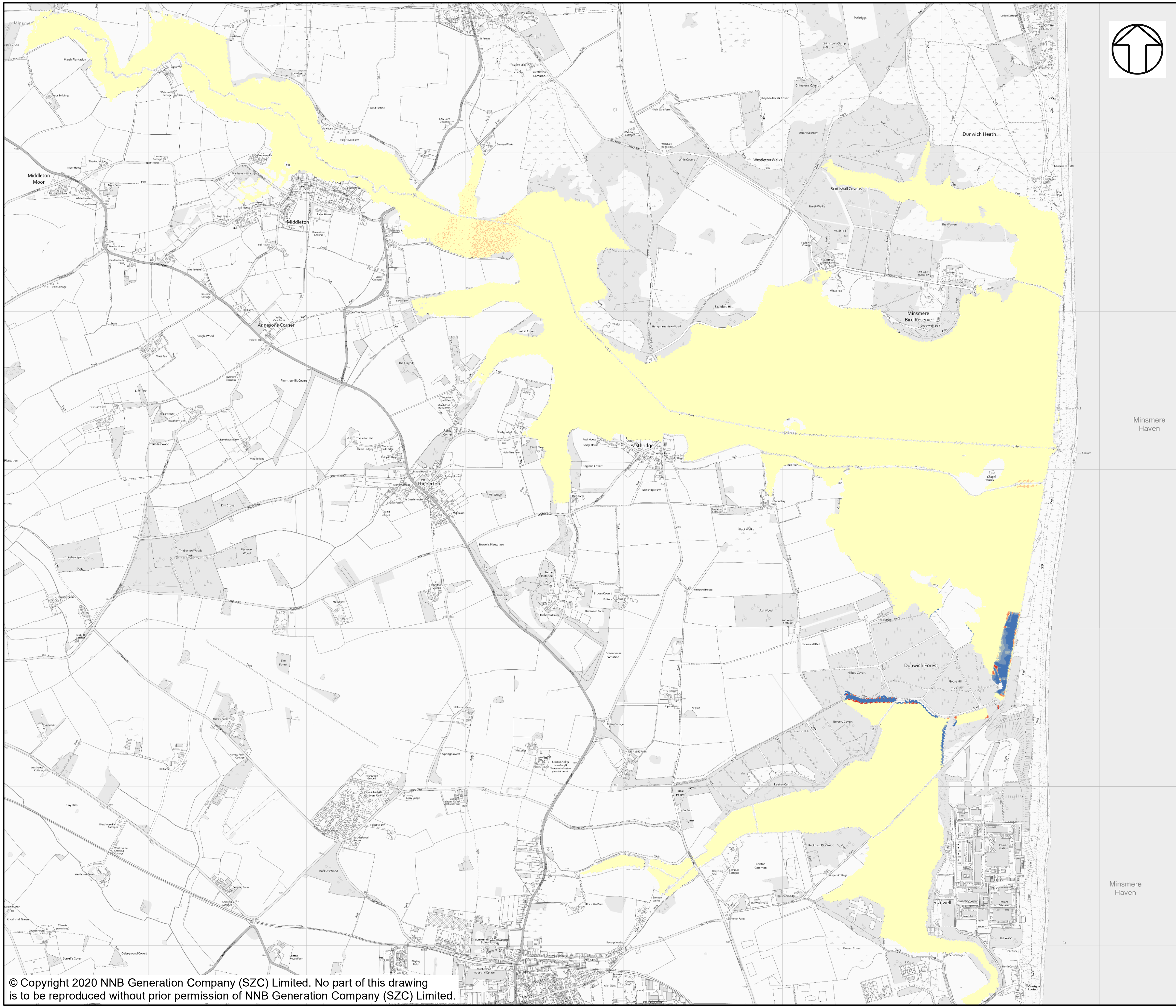
The Sizewell C Project

5.2 Main Development Site Flood Risk Assessment
Appendices 1-7 Part 6 of 14

Revision:	1.0
Applicable Regulation:	Regulation 5(2)(e)
PINS Reference Number:	EN010012

May 2020

Planning Act 2008
Infrastructure Planning (Applications: Prescribed Forms and Procedure) Regulations 2009



NOTES

KEY

DIFFERENCE 1000 YEAR +25%CC

FLOOD DEPTH DIFFERENCE (M)

- 0.3 - 2
- 0.1 - 0.3
- 0.03 - 0.1
- 0.01 - 0.03
- 0
- -0.03 - -0.01
- -0.1 - -0.03
- -0.3 - -0.1
- -2.5 - -0.3

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DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD DEPTH
1000 YEAR +25%CC

DRAWING NO:
FIGURE E.13

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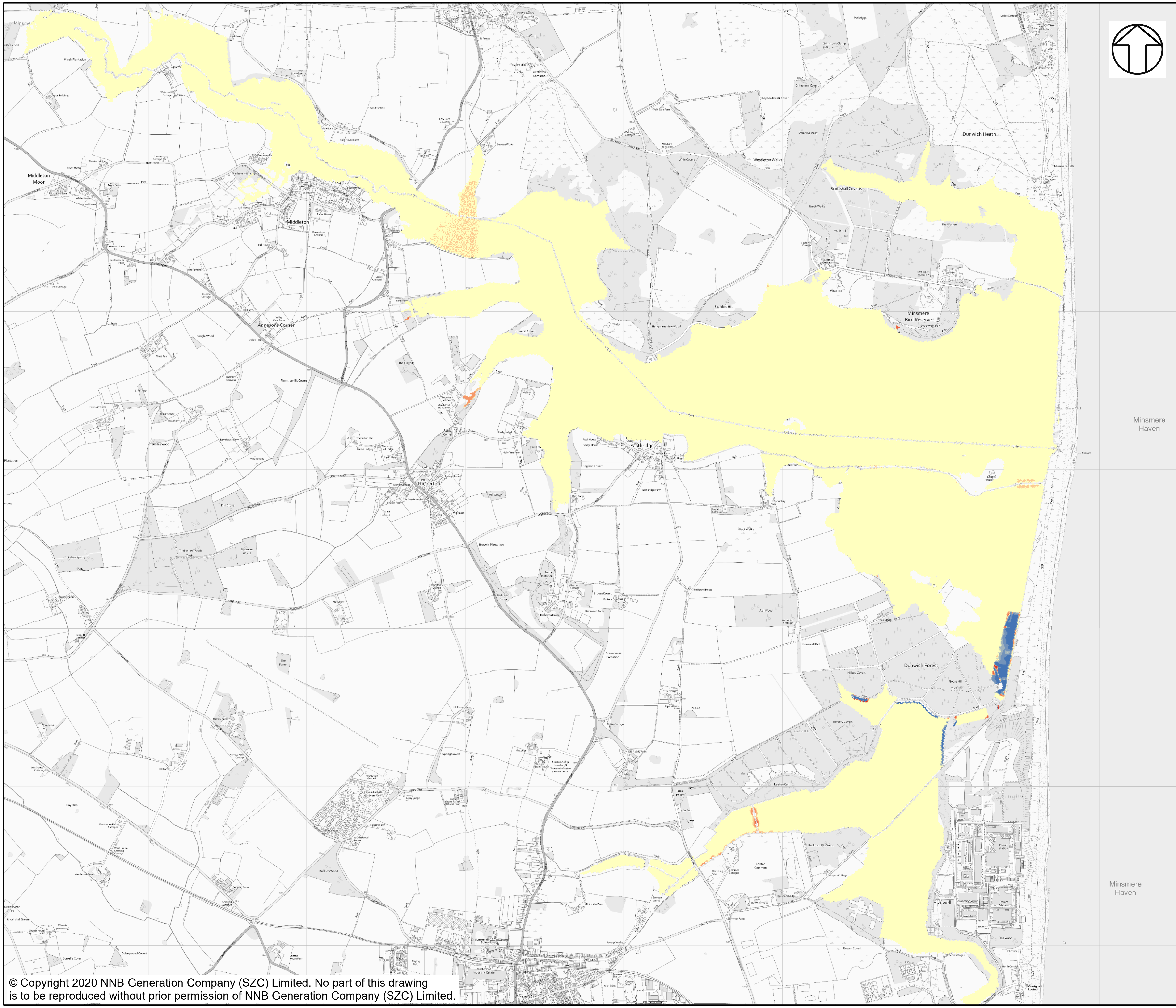

NOTES

KEY

DIFFERENCE 1000 YEAR +35%CC

FLOOD DEPTH DIFFERENCE (M)

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- 0.1 - 0.3
- 0.03 - 0.1
- 0.01 - 0.03
- 0
- -0.03 - -0.01
- -0.1 - -0.03
- -0.3 - -0.1
- -2.5 - -0.3

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD DEPTH
1000 YEAR +35%CC

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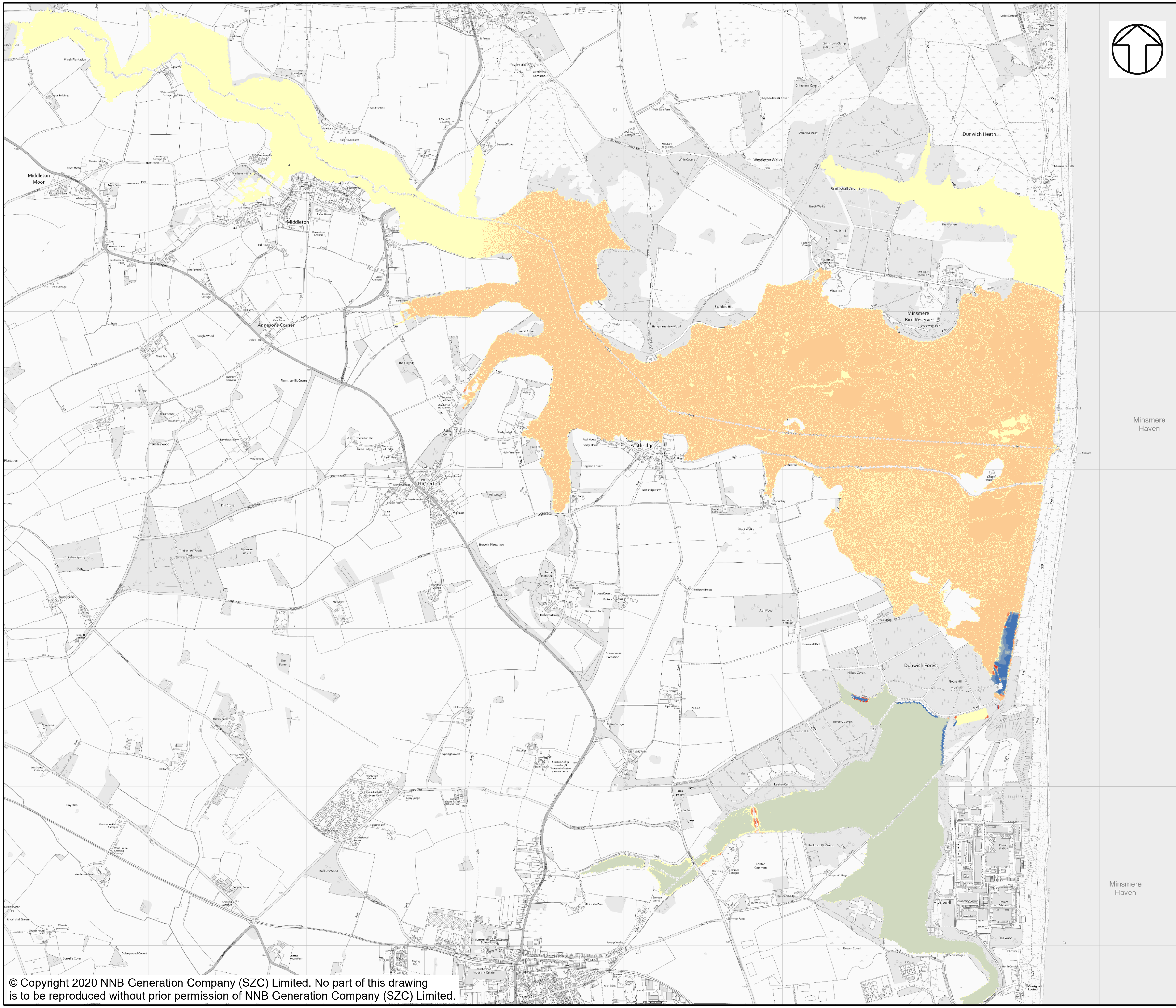

NOTES

KEY

DIFFERENCE 1000 YEAR +65%CC

FLOOD DEPTH DIFFERENCE (M)

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- 0.1 - 0.3
- 0.03 - 0.1
- 0.01 - 0.03
- 0
- -0.03 - -0.01
- -0.1 - -0.03
- -0.3 - -0.1
- -2.5 - -0.3

NOT PROTECTIVELY MARKED

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SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD DEPTH
1000 YEAR +65%CC

DRAWING NO:
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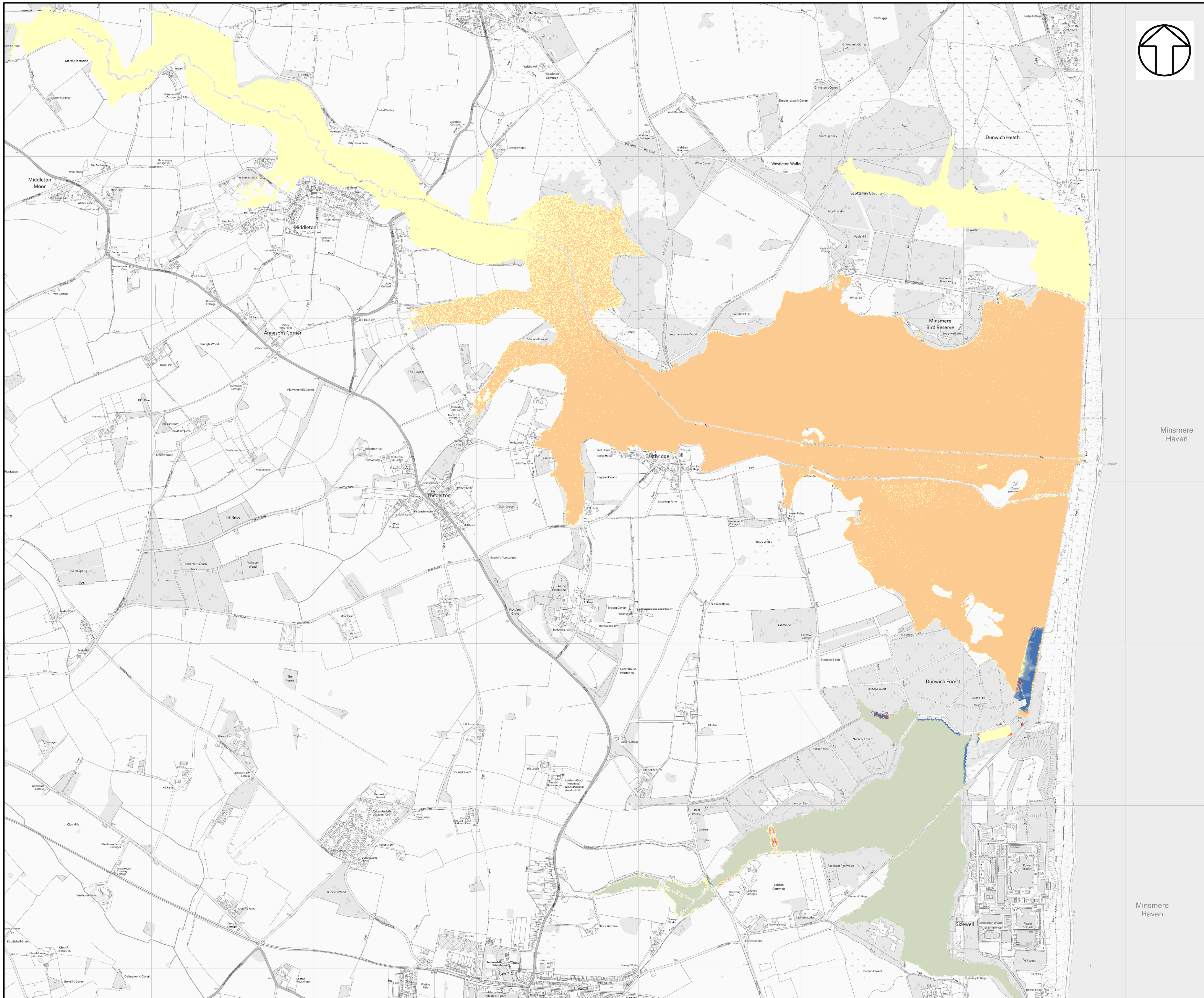

