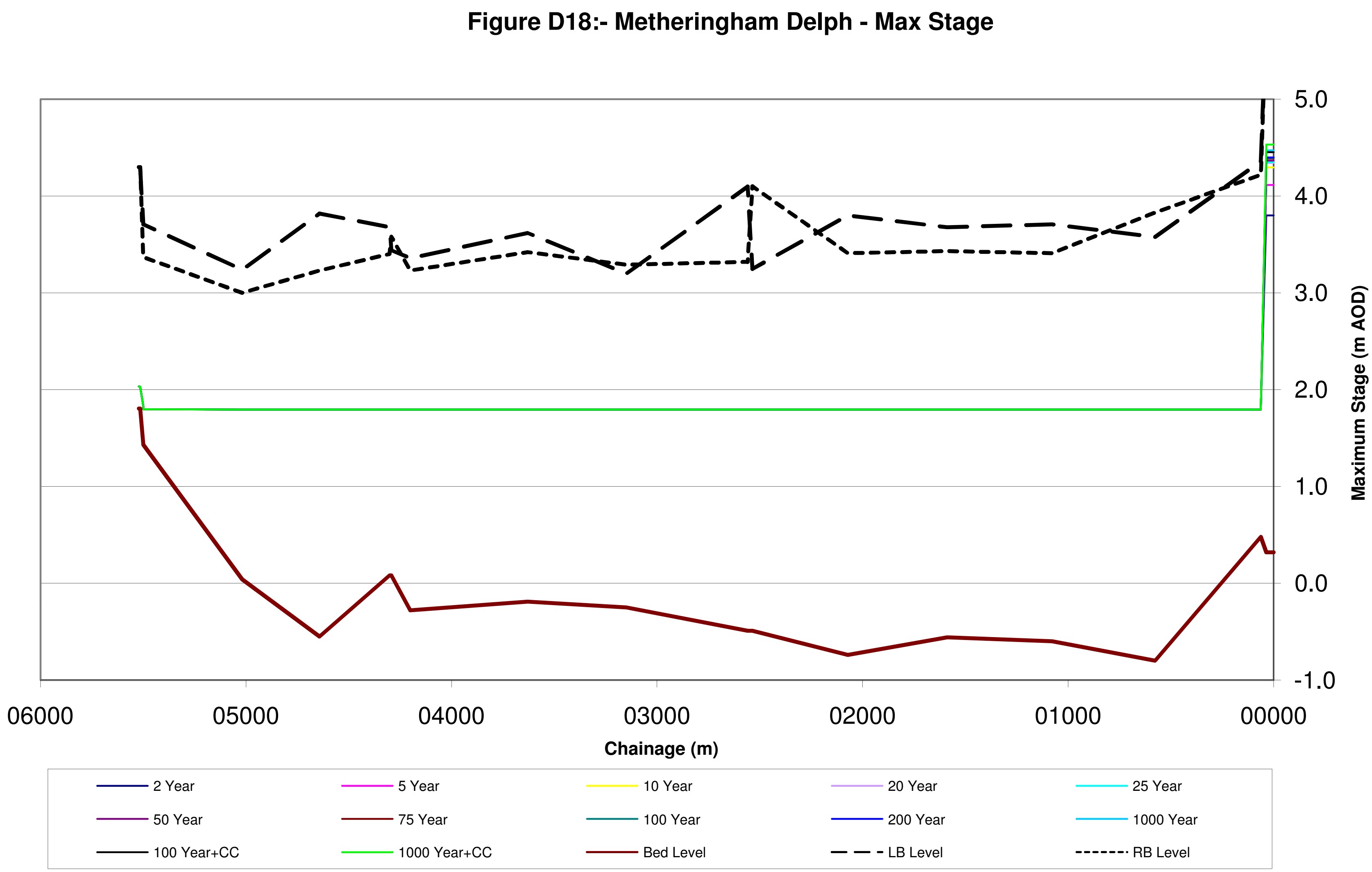


Lower Witham Catchment Long Sections

Figure D17:- Marsh Drain - Max Stage

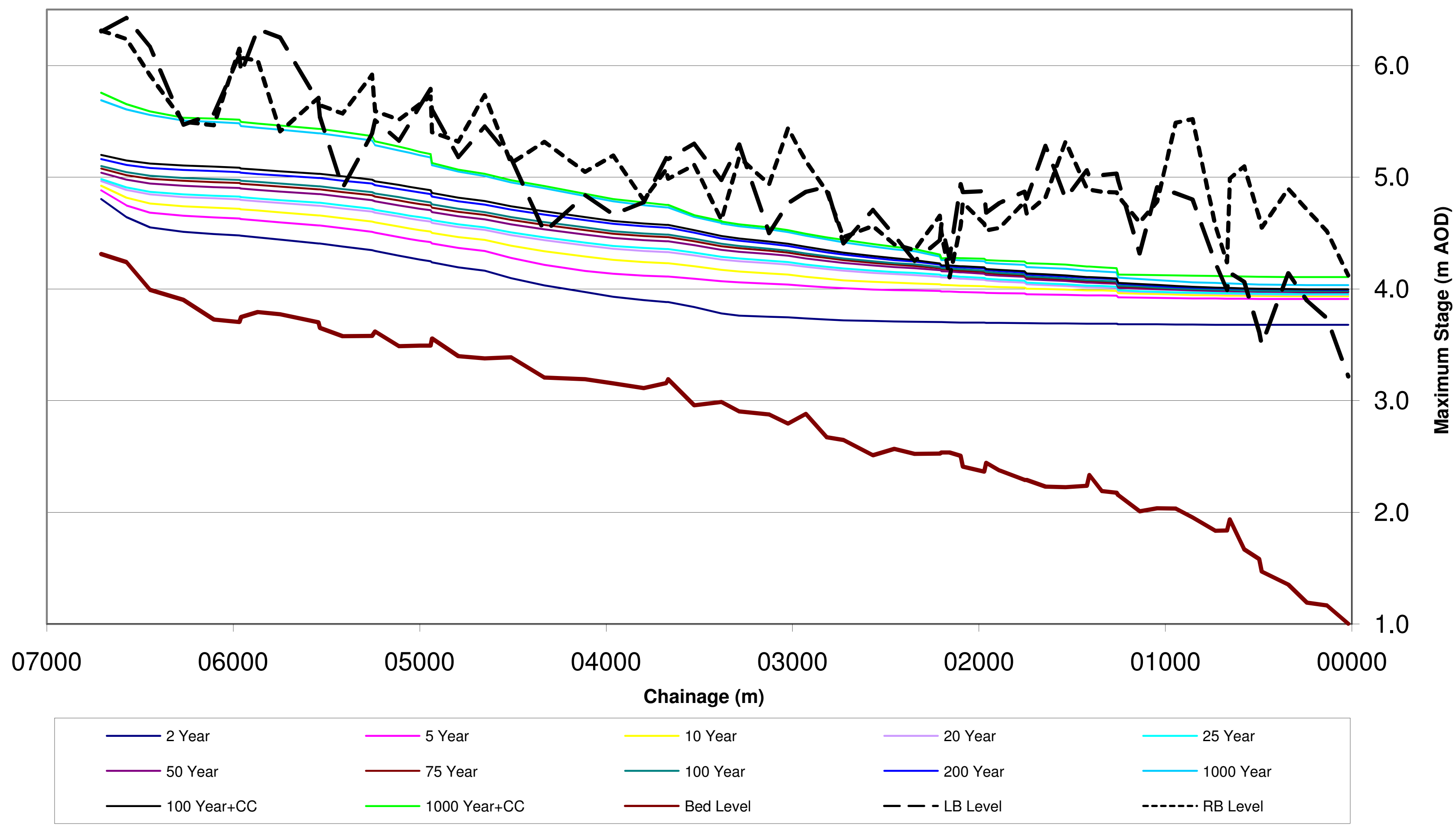


Lower Witham Catchment Long Sections

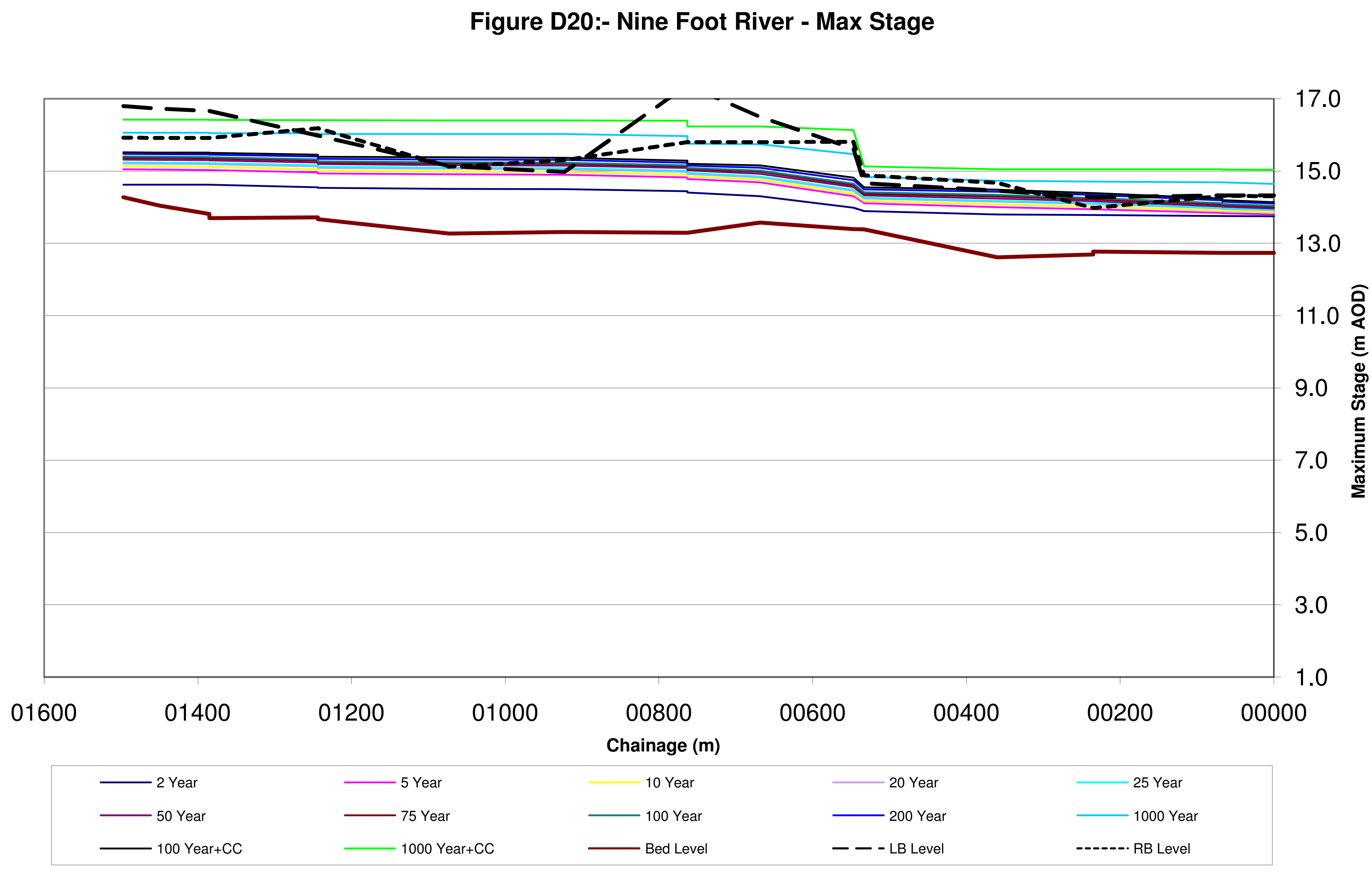


Lower Witham Catchment Long Sections

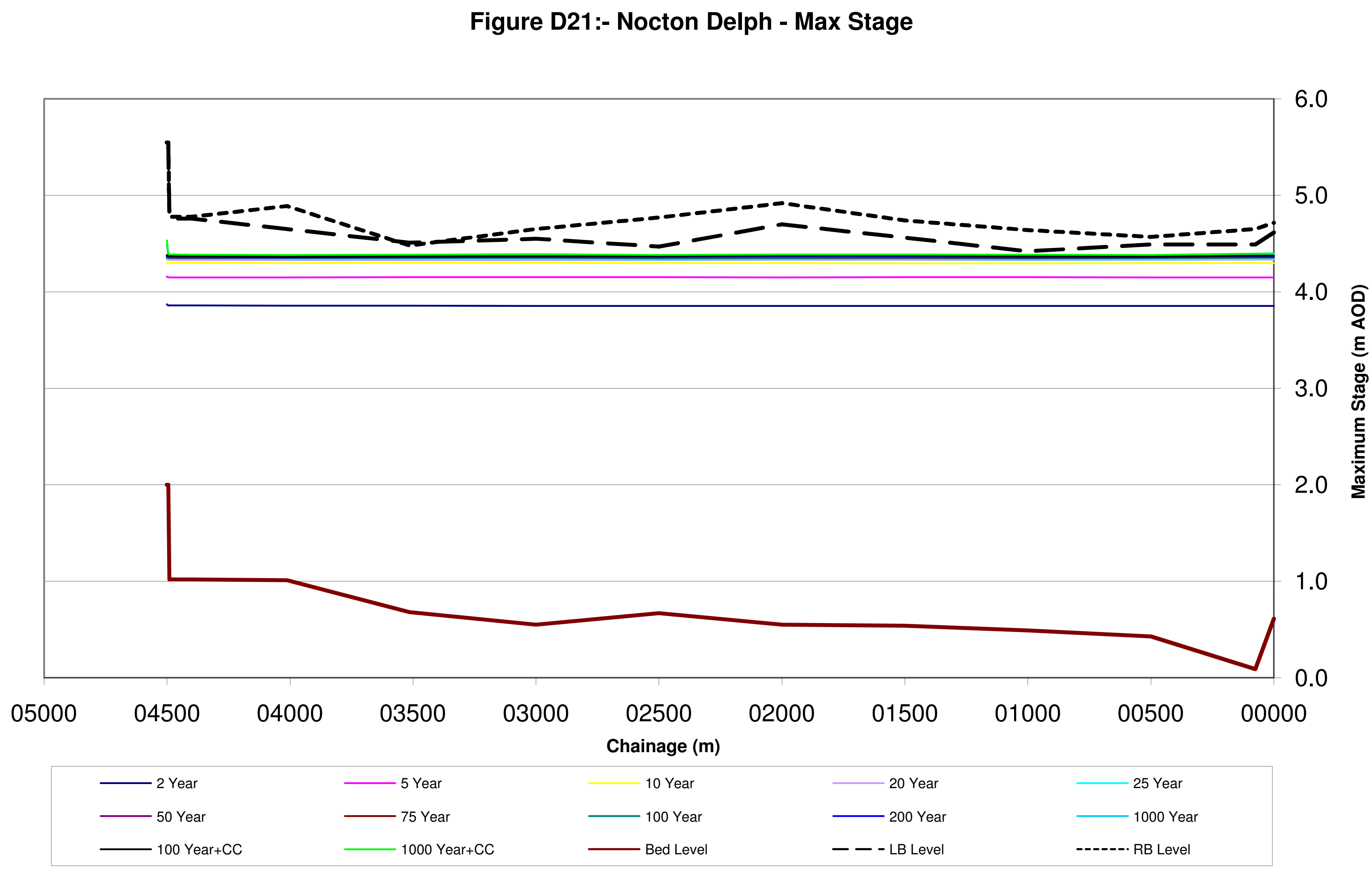
Figure D19:- New Cut - Max Stage



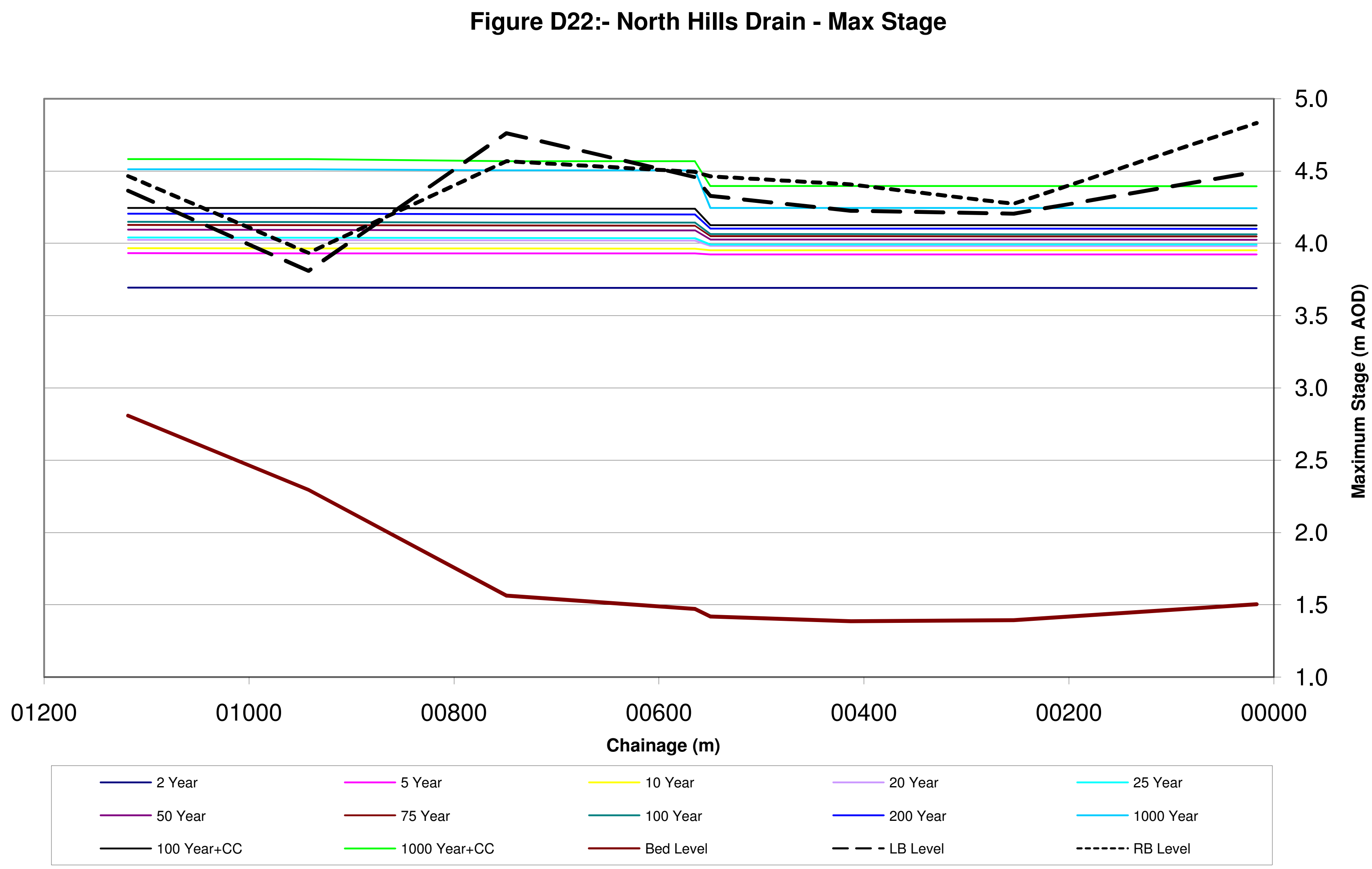
Lower Witham Catchment Long Sections



Lower Witham Catchment Long Sections

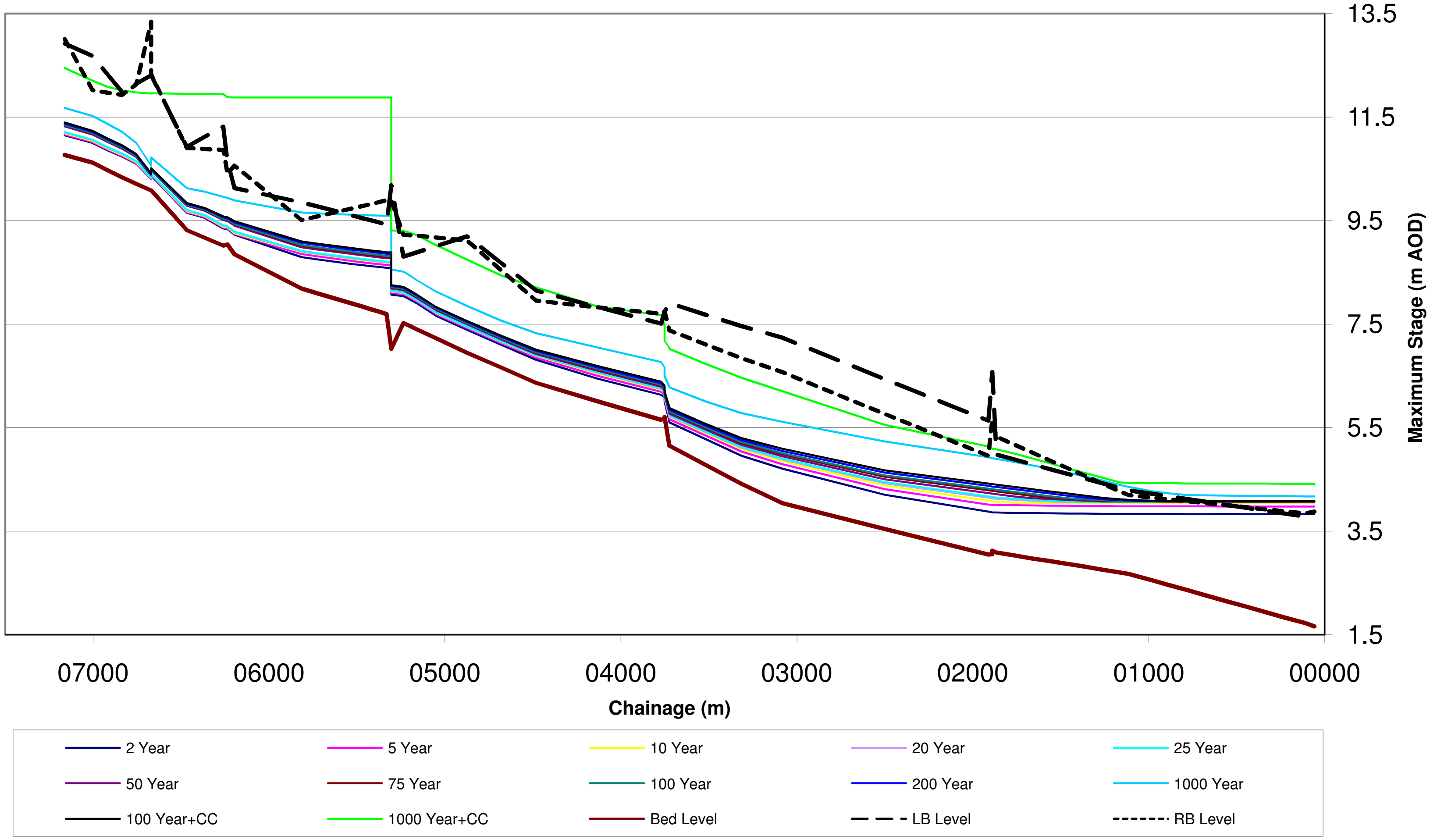


Lower Witham Catchment Long Sections



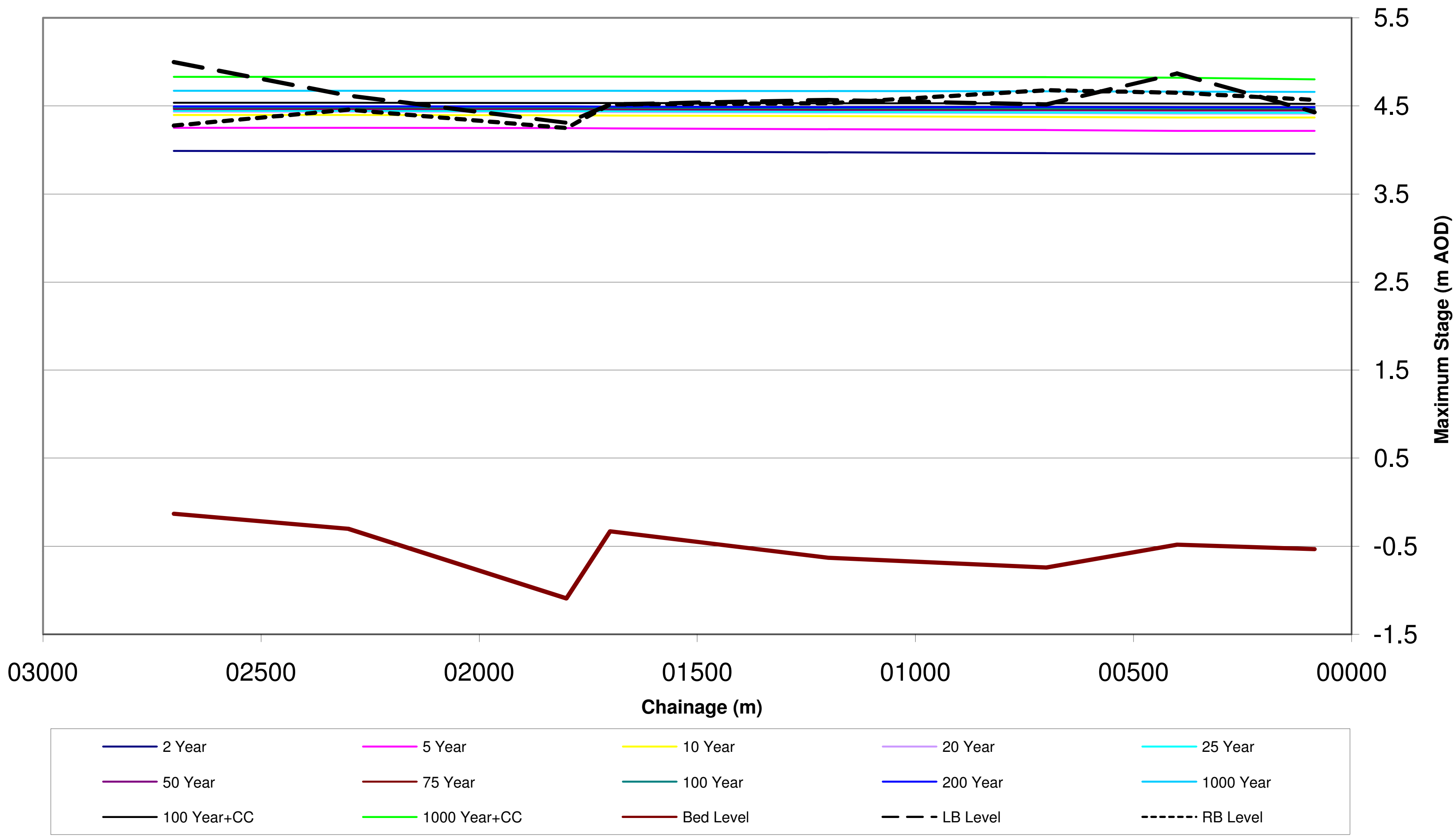
Lower Witham Catchment Long Sections

Figure D23:- Old Slea - Max Stage



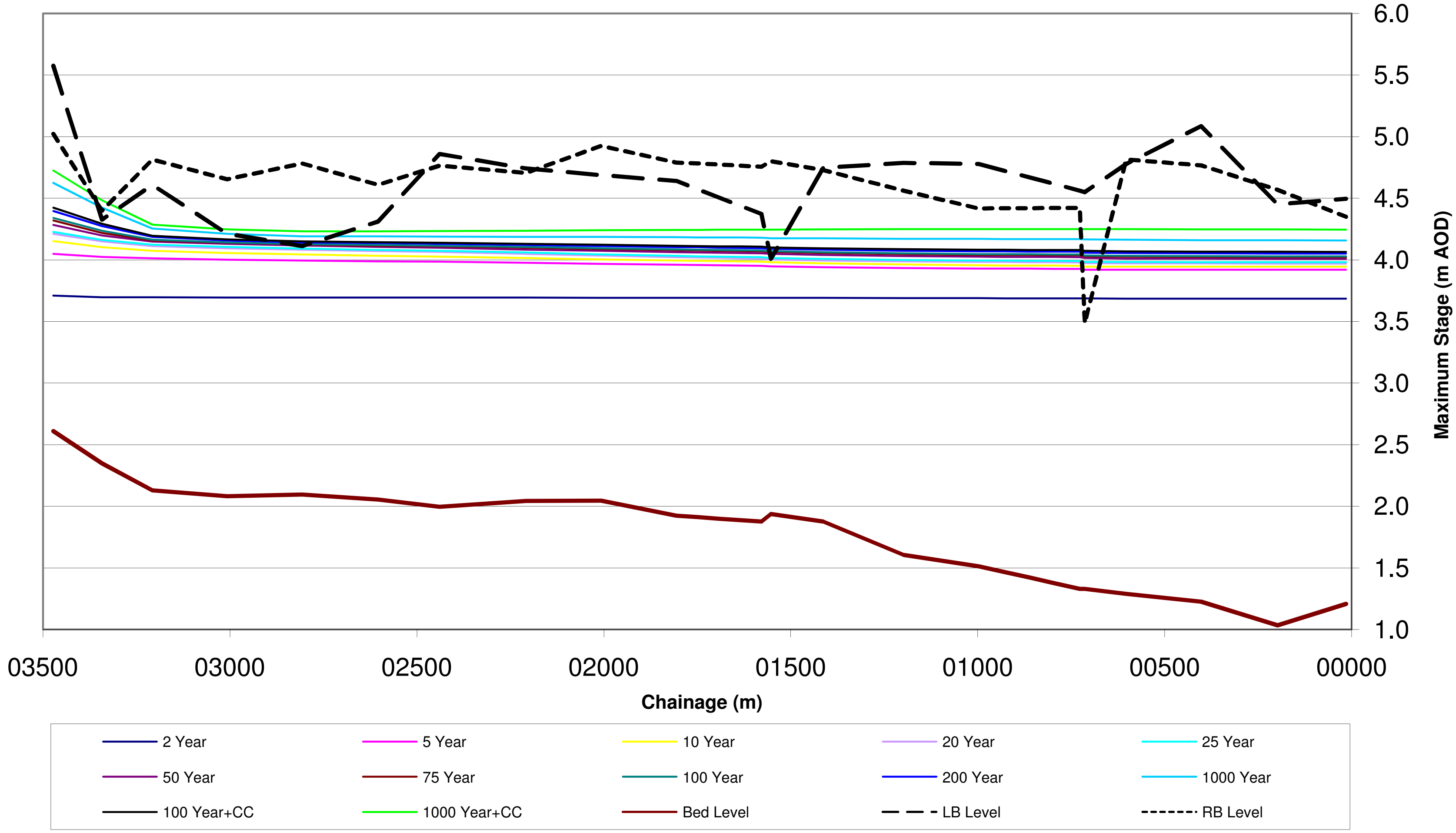