NOTES

KEY

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- 0.03 - 0.1
- 0.01 - 0.03
- 0
- -0.03 - -0.01
- -0.1 - -0.03
- -0.3 - -0.1
- -2.5 - -0.3

NOT PROTECTIVELY MARKED

SZC
EDF ENERGY
CGN

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD DEPTH
1000 YEAR +80%CC

DRAWING NO:
FIGURE E.16

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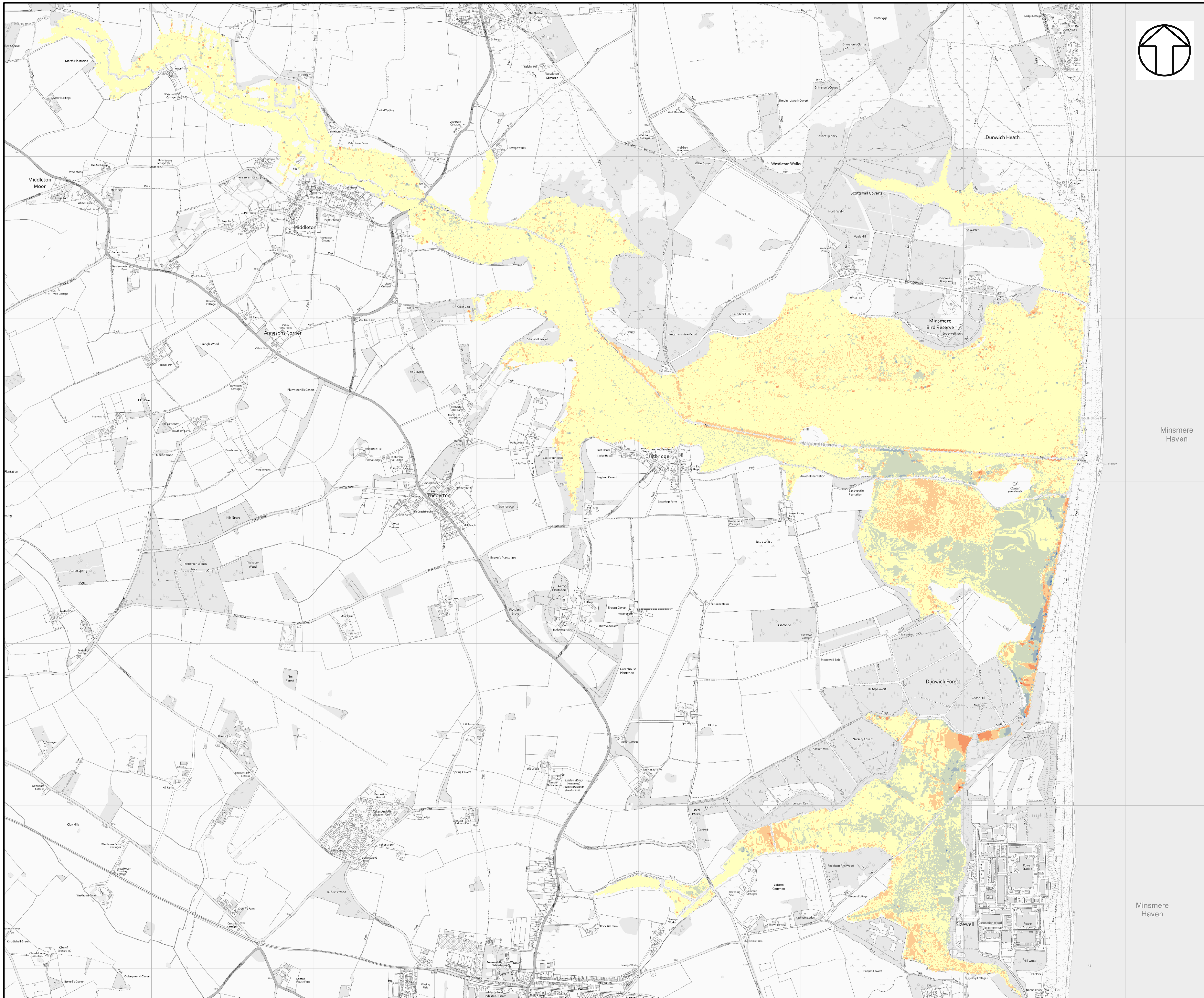

NOTES

KEY

DIFFERENCE 5 YEAR +35%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.1 - 0.15
- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

NOT PROTECTIVELY MARKED

DOCUMENT:
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FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
5 YEAR +35%CC

DRAWING NO:
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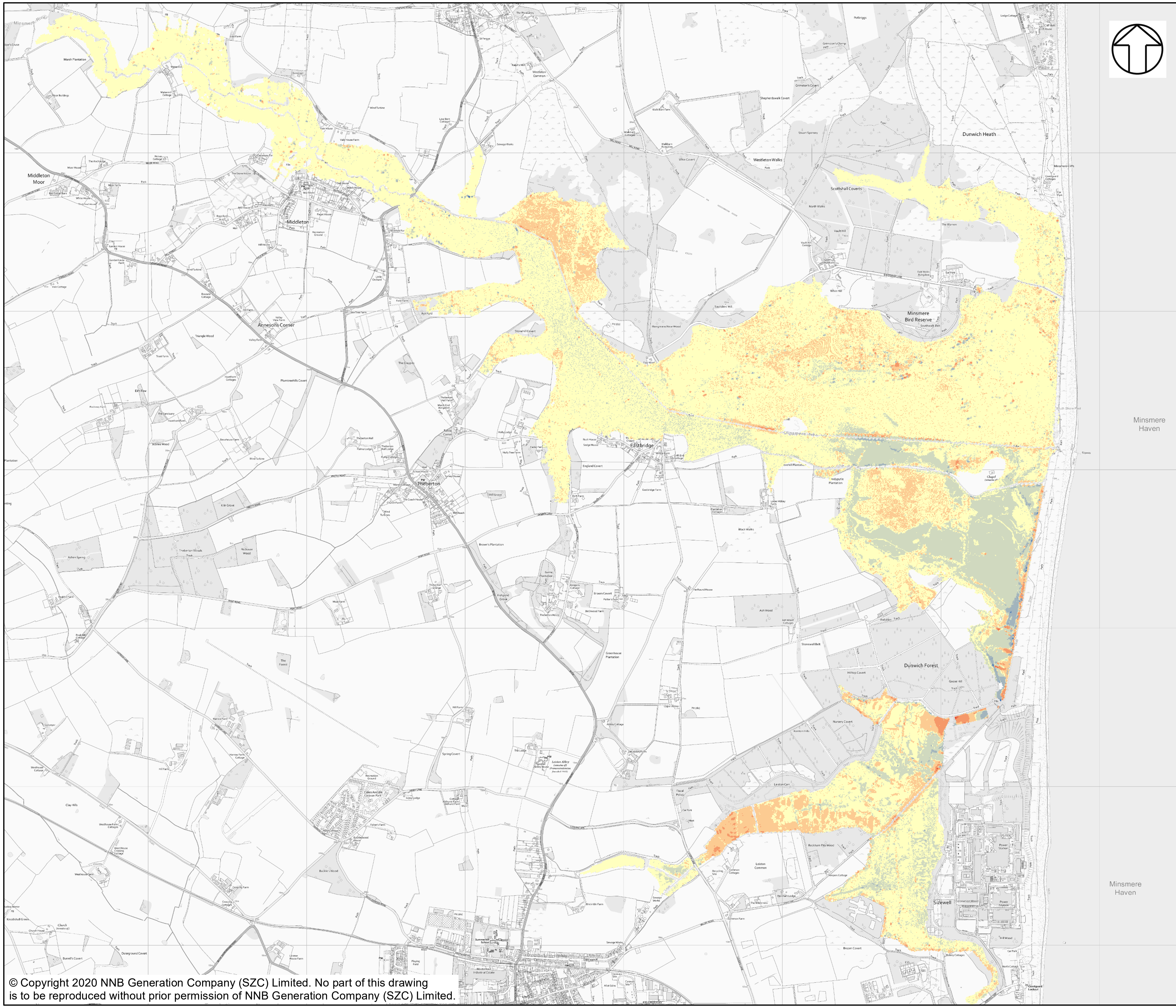

NOTES

KEY

DIFFERENCE 5 YEAR +65%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.1 - 0.15
- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

NOT PROTECTIVELY MARKED

EDF ENERGY SZC CGN

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
5 YEAR +65%CC

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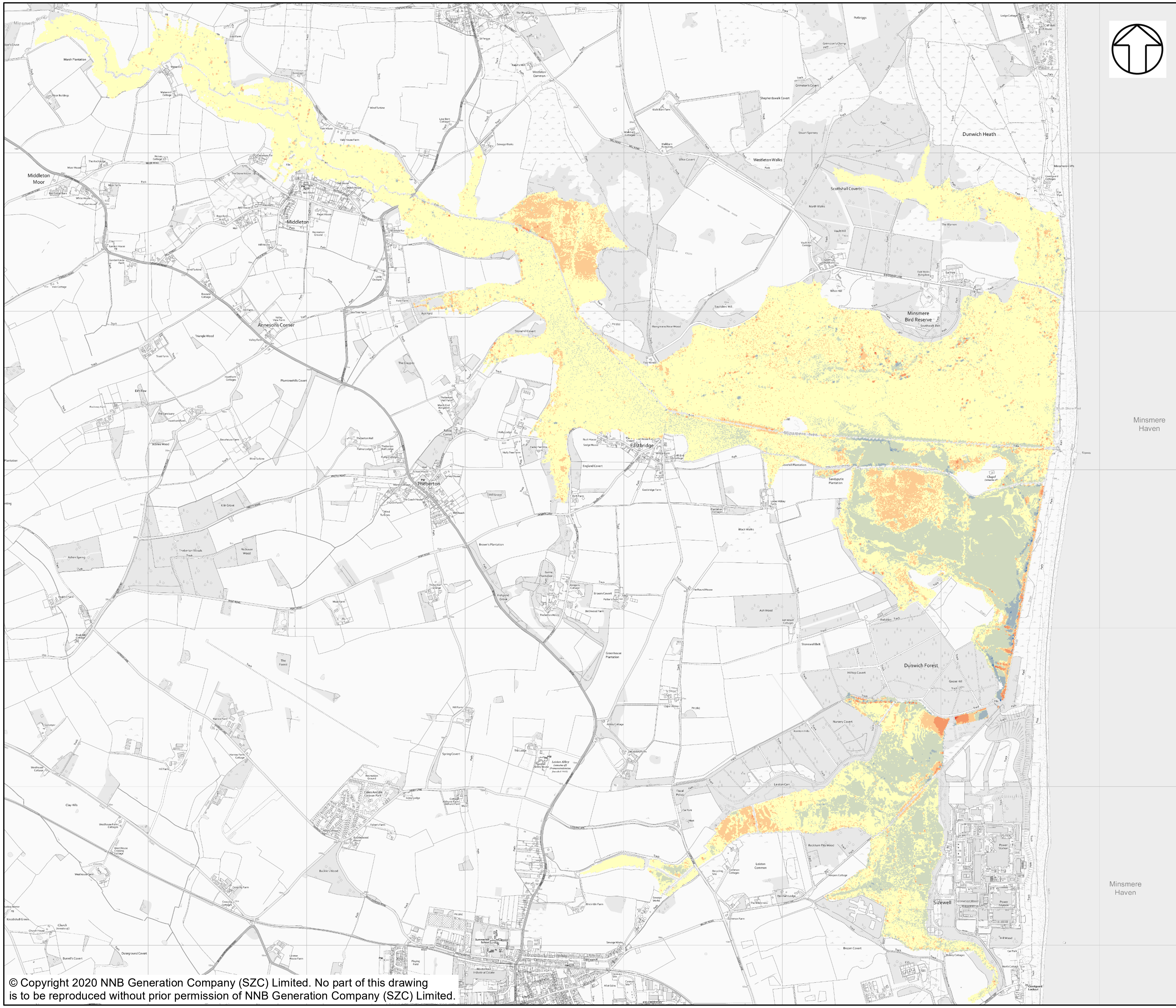

NOTES

KEY

DIFFERENCE 20 YEAR +35%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
20 YEAR +35%CC

DRAWING NO:
FIGURE E.19

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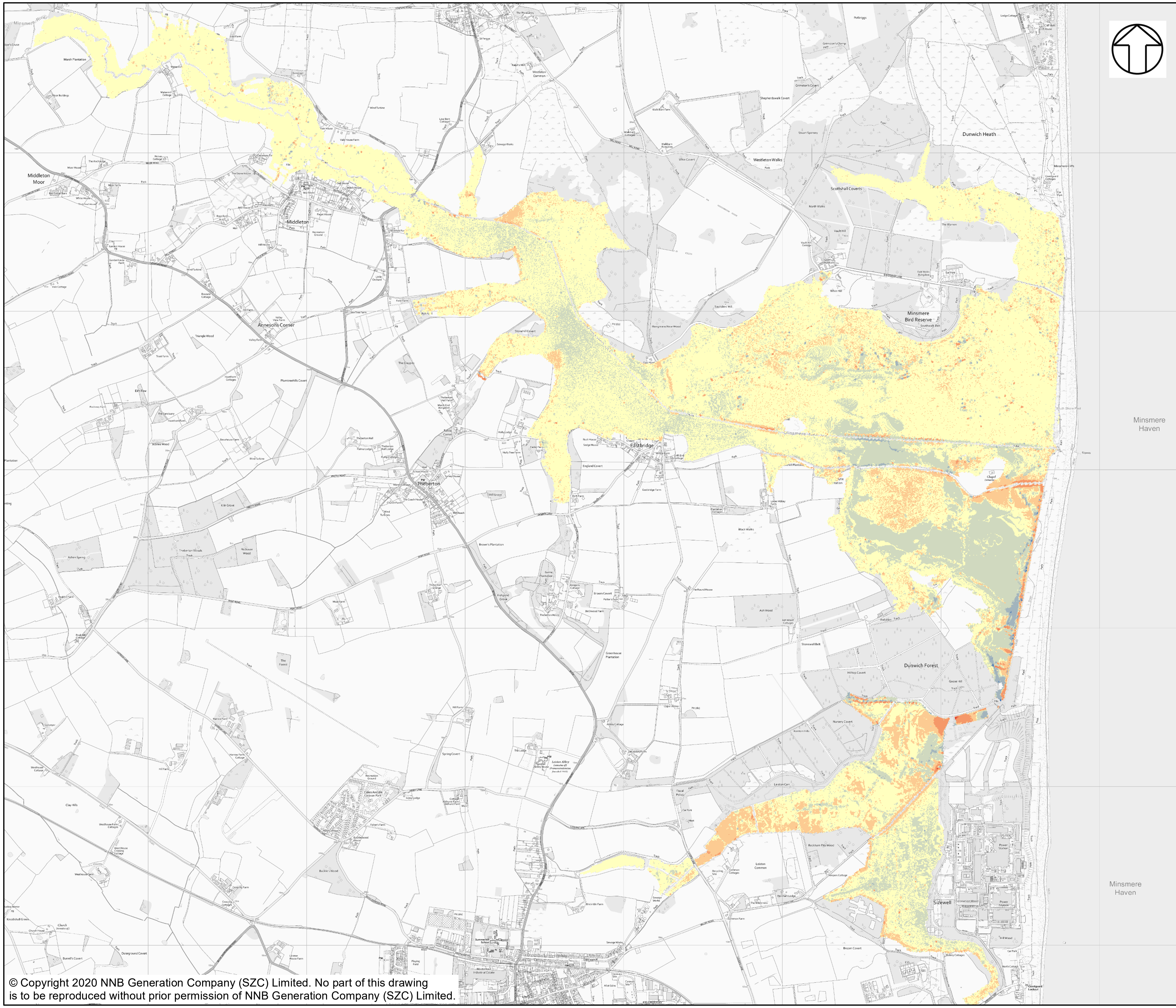

NOTES

KEY

DIFFERENCE 20 YEAR +65%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

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SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
20 YEAR +65%CC

DRAWING NO:
FIGURE E.20

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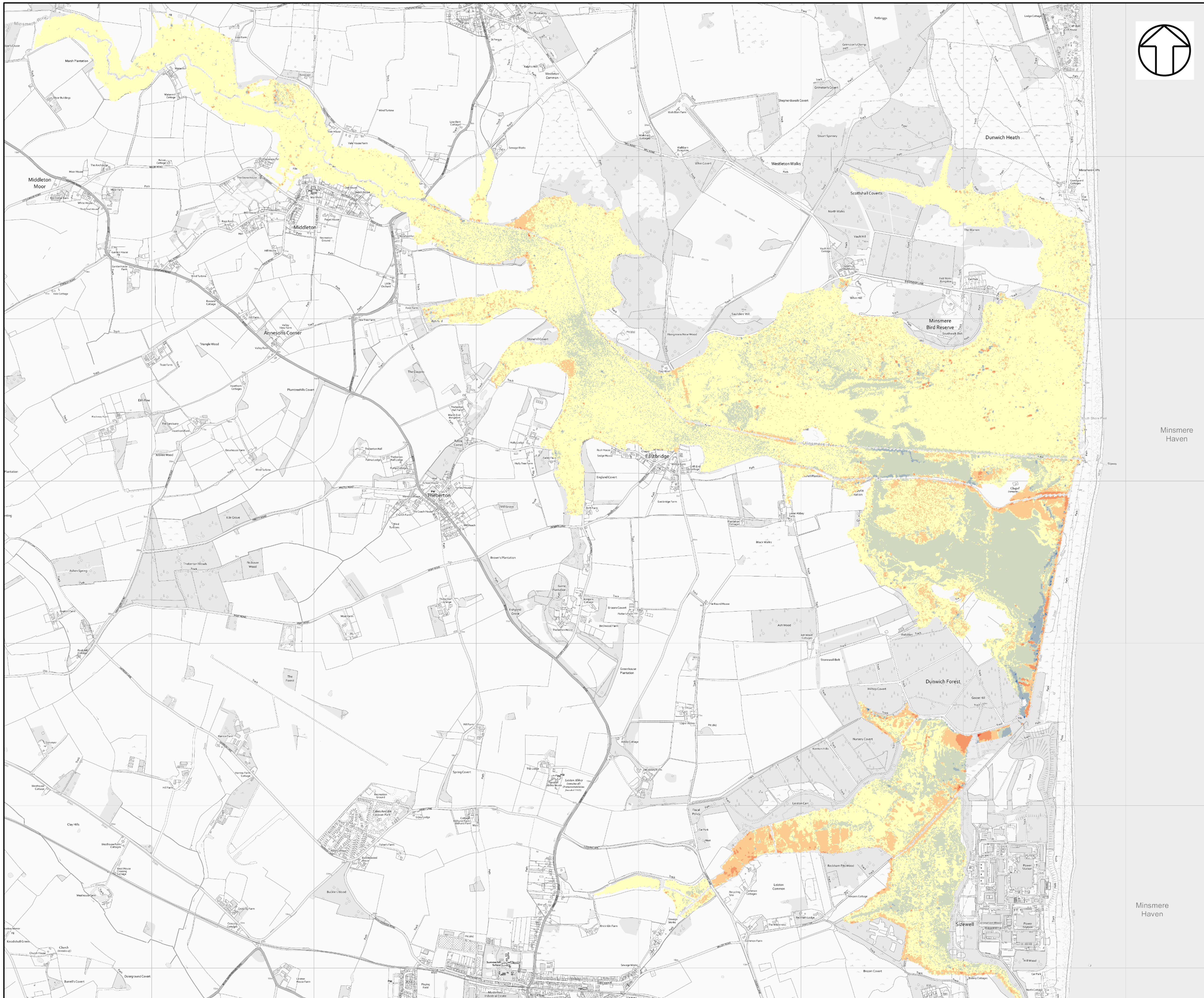

NOTES

KEY

DIFFERENCE 100 YEAR +35%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

NOT PROTECTIVELY MARKED

EDF ENERGY SZC CGN

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
100 YEAR +35%CC

DRAWING NO:
FIGURE E.21

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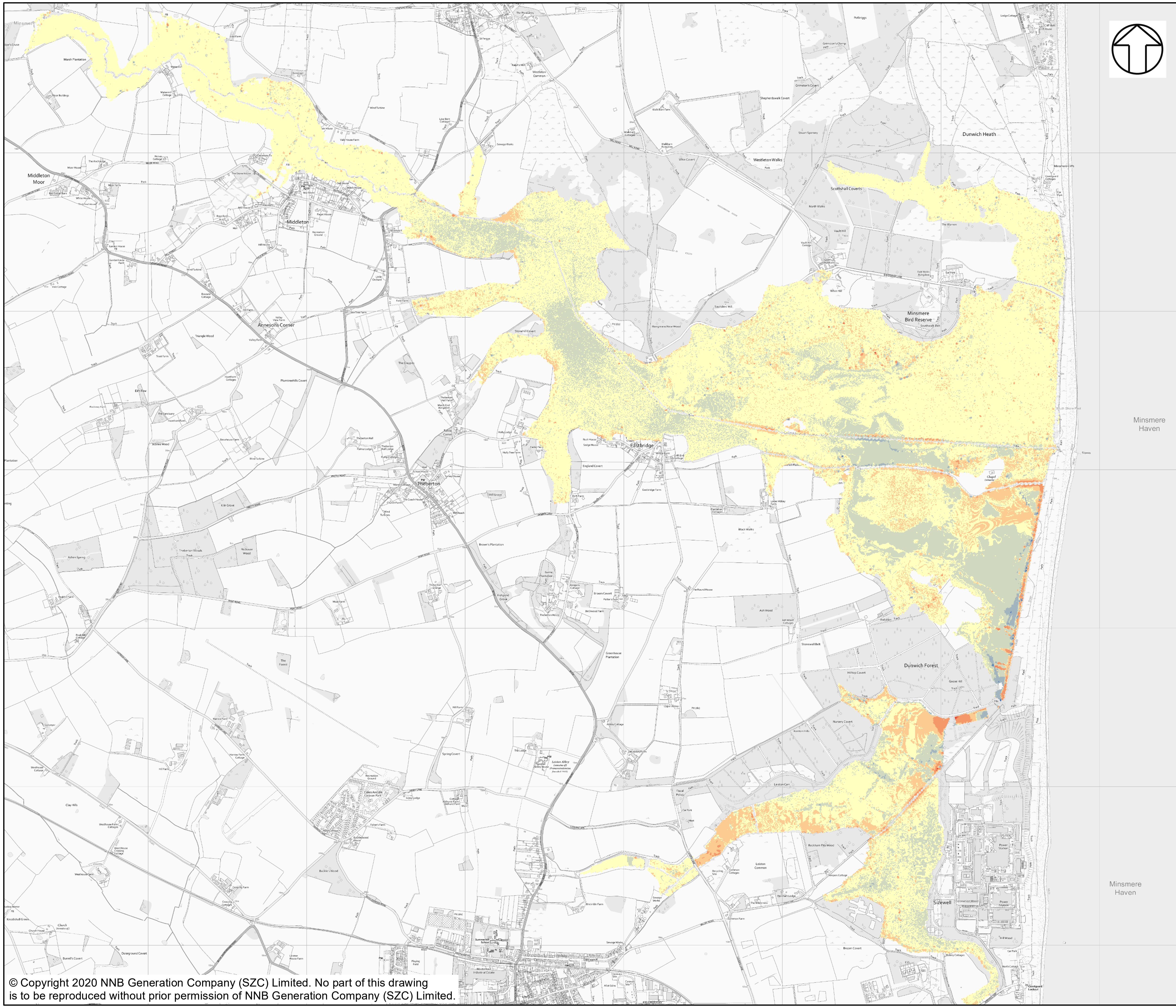

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DIFFERENCE 100 YEAR +65%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.001 - 0.01
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- -0.01 - -0.1
- -0.15 - -0.1
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NOT PROTECTIVELY MARKED

EDF ENERGY SZC CGN

DOCUMENT:
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FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
100 YEAR +65%CC

DRAWING NO:
FIGURE E.22

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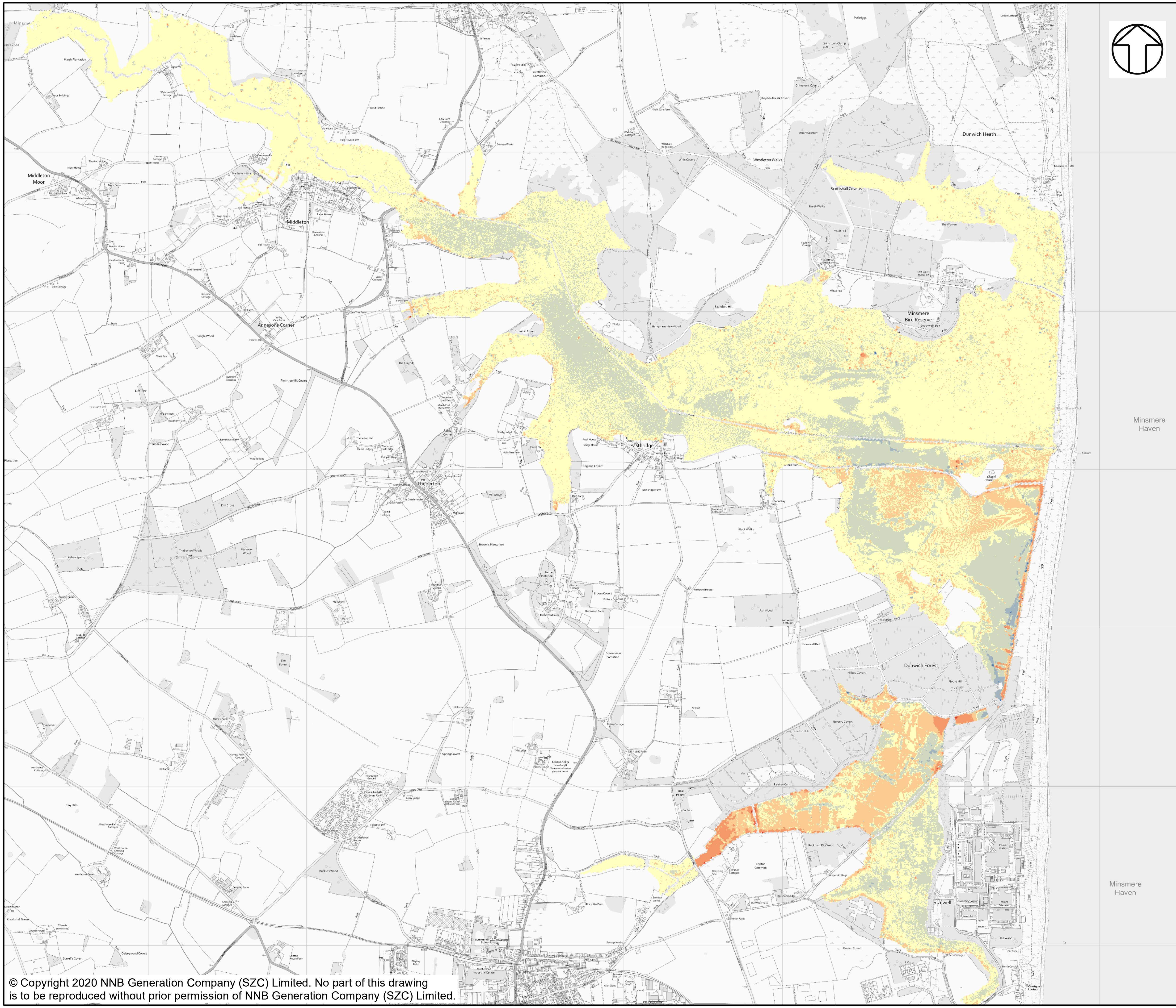

NOTES

KEY

DIFFERENCE 1000 YEAR +35%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.01 - 0.1
- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

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FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
1000 YEAR +35%CC

DRAWING NO:
FIGURE E.23

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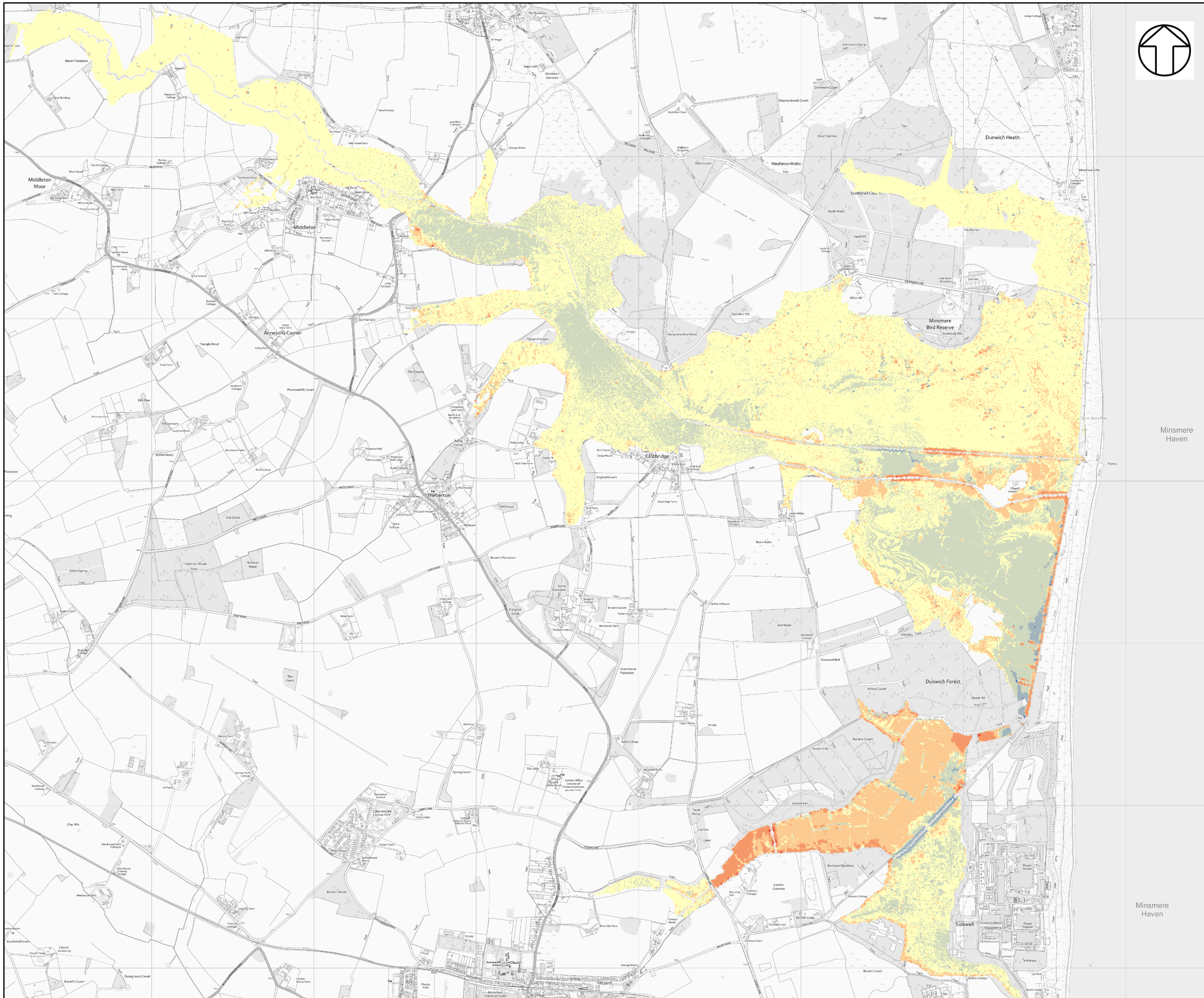