Lower Witham Catchment Long Sections

Figure D24:- Old Witham - Max Stage



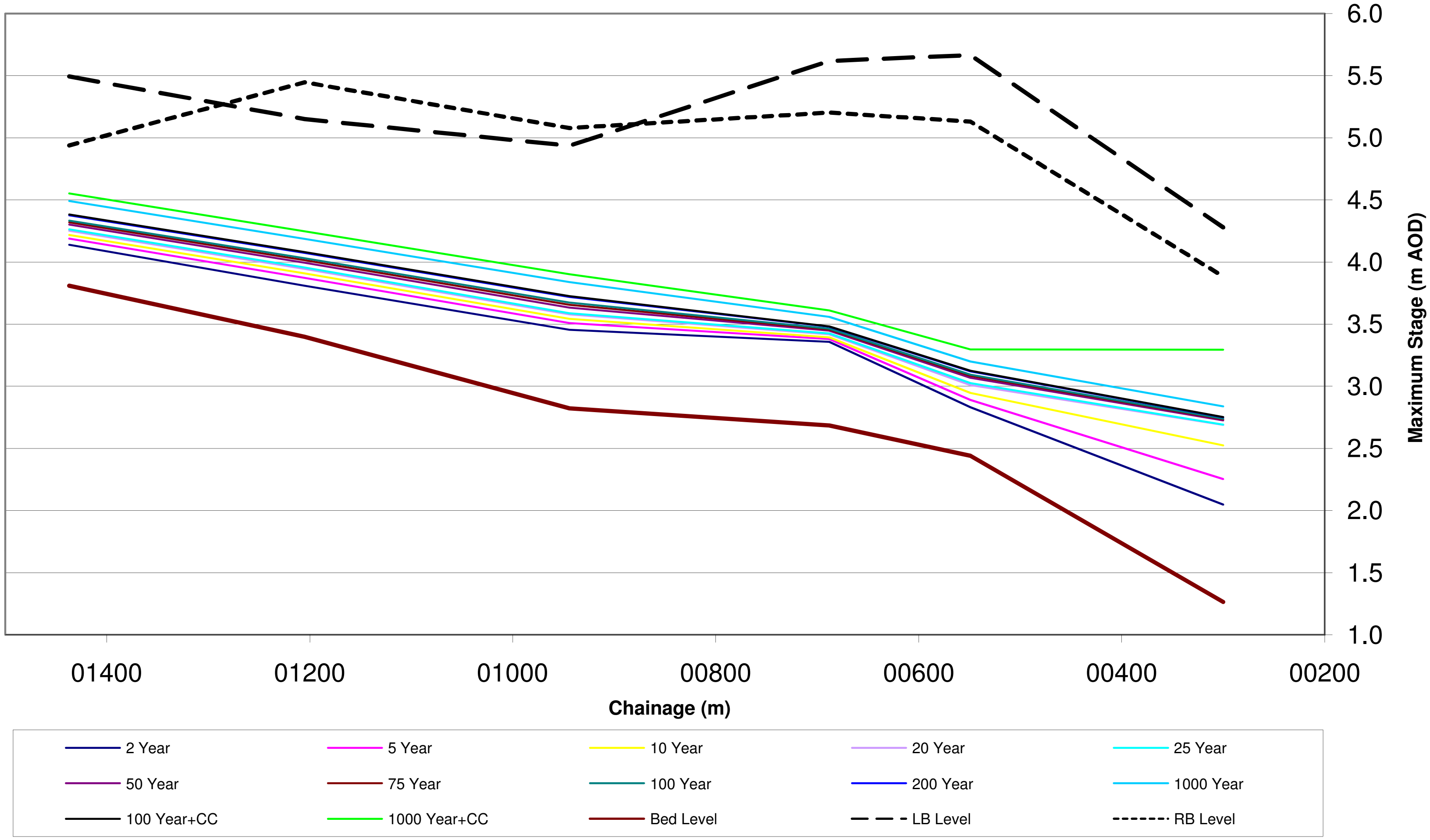
Lower Witham Catchment Long Sections

Figure D25:- Queen Dyke - Max Stage



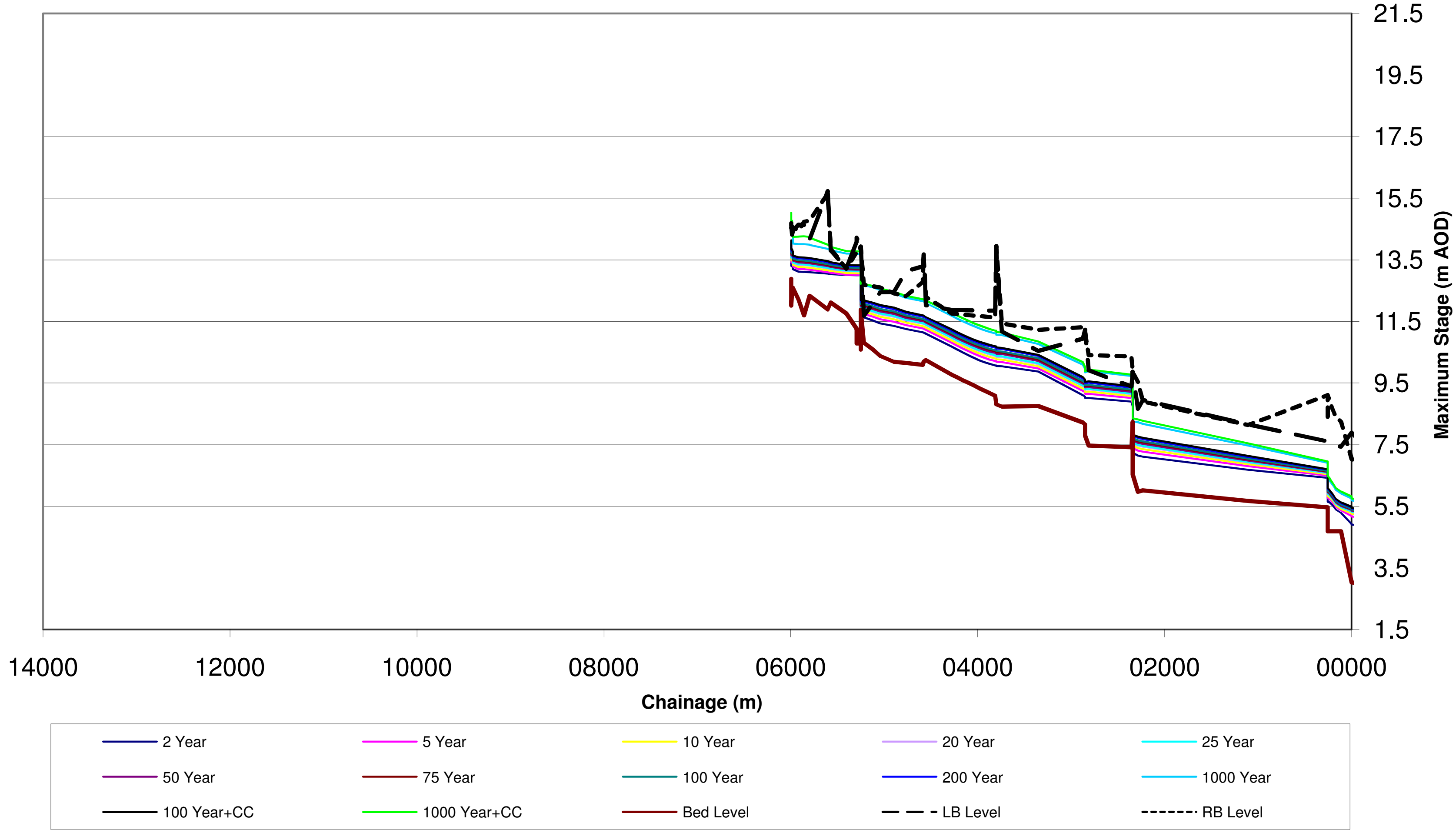
Lower Witham Catchment Long Sections

Figure D26:- Ruskington Catchwater - Max Stage

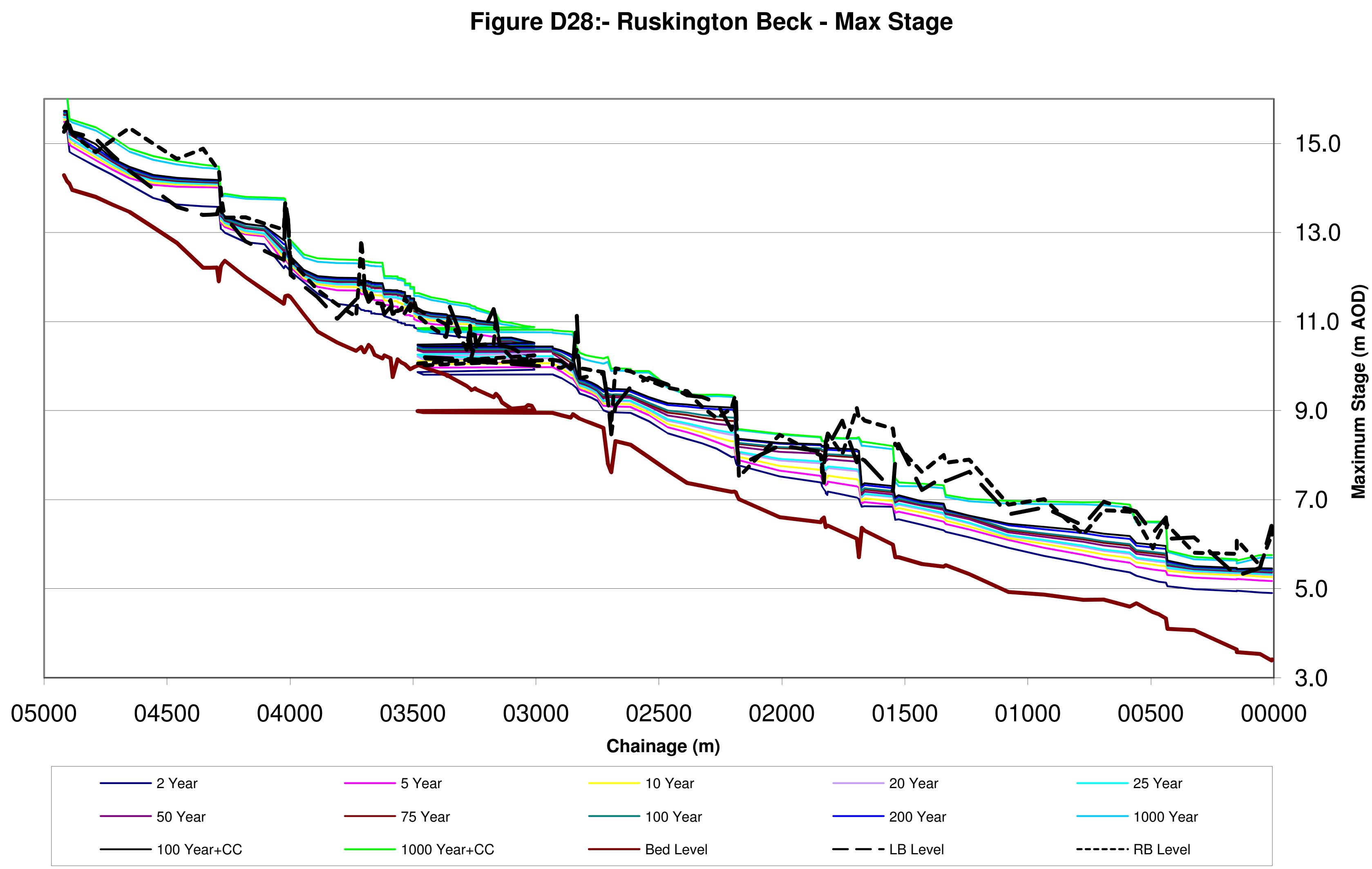


Lower Witham Catchment Long Sections

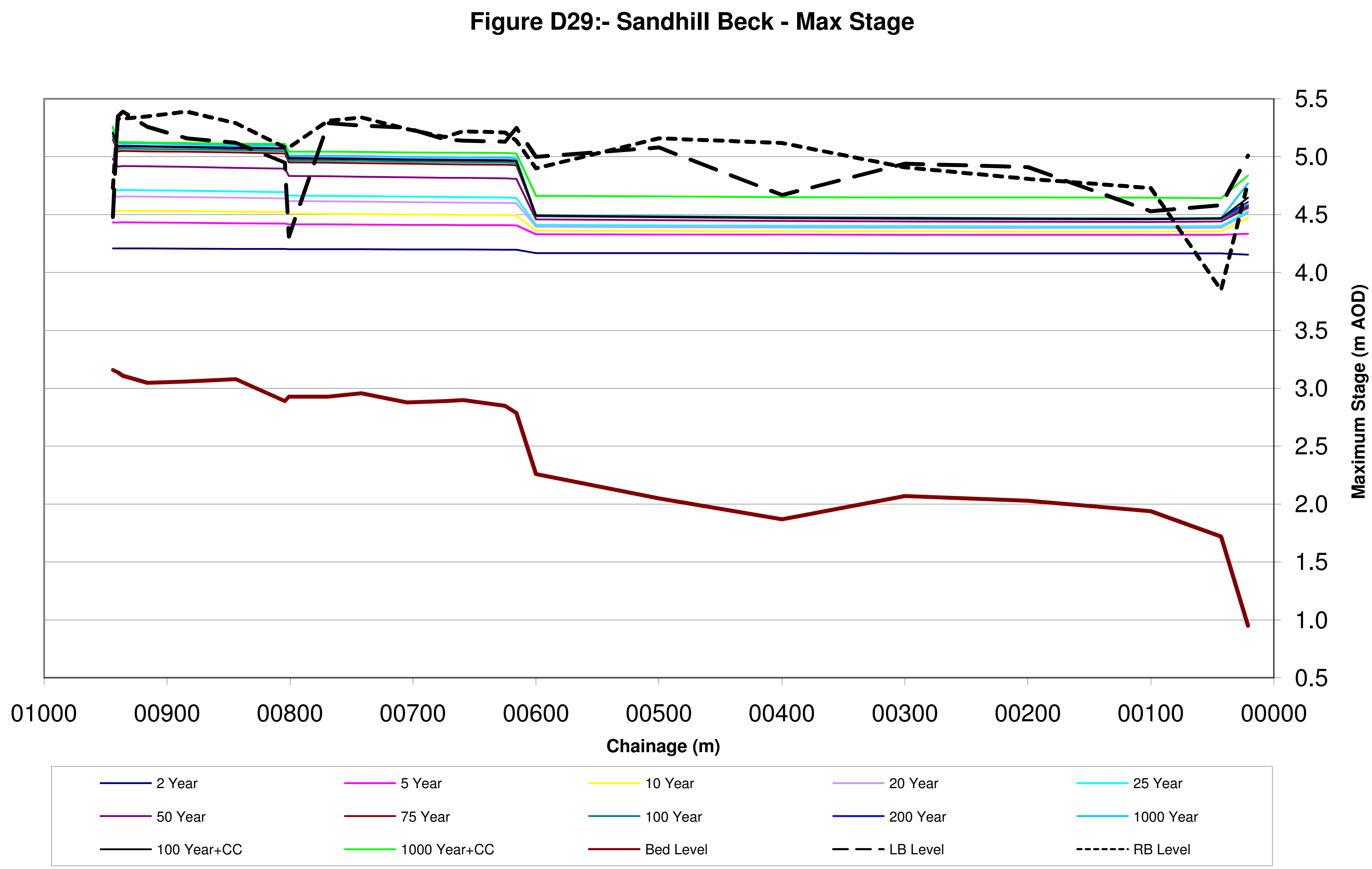
Figure D27:- River Slea - Max Stage



Lower Witham Catchment Long Sections

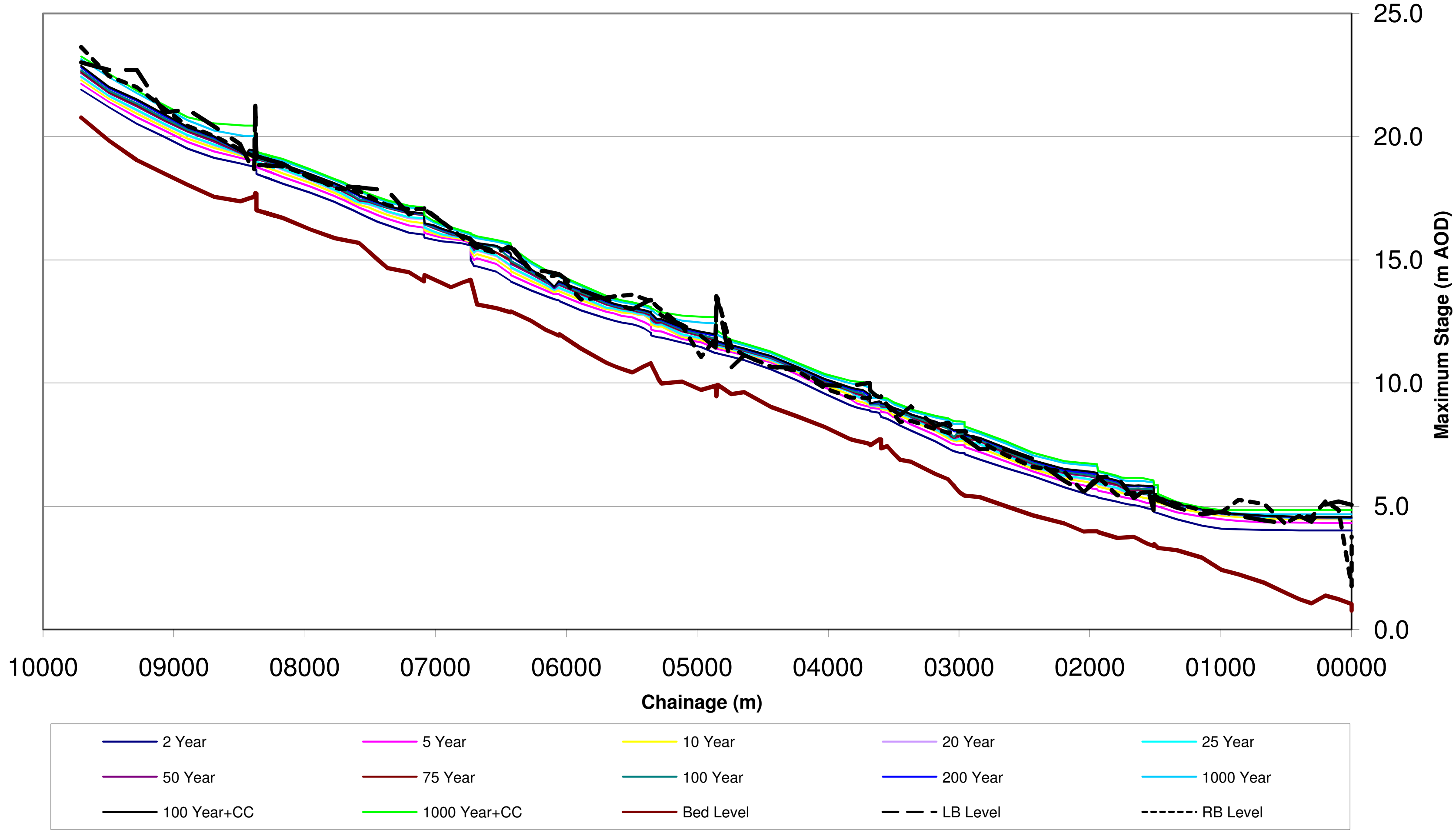


Lower Witham Catchment Long Sections

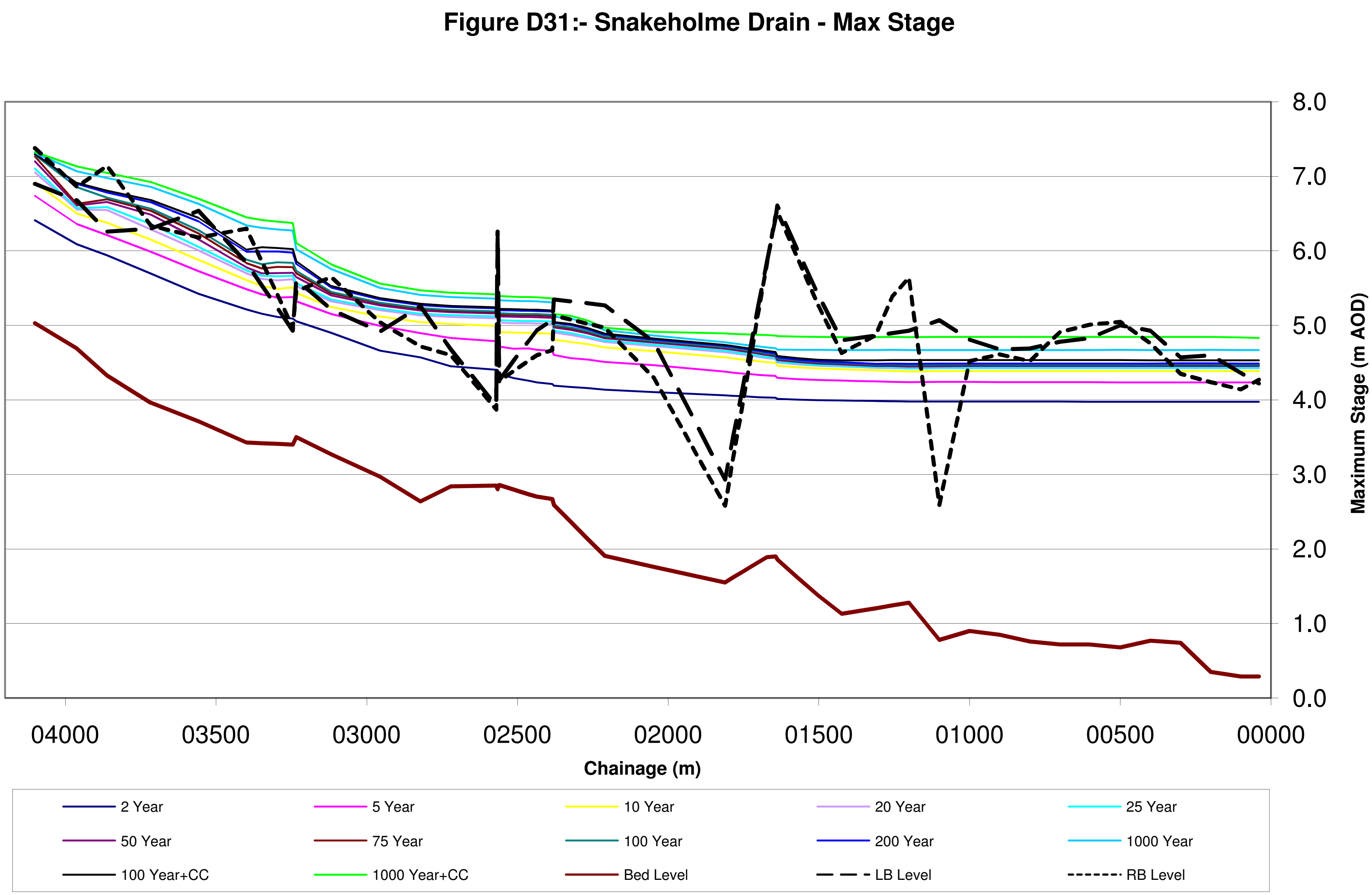


Lower Witham Catchment Long Sections

Figure D30:- Stainfield Beck - Max Stage