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KEY

DIFFERENCE 1000 YEAR +65%CC

FLOOD VELOCITY DIFFERENCE (M/S)

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- 0.001 - 0.01
- 0
- -0.001 - -0.01
- -0.01 - -0.1
- -0.15 - -0.1
- -2.5 - -0.15

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD VELOCITY
1000 YEAR +65%CC

DRAWING NO:
FIGURE E.24

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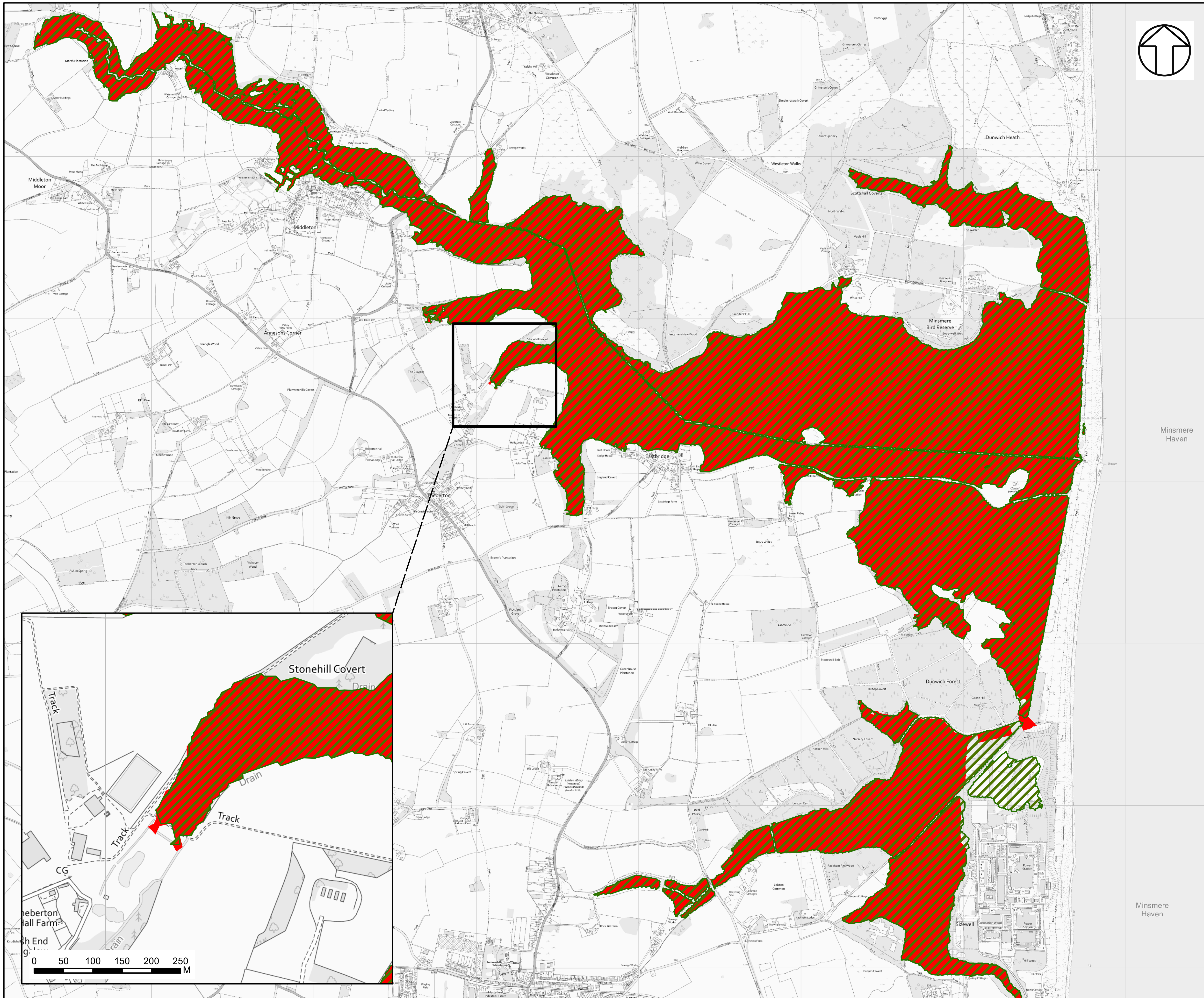

NOTES

KEY

FLOOD EXTENTS 100 YEAR +35%CC

- BASELINE FLOOD EXTENT 100 YEAR +35%CC
- WITH SCHEME FLOOD EXTENT 100 YEAR +35%CC

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD EXTENTS
100 YEAR +35%CC

DRAWING NO:
FIGURE E.25

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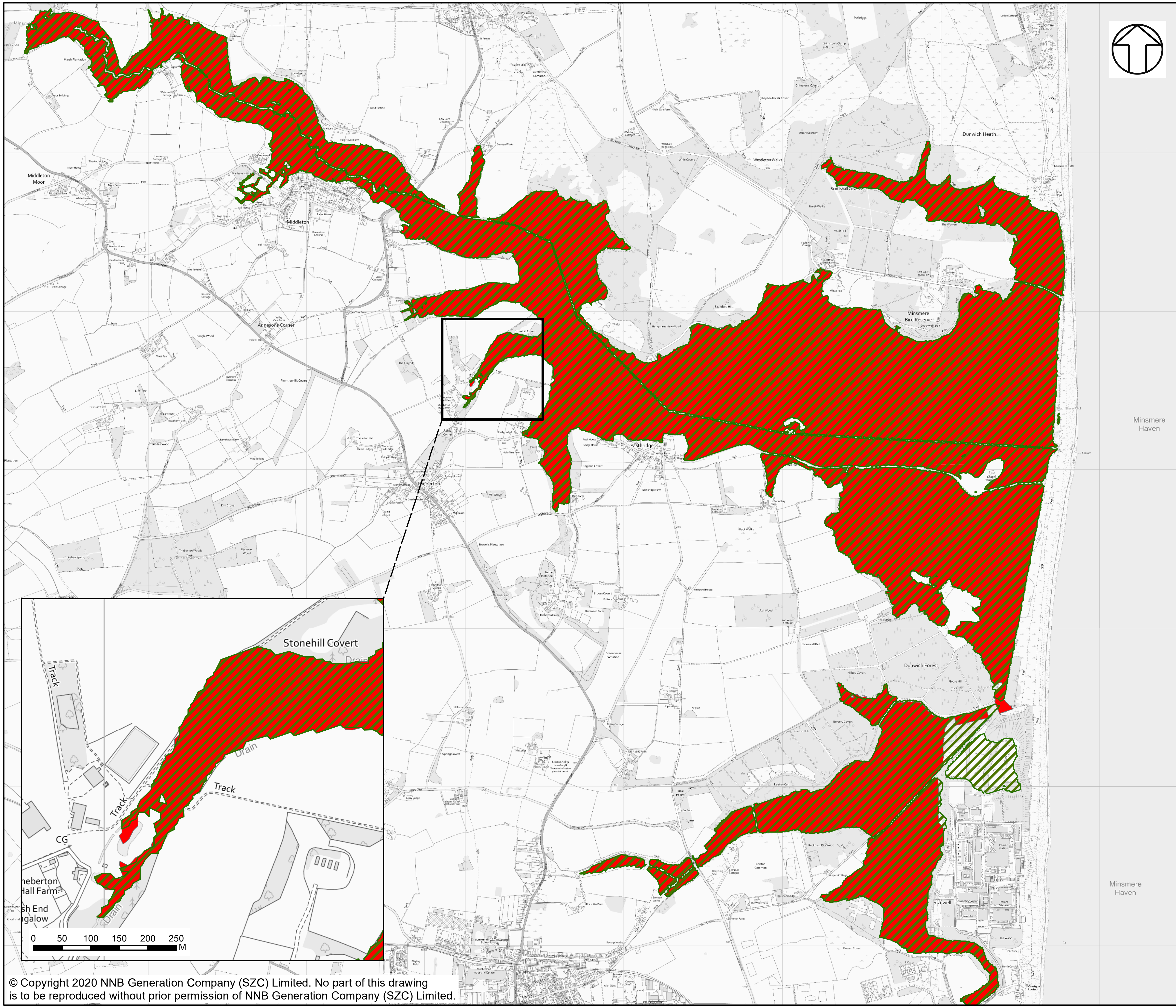

NOTES

KEY

FLOOD EXTENTS 1000 YEAR +35%CC

BASELINE FLOOD EXTENT 1000 YEAR +35%CC

WITH SCHEME FLOOD EXTENT 1000 YEAR +35%CC

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
FLOOD RISK ASSESSMENT
FLUVIAL MODEL RESULTS

DRAWING TITLE:
DIFFERENCE IN FLOOD EXTENTS
1000 YEAR +35%CC

DRAWING NO:
FIGURE E.26

DATE:	DRAWN:	SCALE :
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APPENDIX 3 - FLUVIAL HYDROLOGY REPORT

REPORT

SZC FRA - Hydrology Review and Design Event Methodology

Client: EDF Energy

Reference: PB6582_Hydrology_RP_001

Status: Draft/P03

Date: 03 March 2020

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Document title:	SZC FRA - Hydrology Review and Design Event Methodology
Document short title:	
Reference:	PB6582_Hydrology_RP_001
Status:	P03/Draft
Date:	03 March 2020
Project name:	SZC Flood Risk Assessment
Project number:	[redacted]
Author(s):	
Drafted by:	[redacted]
Checked by:	[redacted]
Date / initials:	28/02/20 HW
Approved by:	[redacted]
Date / initials:	03/03/20 FO

Classification
Project related

MANAGEMENT SYSTEM CERTIFICATION
DNV·GL
ISO 9001 = ISO 14001
OHSAS 18001

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Appendices

1 Introduction

This report has been prepared to summarise the hydrological assessment that has been carried out to inform the hydraulic modelling which is required to assess fluvial flood risk at the proposed Sizewell C new nuclear build development. The hydraulic modelling will form part of a comprehensive Flood Risk Assessment (FRA) being prepared as part of the Development Consent Order (DCO) application. The purpose of the fluvial hydraulic model is to test the order of magnitude of potential impacts on the fluvial system from the proposed Sizewell C platform.

This report builds on work completed for the Sizewell C FRA Scoping Report and early stages of the FRA between 2014 and 2017. It provides a summary of work carried out to date and gives greater detail related to the hydrological methodology. It summarises the hydrological calculations undertaken to derive the design event hydraulic model boundaries to inform the fluvial modelling for the main Sizewell C development FRA.

1.1 Overview of hydrological features

The proposed Sizewell C site is located in eastern Suffolk within the catchment of the Rivers Minsmere, Leiston Drain and Scotts Hall Drain, with a combined catchment area of approximately 80km^2 (see **Figure 1.1**). The catchment has a variable soil composition, is predominantly rural and receives relatively low annual rainfall of less than 600mm.

The River Minsmere rises south-west of Halesworth before flowing eastwards, bypassing the villages of Yoxford and Middleton. Downstream of Eastbridge, the embanked Minsmere New Cut flows through the Minsmere Levels Site of Special Scientific Interest (SSSI), whilst the Old Minsmere River drains the northern areas of the RSPB Reserve, re-joining the New Cut just upstream of the Minsmere Tidal Sluice structure.

The Scotts Hall Drain routes water from the northern and eastern areas of the Minsmere Levels towards the Minsmere Tidal Sluice. The Leiston Drain, a small watercourse in the vicinity of the Sizewell Nuclear Power Stations and the town of Leiston, drains the southern area.

The Minsmere Tidal Sluice drains freshwater by gravity through two outfall pipes discharging into the North Sea. The sluice structure has four flap gates (two for the Minsmere New Cut, one for Scotts Hall Drain and one for Leiston Drain). The main chamber is divided internally into two low level chambers separated by a wall (see **Figure 1.2**) over which water spills if it exceeds the top of the dividing wall at approximately 1.07m AOD. **Figure 1.3** shows the extents of the RSPB reserve meadows and surrounding floodplain features.

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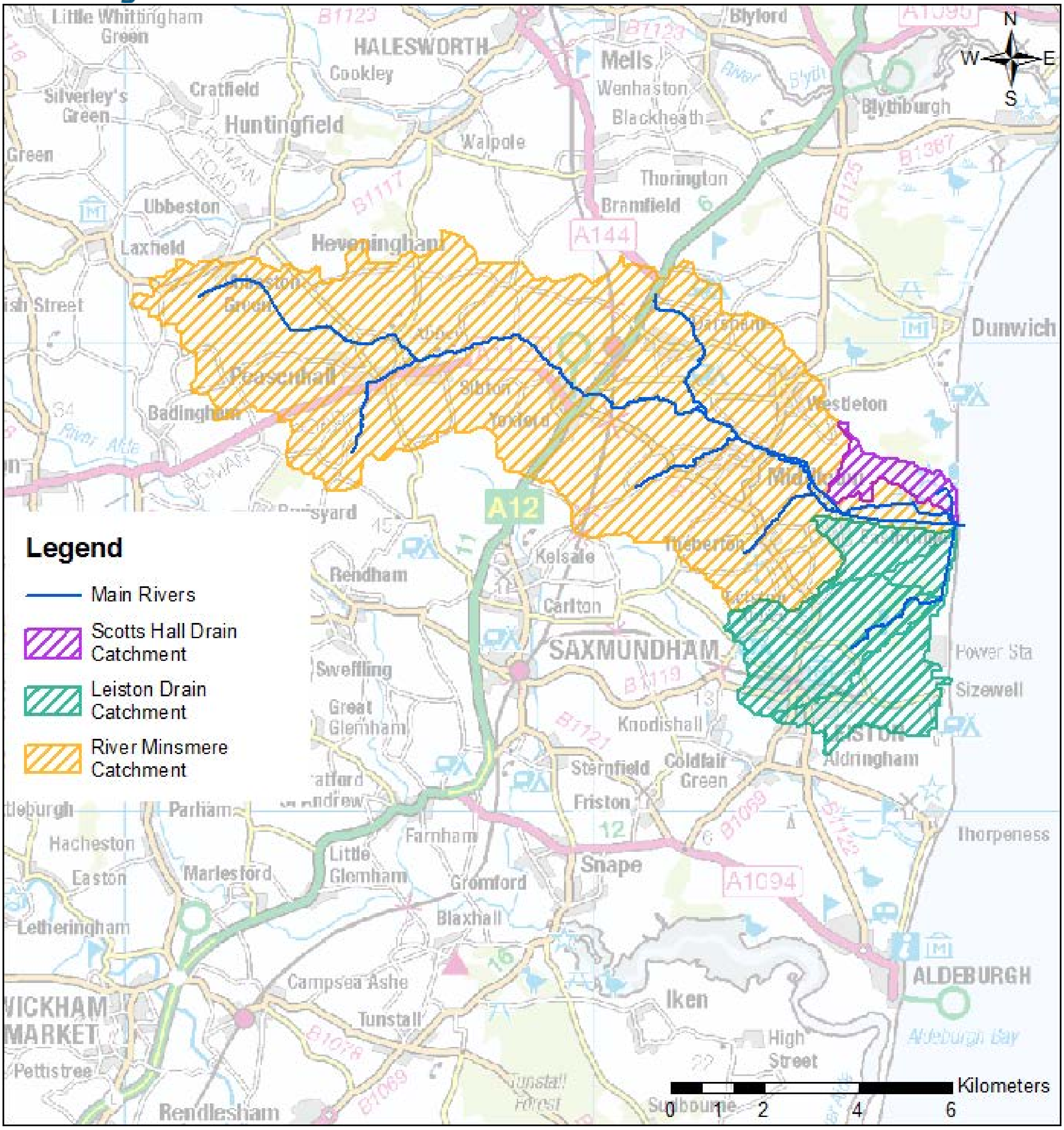

Figure 1.1. River Minsmere Catchment Area and Drainage Network

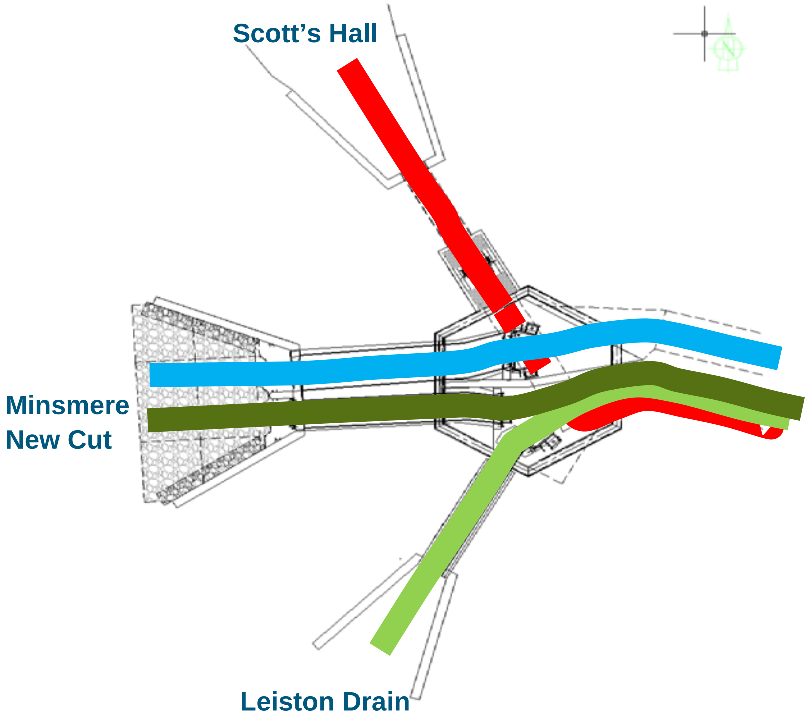

Figure 1.2. Minsmere Sluice Schematisation

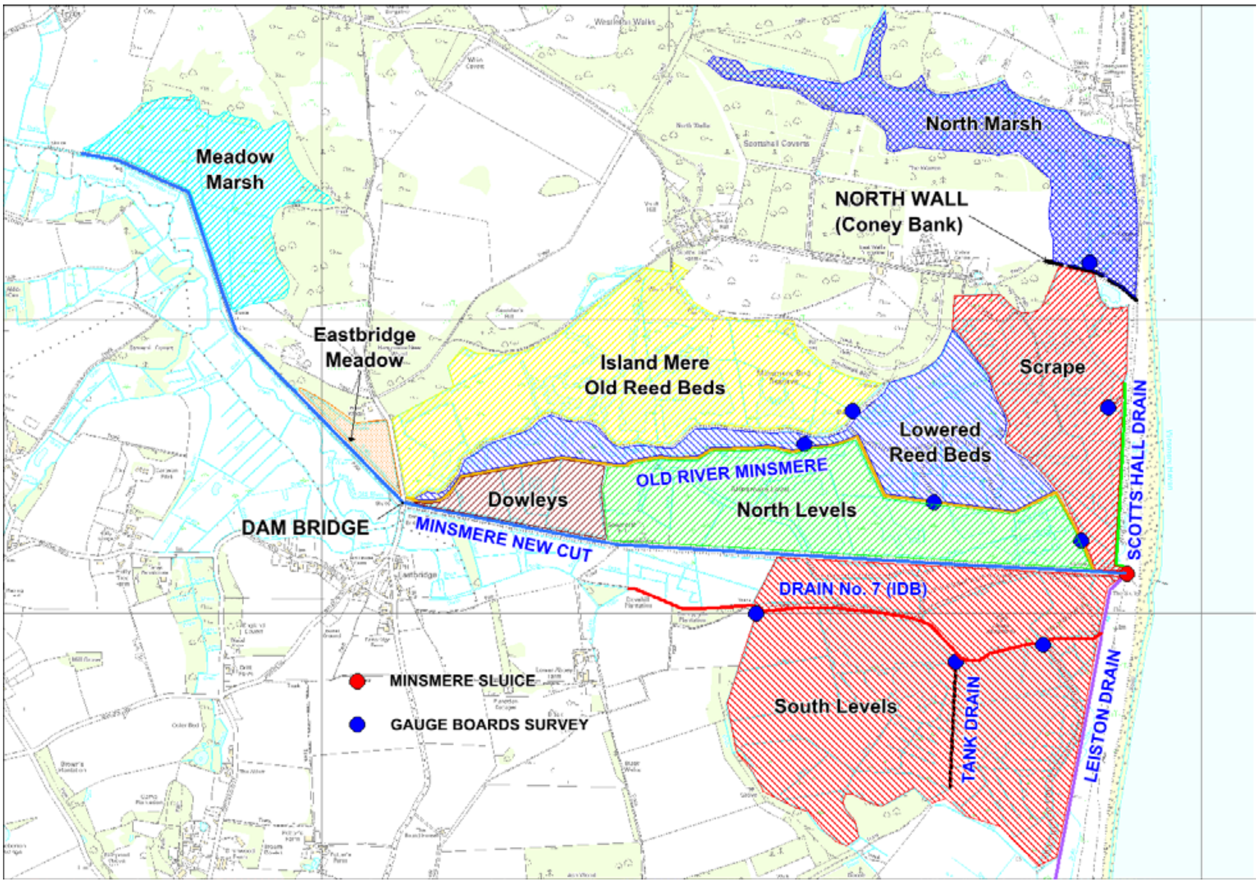

Figure 1.3. RSPB and Surrounding Floodplain Feature Location

1.2 Previous Studies and Environment Agency Discussions

A key existing reference point is the “Flood Study of River Minsmere and Leiston Drain” (Ref 1), hereafter referred to as the JBA 2013 Study. The JBA 2013 Study was commissioned by the Environment Agency, with a view to understanding the complex drainage pathways of the River Minsmere catchment and investigating their response to potential environmental change. The hydrological methodology within the JBA 2013 study was reviewed by Royal HaskoningDHV in 2014, on behalf of EDF Energy. The review referenced guidance given in the Flood Estimation Handbook (Ref 2) and relevant Operational Instructions (Ref 3). Emphasis was given to understanding the hydrological method adopted for the JBA 2013 Study and the justification for adoption of the chosen methodology. In particular, this review aimed to determine if additional analysis was required for the Sizewell C FRA.

The review pointed out that the hydrology used in the JBA 2013 Study was based solely on the Revitalised Flood Hydrograph (ReFH) rainfall-runoff method, as no suitable donor site to transfer data for a statistical single site analysis was identified. Data collected at the Middleton gauging station (station number 35022), which has been operational since 1976 was indicated, in the JBA 2013 study, to be of insufficient quality for this purpose. In such cases, common hydrological practice would be to undertake a pooling group analysis using a group of hydrologically similar gauged stations, however that was not undertaken in the JBA 2013 Study due to perceived lack of readily identifiable suitable donor sites. However, Royal HaskoningDHV proposed undertaking an additional assessment to find suitable donor sites for the catchment to test the statistical method.

Whilst ReFH is generally preferred over the FEH rainfall-runoff method, the ReFH technique has a number of limitations. For example, ReFH is recommended for use with caution for flow estimation beyond the 1 in 150 year event as it has not been tested for return period events that are longer than 150 years (Ref 3). In such cases, the ReFH method should be applied with caution and compared with the FEH statistical method. Another limitation of ReFH is that the technique is not suitable for catchments classed as permeable. A review of the FEH catchment descriptors indicates that for a number of the tributary catchments at Sizewell the BFIHOST value is above this threshold which would suggest that ReFH method may not be the most suitable. Therefore, a comparison of the rainfall-runoff methods was carried out.

The ReFH method was updated in 2015 to produce ReFH2, which improves the hydraulic modelling of the interaction of greenfield and impervious areas of the catchments. The majority of the changes between ReFH and ReFH2 focus on the hydraulic modelling of urban catchments and the application of hydraulic modelling at the development site and plot scale rather than the catchment scale. Further discussion on the site-specific comparison of the application of ReFH2 is set out in Section 3.2.1.

The downstream reaches of the catchment around Sizewell C, the Sizewell Belts and Minsmere Levels, are low and flat, and have a large attenuation effect in response to rainfall in the lower catchment. Therefore, it was proposed to also consider methods for lowland catchments, including the pumped catchment hydrology (Ref 4). However, this technique is intended for lowland systems considerably larger than the Sizewell study area. Furthermore, it is also recommended for catchments that have a means of calibration, using either records of pump rate or outflow rate. As there is no data record of outfall rate through the Minsmere Sluice, it is not possible to calibrate and adjust the flood volumes produced using the Pumped Catchment Method and therefore is not considered an appropriate methodology for the current study.

An alternative method was therefore required to improve on the representation of inflows to the lowland system (i.e. land within the 3m AOD contour) and it was proposed that Direct Rainfall is applied to the hydraulic model. This technique has the added benefit of allowing the 2D model domain to route the water through the system, better representing flood mechanisms including lakes and embankments. A hybrid approach to the hydrology is therefore considered reasonable due to the variable nature of the catchment.

For more details on the hydrology review from the JBA 2013 Study, refer to the Technical Note on hydrology review and proposed methodology for the EDF SZC FRA (Ref 5).

Following issue of the Technical Note above, the approach to the hydrological analysis was discussed at a meeting with the Environment Agency and other Stakeholders, held in Ipswich on 30th January 2015, where a number of comments and actions were raised relating to the hydrological analysis. These were considered and the proposed modifications to the original methodology were presented in Technical Note on hydrology update, issued in August 2015 (Ref 6).

A summary of the approach to the hydrological assessment for Sizewell C carried out to date is described in the following sections, including changes arising from consultation with the Environment Agency.

2 Data Analysis

In accordance with the current Environment Agency's Flood Estimation Guidelines (Ref 7) and the Flood Estimation Handbook guidance (Ref 8), the approach to the hydrology assessment should maximise the use of all available data, including anecdotal and published evidence from historic events. More accurately simulating observed flood events will help improve confidence in the model results.

Although there is a limited amount of reliable data for use in the hydrology study for the Minsmere catchment, a number of datasets were obtained and analysed. This section summarises all available data collected and provides a review of their suitability for use in the hydrological assessment.

2.1 Historic Flood Events

Public records and datasets relating to historic flood events, which should be used to provide suitable validation and to ensure confidence in modelled results, were sought and reviewed where available. These records are necessary to ensure that observed flood mechanisms are replicated within the hydraulic model, however, the records available for the study area are limited.

The most memorable flood was the tidal event in 1953 which caused damage to low lying areas across east Anglia, including Suffolk, however, the flood extent within the study area is unconfirmed. While the Suffolk Local Flood Risk Management Strategy (Ref 9) reported another significant tidal surge in December 2013, which flooded over 200 properties, roads, infrastructure and farmland across Suffolk.

The Preliminary Flood Risk Assessment (Ref 10) was updated in 2017 to reflect the recent flood history. Of the seven reports, none were relevant to the proposed Sizewell C development. However, while the town of Leiston is not within the study area, it is located on the edge of the catchment. Leiston was reported to have experienced significant surface water flooding. The surface water event of 8th July 2012 flooded approximately 25+ properties. Subsequent events have occurred on 13th October 2013, 27th May 2014, and a further six events in 2016. These multiple events resulted in the preparation of the Surface Water Management Plan for Leiston. While none of these events were within the study area, the proximity of Leiston means the study area is likely to have also experienced a similar event.

The SFRA (Ref 11) for this area focuses on urban areas, as is the nature of these assessments. The SFRA provides a chronology of flood events within Suffolk, while there is no specific mention of the catchments, the events listed in **Table 2.1** may have had an impact on the site through either tide locking leading to fluvial inundation or fluvial flooding.

Royal HaskoningDHV

Table 2.1. Extract of 'Table 3-1 Recorded Flood Incidents in East Suffolk' from the East Suffolk Level 1 SFRA that may have affected the fluvial study area

Date	Source	Location	Description (as recorded)
1888	Tidal	East Suffolk Coastline	Coastal flooding arising from the North Sea
1953	Tidal	East Suffolk Coastline	Coastal flooding from the North Sea following a full northwest gale and a swelling tide. Coastal flood defences were breached in 1,200 locations. 5 people were killed in Southwold and 39 in Felixstowe. There were 700 flooded properties in Felixstowe, 30 in Southwold and 400 in Lowestoft. Railway from Lowestoft to Norwich, main road in Aldeburgh and railway station in Woodbridge all closed/abandoned.
1976	Tidal	East Suffolk Coastline	Major tidal surges
1978	Tidal	East Suffolk Coastline	Major tidal surges
Feb 1993	Tidal and Fluvial	East Suffolk Coastline	Combined flooding due to a number of low-pressure systems passing the area and generating runoff from saturated catchments, resulting in £250k of damage.
Oct 1993	Pluvial and Fluvial	East Suffolk Coastline	Pluvial and fluvial flooding in the area leading to the damage of 67 properties
1995	Tidal	East Suffolk Coastline	£800k damage to the area due to widespread tidal flooding
2007	Tidal	East Suffolk Coastline	Strong winds, high tides and a storm surge resulted in extensive flooding where 6 homes were flooded in Southwold.

The Sizewell C Main Development Site Surface Water Conceptualisation Report (Ref 12) confirms that both the Leiston Drain and the Minsmere River are in low, flat valleys that are naturally wet and have water levels in the surface water drains that are controlled and regulated to maintain a wetland habitat for environmental management purposes. Therefore, with limited information and records about historic flood events, the local observations of the flood flow mechanism have been mostly based on anecdotal and historic observations from the RSPB.