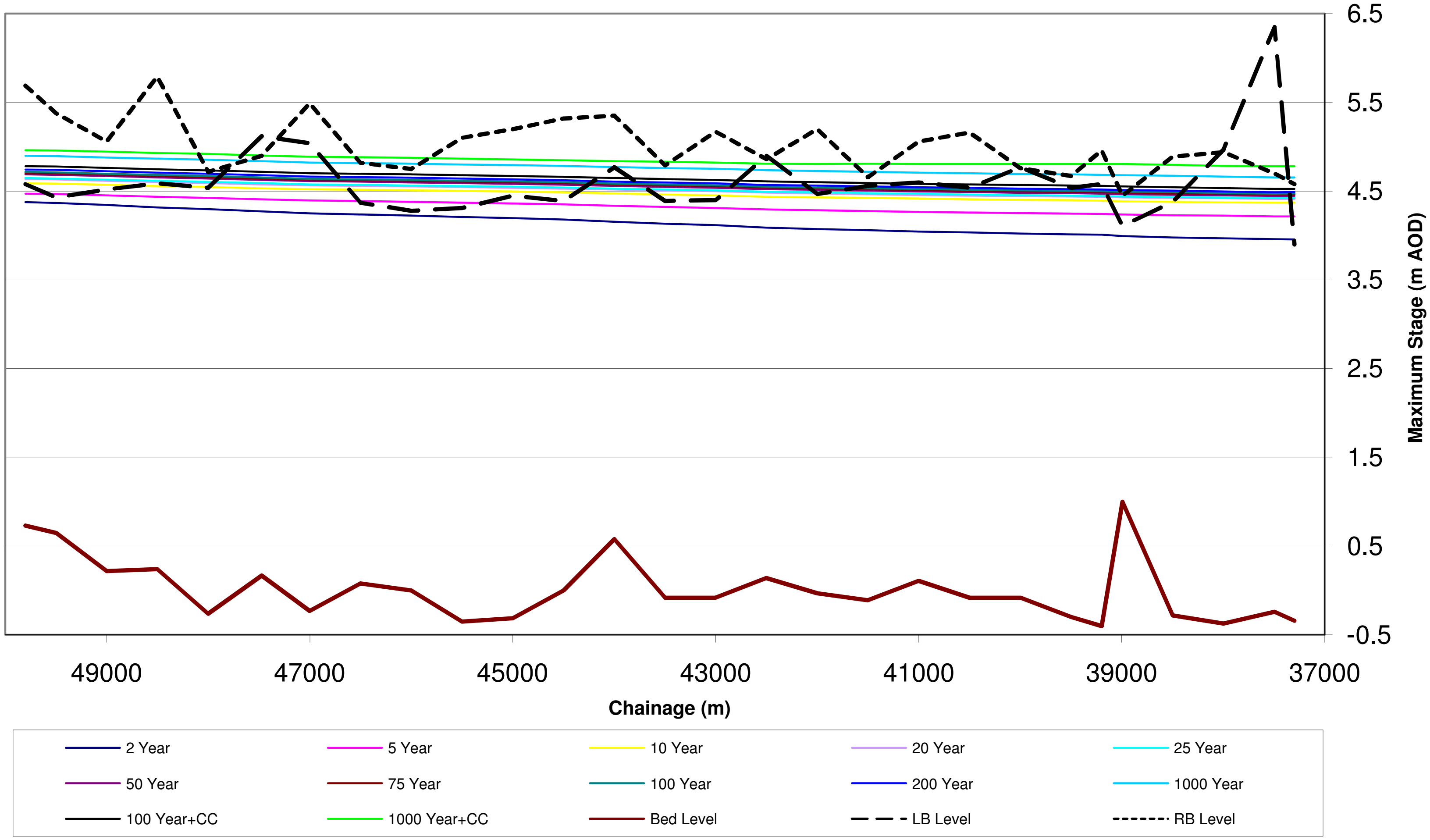


Lower Witham Catchment Long Sections



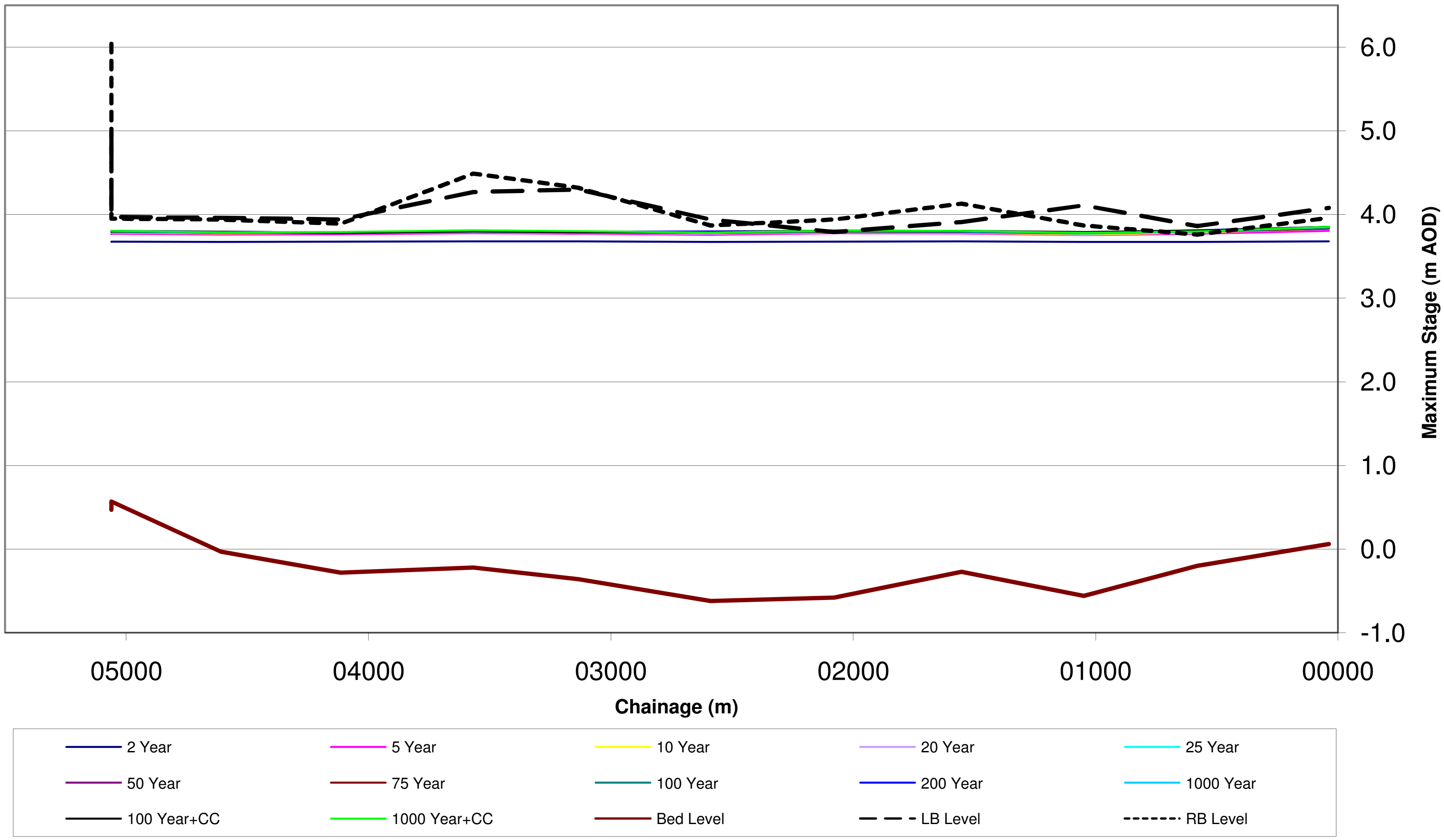
Lower Witham Catchment Long Sections

Figure D32:- South Delph - Max Stage

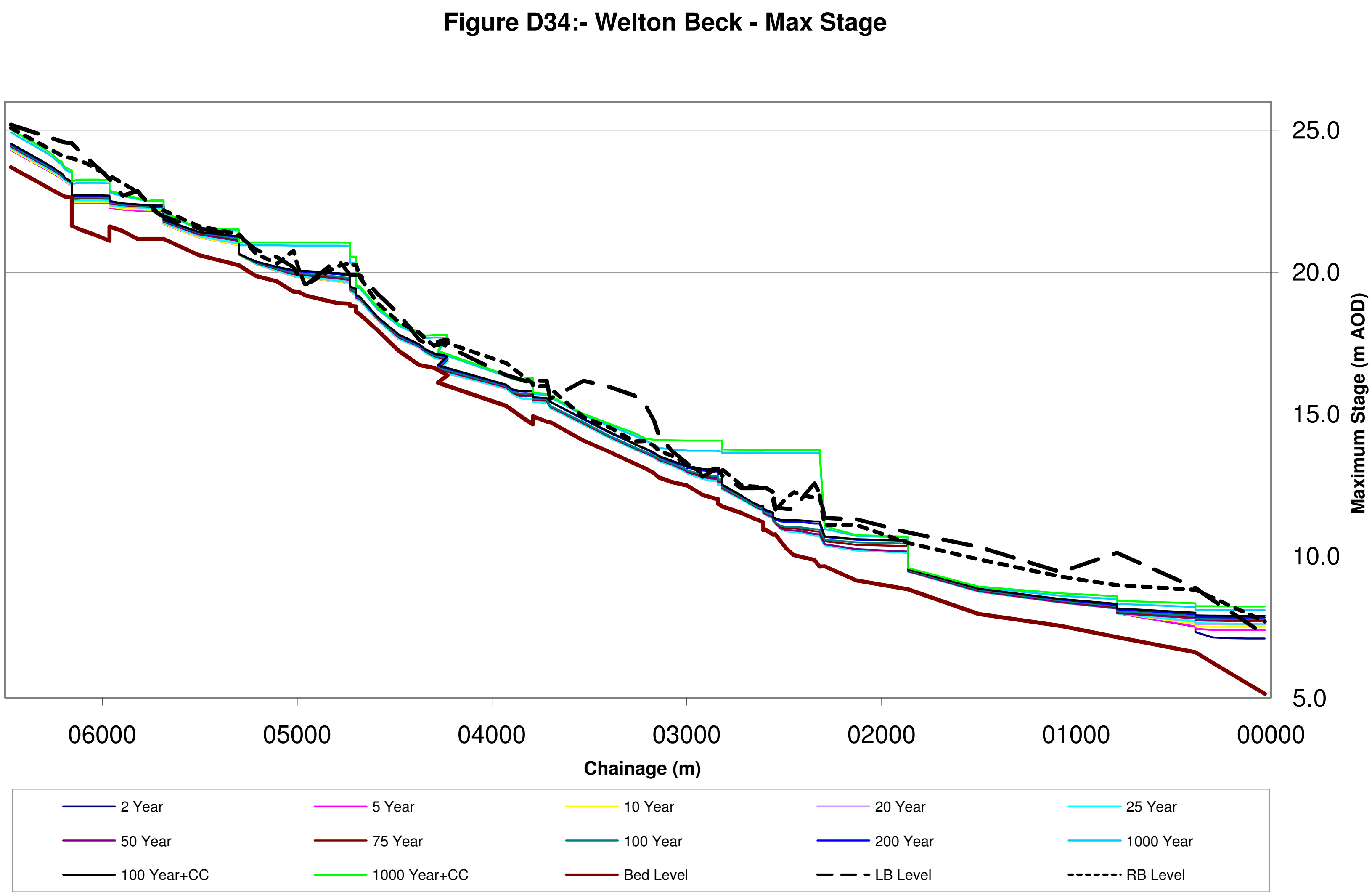


Lower Witham Catchment Long Sections

Figure D33:- Timberland Delph - Max Stage



Lower Witham Catchment Long Sections



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Appendix E - Lower Witham Sensitivity Analysis – Defended

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Appendix F - Lower Witham Freeboard AEP & SoP Analysis – Defended

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Freeboard Determination Proforma (EXAMPLE)

Defence Type	Raised Defence Type