In relation to the flood flow observations, two key pieces of evidence have been used to define the key flood mechanisms. These are 'mechanisms of flooding of the '*Freshwater flooding and drainage at Minsmere RSPB Reserve*' (Ref 13) Report; and observations from [redacted] of the RSPB (summarised in an email dated 27th October 2014).

The 2006 Black & Veatch Report (Ref 13) states:

"The following account of flooding processes within the RSPB reserve is taken from personal communications with Minsmere RSPB site manager [redacted]

- RSPB note that flooding tends to occur because the Minsmere sluice gates close at high tide and there is not enough storage in the watercourse channels to contain the water coming down the river system. At times of flood or when the sluice cannot drain, water spills back from the sluice for around 400-500m and spills over the lowest areas of the banks;

- A small washland (c81) is positioned south of the New Cut, where the bank level is artificially low. This is filled when water backs up from the sluice. However, it is too small to contain all flood events;
- When flooding occurs, the New Cut (more so than Leiston Drain) overtops and floods into the North levels (where horses are) opposite the washland at c81. The northern bank also has several large holes in it which allows water into the northern extent of the reserve. These are scheduled for repair by the Environment Agency – It is believed that these works were undertaken by the Environment Agency as part of the Minsmere Sluice and Embankment Works in 2014-2015;
- Floodwater can also overtop further upstream towards Eastbridge Levels which run along Drain #7. The rear gardens of houses in Chapel Road, Eastbridge have been affected by flooding in the past when Drain #7 overspills. Floodwater overtopped upstream of Eastbridge then passes down Drain #7 and can overtop the banks of Drain #7 onto the south levels over the southern bank into compartments 87, 92 and 96;
- Once flood water has entered the washland c81, water can then overtop the southern bank of the washland (c81) and run southwards across compartment c83. This then drains into Drain #7 and then again into the southern levels via the route above;
- Further upstream, Meadow Marsh (c40) and c41 can also be flooded by the New Cut overtopping its northern bank;
- Drainage of the Southern levels and the Northern levels cannot start until the water levels in the New Cut, Leiston Drain and Drain #7 are low enough to allow gravity flow.

A summary of the typical flood chronology as described by [redacted] from the RSPB (meeting date 22nd October 2014) includes:

- Flooding upstream of Dam Bridge in Eastbridge Meadow (the triangular area immediately North of Dam Bridge which is a separate hydrological unit from Meadowmarsh – the Africa-shaped block of reedbed and fen to the north-west) often occurs first as flows increase (however recent works by the Environment Agency to the sluice upstream of Dam Bridge have occurred which may change this – although it is thought it will not change the mechanism). Recent experience indicates that Eastbridge Meadow is still flooding due to overtopping of the New Cut bank despite repairs to the Environment Agency water control structure into the New Cut;
- Water levels in the North Levels will rise (Old Minsmere River) due to backing up from the east (Minsmere New Cut levels at the sluice will impede drainage) and water will begin to flow into the lowered reed beds over the banks into these areas (water does not flow through the pipes unless the upstands are out - this is opposite the washland);
- Water levels in the South Levels increase due to water spilling from the Leiston Drain and over time, Drain #7. This is due to backing up from the Minsmere Sluice Complex and not from a direct result of water from the Minsmere New Cut (water in the New Cut is the main driver though as this has the largest capacity of all of the drains exiting at the sluice - this is after water spills from Drain #7);
- As water levels rise, water will flow across the visitor trail between the Minsmere River and the Scrape, and into the Scrape;
- Flooding occurs over part / all of ~450m of the left hand bank (northern) of Minsmere New Cut immediately downstream of Dam Bridge at the lowest bank levels along the watercourse (the bank levels have been increased as asbestos has been removed / capped in specific locations between Dam Bridge and Minsmere Sluice, however it is expected that the flow mechanisms into North Levels will not change due to these changes – see locations on drawing 109417-0010AB General_Arrangement.pdf). It is possible that higher bank levels along the northern side of the New Cut may lead to increased overtopping along the southern side;

- As water levels in the system back up from Minsmere sluice, water will spill into the North and South Levels over low points of both banks of the Minsmere New Cut before water levels fill the system and equalise.

These two commentaries provide similar anecdotal observations on the flood mechanisms and have been referenced when developing the hydrological assessment and within the calibration of the hydraulic model.

2.2 Middleton Gauging Data and Rating Curve

The original methodology (Ref 5) stated that the Middleton gauge would be dismissed for use in statistical analysis, due to drowning and bypassing of the gauging weir at higher flows. Concerns regarding the reliability of the Middleton gauge record is also shared by the Environment Agency, who had low confidence in the existing rating curve for the gauge and stated that the gauge has a modular limit of only 0.4m. For this reason, the site is not included in the current National River Flow Archive (NRFA) Peak Flow dataset (previously HiFlows-UK, dataset 3.3.4) and therefore is not normally recommended for a single site statistical analysis.

Concerns raised during discussions with the Environment Agency regarding the effects of downstream blockage and backwater further confirm the Environment Agency's lack of confidence in this data source. However, since the Middleton gauge has a long data record (September 1976 onwards) and the gauge location "captures" or reflects the main flows into the lowland hydraulic system (representing approximately 60% of the total combined catchment area) it was considered that every effort should be made to maximise the use of this available data source, while understanding its limitations.

The gauge (NRFA 35022) is located at Middleton, near the upstream extent of the River Minsmere in the 1D-2D model reach, as illustrated in **Figure 2.1**. The model upstream boundary and inflow point is set slightly upstream from gauge location. Two tributaries join the River Minsmere upstream of the gauge, which are within the model extent for storage during extreme events. These tributaries do not contain inflows in the model, to avoid double-counting of their flow contributions into the system (which are already reflected in the gauge record at Middleton). The gauge has been operational since September 1976 with hourly stage data up to April 1993 and records at 15-minute intervals since then.

To provide maximum benefit to the hydrological analysis, it is necessary to convert the stage data series to flows. The Environment Agency initially provided a rating for the Middleton gauge. However, this rating functions only for low flows and is seen as unreliable for out of bank flows. Therefore, this rating could not be used for the calibration of the fluvial model. Instead, the 1D-2D linked hydraulic model was used to generate a new rating including floodplain flows. The hydraulic model also makes some representation of the mechanisms that may contribute to drowning of the weir. Using the hydraulic model to derive this rating is based on the assumption that the relationship between flow and water level is reasonable, since there is no downstream water level recorder to validate the downstream flow / stage relationship prior to the onset of drowning.

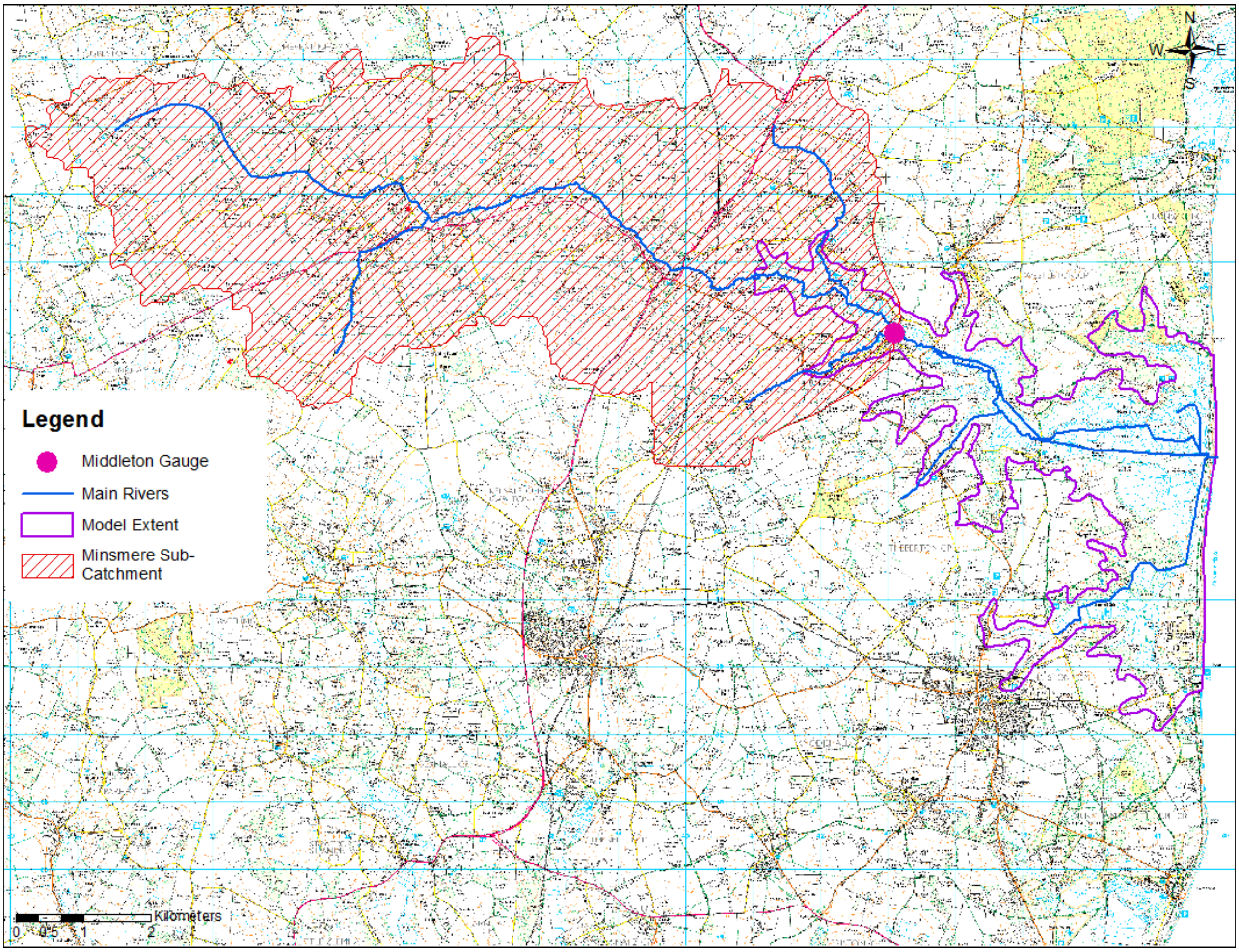

Figure 2.1. Location of the Middleton Gauge Station on River Minsmere

The hydraulic model used for the rating derivation was run with a static tide boundary (set to 0.00m AOD) and baseflow inputs in all other 1D elements of the model other than in the Minsmere New Cut. In the Minsmere New Cut an increasing flow ('stepped' hydrograph) was used, as shown in **Figure 2.2**.

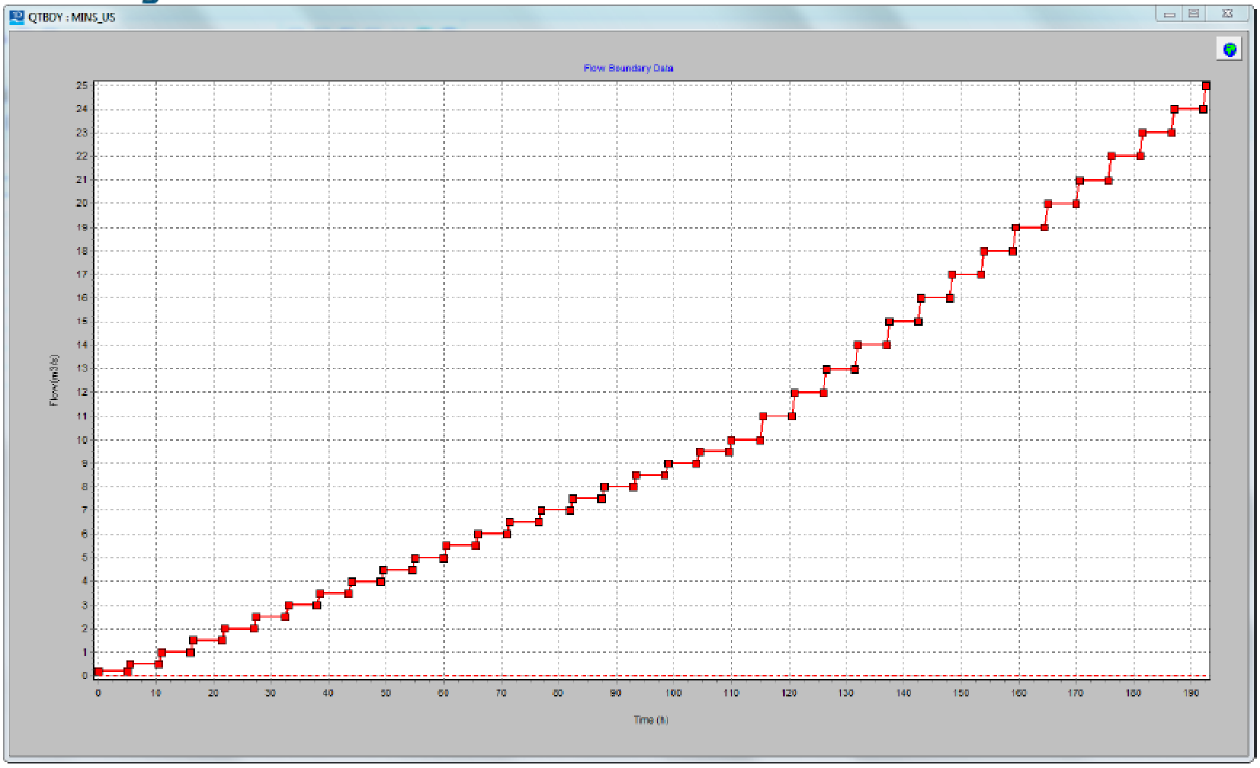

Figure 2.2. 'Stepped' Model Inflow adopted at Minsmere New Cut

The model was tested for a range of different roughness coefficients (0.045, 0.055, 0.065 and 0.085 between the model upstream limit and Dam Bridge) to determine sensitivity to this parameter. These sensitivity tests show that channel roughness has a significant impact on the flood mechanisms and rating relationship. A channel roughness of 0.045 has been chosen for the rating as this resulted in flood mechanisms most closely described by RSPB and other anecdotal information. It is also broadly commensurate with channel form and vegetation roughness observed during site visits.

Observations of the model results indicated that for the different roughness coefficients the flood mechanisms varied. The key observations for the stepped hydrograph are:

1) The first flooding mechanism observed for all roughness coefficients was water flooding into the washland ~600m to 1,000m upstream of Minsmere Sluice on the right-hand bank (as expressed in both commentaries by the RSPB).
2) The second flood mechanism for the lower roughness coefficient model runs (0.045 and 0.055) was water spilling from the Minsmere New Cut upstream of Dam Bridge into both Eastbridge Meadow (left hand bank) and Eastbridge Levels (right hand bank) upstream of Dam Bridge. Water spills into the Eastbridge Levels only and at a greater rate in the higher roughness coefficient scenarios (0.065 and 0.085). This second mechanism is also described in both RSPB commentaries.
3) The third mechanism in the lower roughness coefficient model runs (0.045 and 0.055) shows overtopping of the left-hand bank opposite the washland was observed at or just after flooding occurring upstream of Dam Bridge, as described in the RSPB commentary. For higher roughness coefficients this was observed at a later stage (however the model is run with a constant tidal boundary condition, therefore this may be seen to occur earlier in all events if a tidal boundary was applied).
4) The fourth mechanism was observed to show water spilling into Meadow Marsh over the left-hand bank of the Minsmere New Cut.

5) The fifth flooding mechanism observed from the models was water spilling from the right-hand bank of Drain #7 into the Southern Levels. Flooding into the southern levels was not observed from the Leiston Drain (as described in the RSPB commentaries). The mechanisms in this area were reviewed as part of the full model calibration, as this is not a focus area for the assessment of Middleton rating.

These findings indicate that the roughness coefficient (including density of vegetation / seasonality) can influence where water spills from the Minsmere New Cut. Different flood mechanisms may therefore occur at different times of the year for similar magnitude / duration / intensity events. However, the model is able to simulate the observed flood mechanisms for the Minsmere New Cut.

Photographs of the gauging weir are shown in **Figure 2.3** below, at both low and high flows. The lack of separation plates between the central flat-V and horizontal crump sections is unfortunate, as is the lack of a downstream water level recorder to calibrate the downstream channel geometry.

Figure 2.3. Photographs of Middleton GS (both looking downstream)

The rating derivation is presented below in **Figure 2.4** (normal scale) and **Figure 2.5** (log scale).

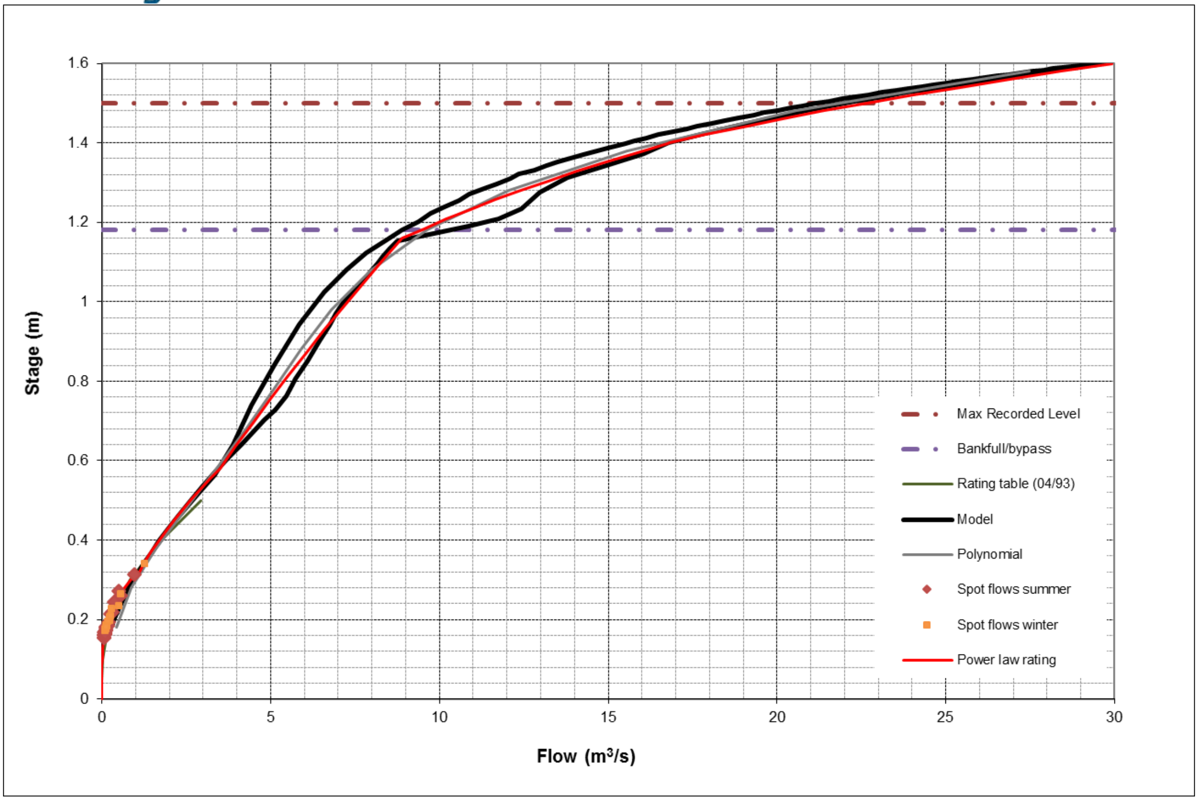

Figure 2.4. Middleton GS rating derivation (normal scale)

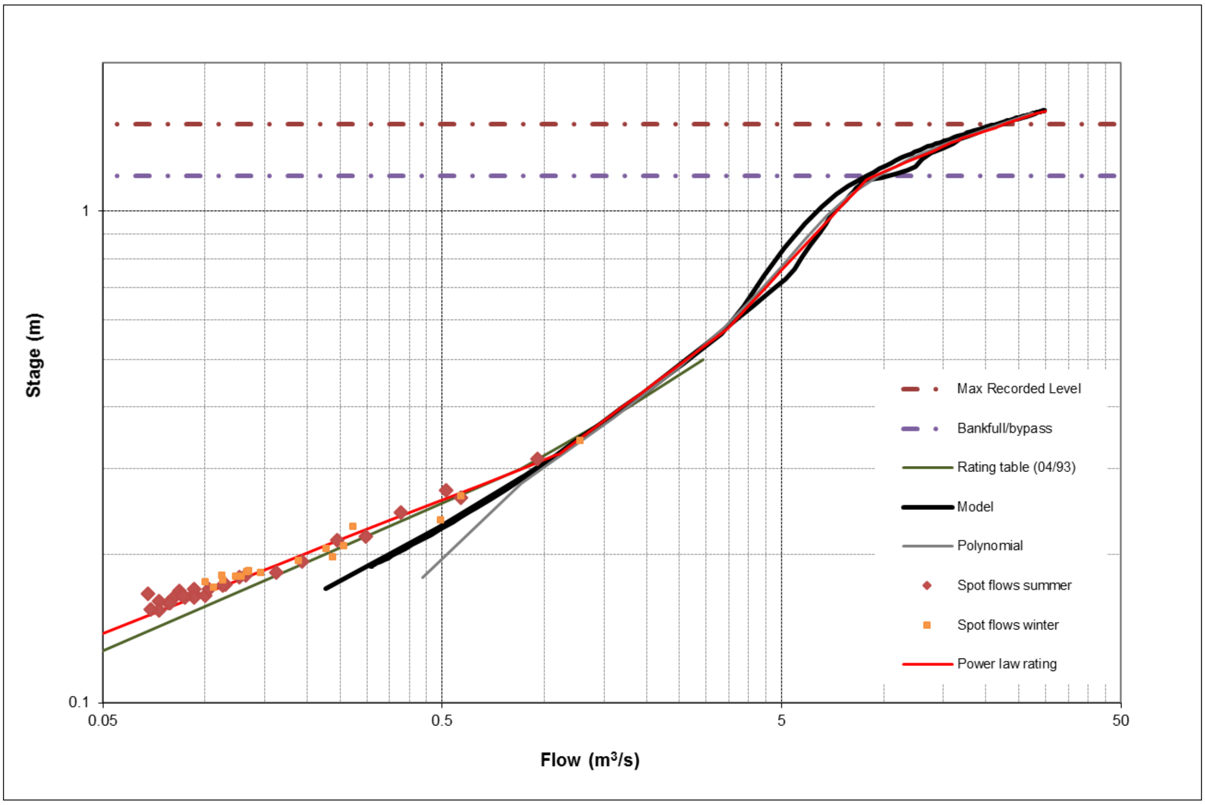

Figure 2.5. Middleton GS rating derivation (log scale)

The majority of the analyses to date used the Polynomial curve fitted equation as presented below:

$$Q=23.698*h^4-195.76*h^3+603.41*h^2-813.91*h+404.49$$

However, as part of the internal check/review process, it was found that the polynomial deviated from spot flow gauging at low flows. An improved power law equation was derived for future use, namely:

Table 2.2. Middleton GS power law rating parameters

Format Q=A*(h+B)^C up to SG Max:				
SG Min	**SG Max**	**A**	**B**	**C**
0	0.32	73.9028	0	3.6926
0.32	0.58	10.1041	0	1.9463
0.58	1.16	7.2879	0	1.3465
1.16	1.4	5.3904	0	3.3785
1.4	1.6	3.8976	0	4.3422

The derived rating curve / equations provides a means of deriving a flow series from the water level series supplied for the Middleton gauge. This allows assessment of the data (mindful of its potentially limited accuracy) to determine improved hydrological parameters such as time to peak (Tp), percentage runoff (PR) and Unit Hydrograph (UH) shape, which are described in subsequent sections of this report. These parameters have been used to modify and improve the other sub-catchment inflows.

The new rating curve is an improvement compared with there being no flow data, although it is acknowledged that a degree of uncertainty remains in this analysis. As there is no comparative rating curve and / or the available data is Stage only, it means there is limited data against which the derived rating curve can be validated. Results from preliminary modelling of calibration events indicate the flow derived from this rating, when input to the hydraulic model, provide good water level hydrograph shape reproduction at Middleton.

2.3 Leiston Temporary Gauge Data

Flow and water level data has been recorded on the Leiston Drain at a number of temporary gauges within the Sizewell Belts and Sizewell Marshes system, as illustrated in **Figure 2.6**. The data records for most of the gauges start on 27th November 2013, apart from G8 which is only available from May 2015. Flows are calculated from velocity sensors at G1, G5, G6 and G7A, with rectangular thin plate weirs used to calculate flows at G3 and G4. G8 is a water level only gauge. The gauge instrumentation used a Nivus PCM4 to measure the mean velocity. The Velocity Index and Ratings Report (December 2014) contains further information about each of the gauging stations and can be found in Appendix A.

Of most importance to the fluvial hydrology is gauge number “G5”, which was located immediately upstream of Lover’s Lane on the Leiston Drain, as this gauge location captures all of the flow draining from the catchment to the west of the Sizewell system. This gauge also records any flows originating from the Leiston Sewage Treatment Works (STW). During low fluvial flows, the diurnal pattern of the STW discharge is apparent, however, during increased flow events, the contributing flow from the STW is negligible. This reduces the importance of obtaining STW flow records.

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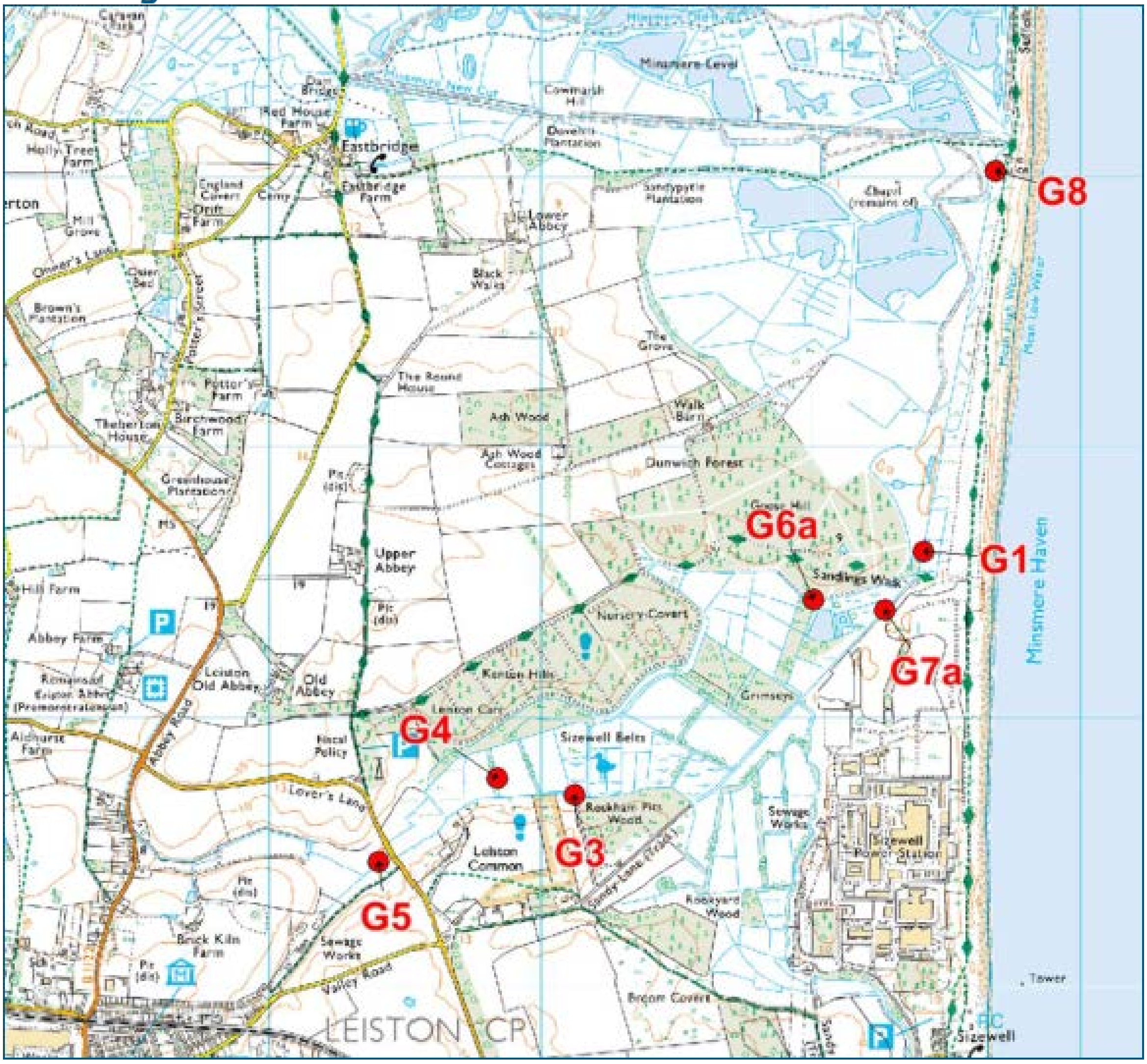

Figure 2.6. Temporary Gauge Locations

Despite only having a short length of record, the gauge has a number of important uses:

- QMED - Whilst ideally based on a minimum of two years of flow data, it is possible to calculate the Median Flow (QMED) using a “Peaks over Threshold” (POT) series. Based on 18 months of available data, when the hydrological analysis was carried out, this gave a QMED value of approximately 1.6m^3/s;
- Sensibility Check – The ReFH method was used to generate flows to the location of the G5 gauge. Based on catchment descriptors (cd’s), this gave a QMED flow of 0.3m^3/s and a 100-year return period flow of 1.1 m^3/s. These values were exceeded 17 and 3 times respectively in an 18-month data series, thereby indicating that the ReFH method based solely on cd’s significantly underestimates flows at this location;
- Improved rainfall runoff parameters – The availability of rainfall data from Thorpeness and a concurrent data series, enabled calculation of “real”, event-specific, PR values and generation of event-specific Unit Hydrographs. These were then used to improve the “theoretical” default values presented in FEH;
- Donor site – The catchment descriptors of the G5 gauge were found to be “hydrologically similar” to the other sub-catchment inflows (with the exception of the main Minsmere upstream inflow). It is

therefore assumed to be appropriate to use the G5 gauge as a "donor site" and "transfer" the modifications made to the default parameters to the other sub-catchment inflows.

Comparison of key catchment descriptors for all the sub-catchments is provided further in Section 3.1.2 and a full list of catchment descriptors is included in Appendix B.

The G5 gauge captures the seasonal variation in flows through this part of the catchment; however, it is noted that while showing increased fluvial flows there were no significant events which resulted in flooding in the catchment during the period of data collection. It is acknowledged that the G5 gauge is not an ideal match to all of the sub-catchments, however it has enough similarities to represent hydrological catchment response and, considering the lack of suitable data from other temporary gauging station, it was found appropriate to adopt G5 as donor site.