Location	Left Bank - Cross-Section BD_01500
Reference	NFCDD Asset ID 892

Note that comments/values in red are applicable to all defences in the Lower Witham catchment. All other values are specific to this defence asset only

	Physical Process			
Wave Overtopping Allowance				Section 4.3
Fetch length (m)	42			Step 1
Allowance from Figure 4.2 (m)	0.119			Step 2 Figure 2
Modification factor	0.9			Step 3 Table 4.3
Modified Wave Overtopping Allowance	0.107			Step 3 Table 4.3
Actual allowance provided (m)		0.1	**A**	Step 4 Table 4.4

or:

Table 4.4 advises us to Flood and Reservoir Safety			
Allowance from Floods and Reservoir Safety (m)			**A**

Settlement allowance (Cohesive Soils)				Section 4.4
	Existing defences are generally assumed to have fully settled			
Embankment	*Advice notes suggest against accounting for settlement (section 4.4 pp24)*			
Height (m)				
Slopes (1:X)				
Crest width (m)				
Foundation soil type				
Coefficient μ_g				Step 1 Table 4.5
Influence factor I_P				Step 2 Table 4.4
Coefficient of volume compressibility m_v				Step 3 Test results or Table 4.6
Applied load p (kN)				Step 4
Foundation width B (m)				Step 4
Settlement allowance (m)		0	**B**	Step 5

Wall	*Advice notes suggest against accounting for settlement (section 4.4 pp24)*			
Foundation base width (m)				
Foundation length (m)				
Foundation soil type				
Coefficient μ_g				Step 1 Table 4.5
Influence factor I_P				Step 2
Coefficient of volume compressibility m_v				Step 3 Test results or Table 4.6
Applied load p (kN)				Step 4
Foundation width B (m)				Step 4
Settlement allowance (m)		0	**B**	Step 5

Superelevation	*Low velocities (and watercourse is relatively straight along defence lengths)*			
	Subsequently, it is assumed that superelevation is not significant anywhere			
River width B (m)				Section 4.7
Radius of bend r (m)				
Flow velocity (m/s)				
Superelevation significant ? Y/N				Figure 4.5
Superelevation allowance (m)		0	**C**	

Other physical processes

Advice notes with respect to other physical processes (see sections 4.5, 4.6, 4.8 and 4.9) state no freeboard allowance should be made for these processes

		0	**D**	
TOTAL ALLOWANCE FOR PHYSICAL PROCESSES		0.1	**E**	A+B+C+D

Uncertainty Allowance - Quick Method

Section 5.3

Hydrological Data Step 1

Description of information available

Good range of flow and level gauges; data of good quality; suitable geograpical spread; no flow recorders on main River Witham

Score from Table 5.1	2

Hydrological Analysis Step 1

Description of information available

Moderate to long duration storms give greatest discharges; length of flow records between 10 and 30 years

Score from Table 5.2	3

Hydraulic Data Step 1

Description of information available

Recent survey data of well defined channel sections; data of good quality; floodplain flow limited; roughness not significantly variable.

Score from Table 5.3	2

Hydraulic Analysis Step 1

Description of information available

Well defined boundary data; well defined flood routing; transient model.

Score from Table 5.4	2

Significance of physical process Step 1

Description of information available:

Uncertainties in physical processes are negligible.

Score from Table 5.5	1

Consequeance of failure Step 1

Description of information available:

Catchment includes a good proportion of high grade agricultural land; some properties at risk of flooding.

Score from Table 5.6	4

Total score	14		Step 2
Mean annual maximum water level (m)	1.57		
Derived flood level (m)	2.70	HIGH indicative SoP	
TOTAL UNCERTAINTY ALLOWANCE	0.26	**F**	Step 3
REALITY CHECK (OK ?)	OK		
TOTAL FREEBOARD ALLOWANCE	0.36		**E+F**