This data was analysed and used to inform model calibration and validation and to derive improved values for Tp, PR and UH shape using Strip_UH described in Section 3.1.3.

2.4 Calibration and Validation Data

Following analysis of the data collected from the Middleton Gauge and the temporary gauges, three separate events, January 2003, March 2010 and January 2016, have been selected for model calibration and validation. Calibration has been conducted using the January 2016 event as more detailed gauge data was available for the Leiston catchment (that was not available for earlier events). The two earlier events were therefore used for model validation. These were the only suitable events identified at the time of model calibration. Additional events might be recorded since, that could be included in the further study (if required) at the later stage of the Sizewell C project.

The updated rating curve for Middleton Gauge was used to derive flow hydrographs from the observed stage data for each of the events. **Figure 2.7** – **Figure 2.9** present the derived hydrographs for the 2003, 2010 and 2016 events respectively.

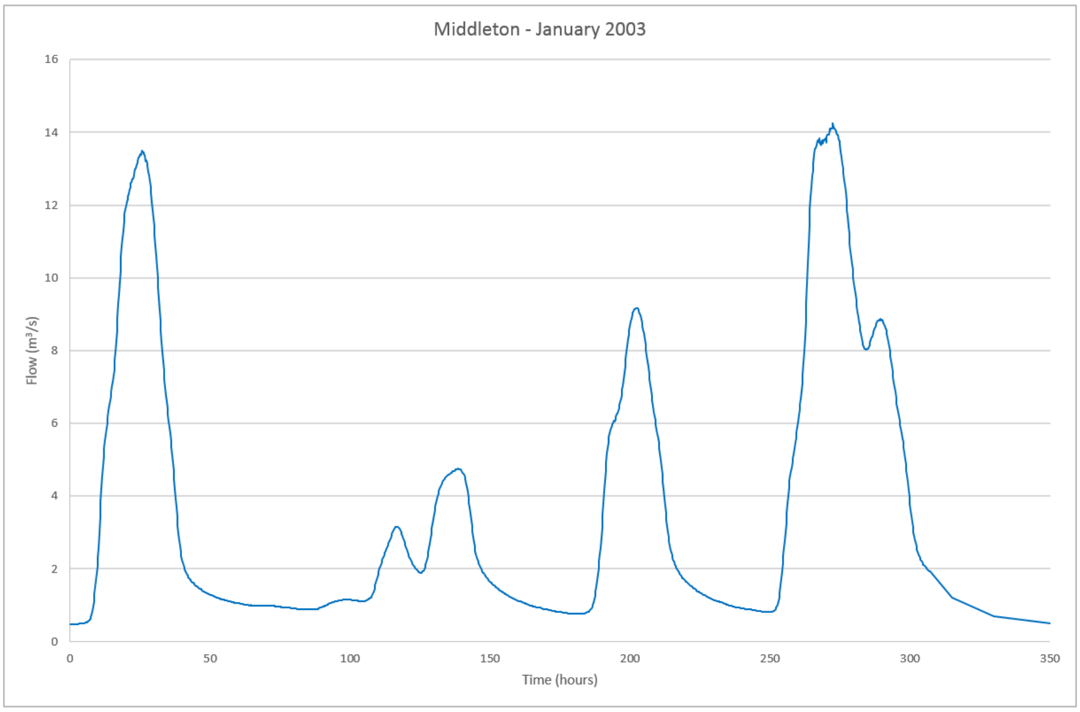

Figure 2.7. Flow at Middleton Gauge - January 2003 Event

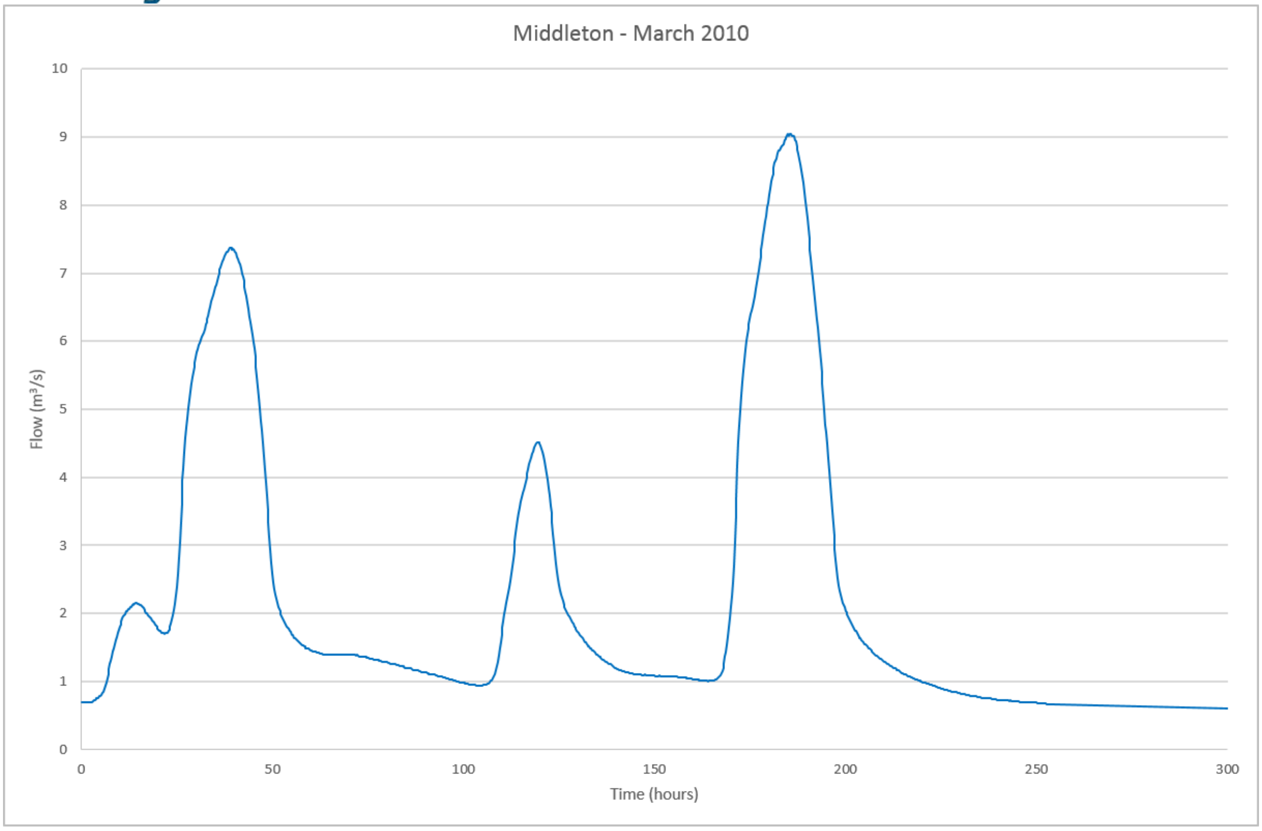

Figure 2.8. Flow at Middleton Gauge - March 2010 Event

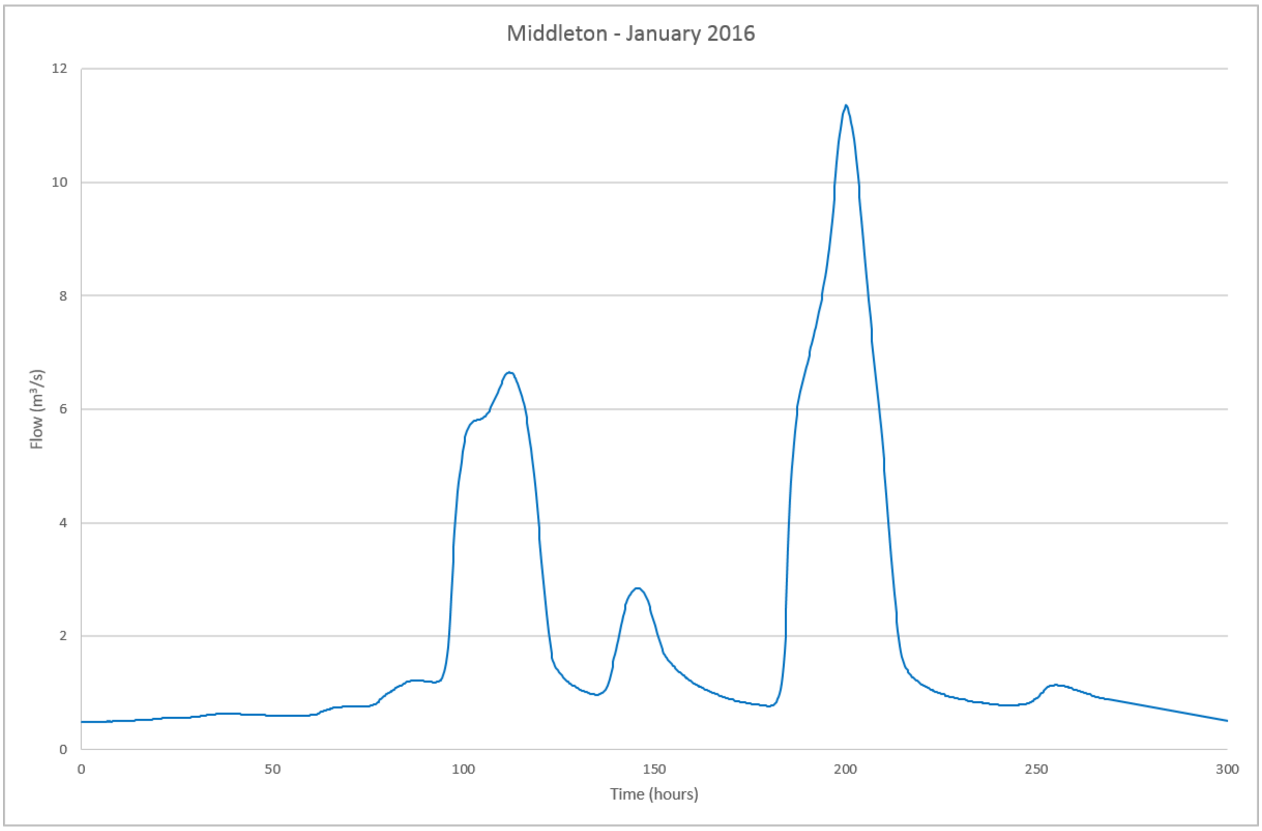

Figure 2.9. Flow at Middleton Gauge - January 2016 Event

Rainfall data for model calibration and validation were taken from two 15-minute tipping bucket rainfall gauges, located in proximity to the catchment. Observed rainfall data from Benhall gauge (for 2010 and 2016 events) and from Earl Soham gauge (for 2003 event) have been used in the model calibration, applied as direct rainfall to the 'Lowland' system.

Rainfall depths were checked against the daily rainfall gauge at Westleton to ensure that there were no anomalies in the volume of rainfall. The derived rainfall inputs for the calibration events enabled the use of a value of 50% for PR for the floodplain upstream of Eastbridge and a value of 90% for PR downstream of Eastbridge, as determined from analysis of the floodplain elevation and slope within the lowland area. Downstream of Eastbridge it is expected that the floodplain would be largely waterlogged during significant events and therefore would exhibit a high percentage runoff whereas upstream of this location, due to the higher (but still relatively shallow) gradient it was expected that there would be a lower runoff, hence the adoption of different PR values.

Tidal boundary data for the calibration event was derived utilising recorded tide data at Lowestoft. The recorded data at Lowestoft was translated to be representative of the tidal levels at Minsmere Sluice. Admiralty harmonics have not been used for this translation as the analysis of the outputs from the transformation and discussions with CEFAS determined that the Admiralty harmonics at Minsmere Sluice are not representative. Instead, high and low tide data for Lowestoft and Sizewell B stations (records from February 2009 to December 2012, supplied by CEFAS) have been utilised to identify a suitable relationship and transformation of tide levels from Lowestoft to Minsmere Sluice.

Figure 2.10 shows the three-limb linear relationship that has been adopted to transform recorded stage data at Lowestoft to Minsmere Sluice, which has been used to create calibration boundary conditions. The 'data' represents all available records, whereas 'Upper' and 'Lower' series show grouped high tide (above 0.5m AOD) and low tide (below -0.1m AOD) levels respectively.

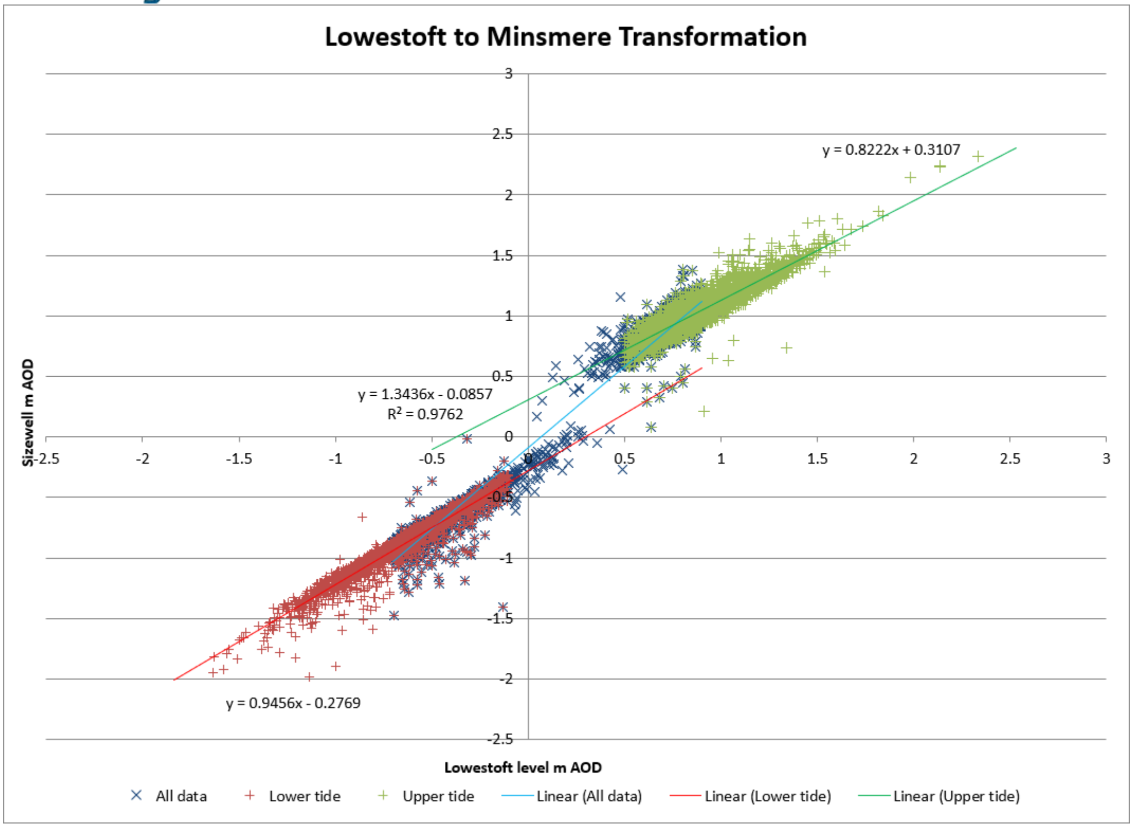

Figure 2.10. Lowestoft and Sizewell B Tide Relationship

In order to test the sensitivity of the model results to tidal boundary conditions, the raw Lowestoft gauge data (converted from m CD to m AOD) has been utilised for the 2003, 2010 and 2016 events, as illustrated in **Figure 2.11** – **Figure 2.13** respectively.

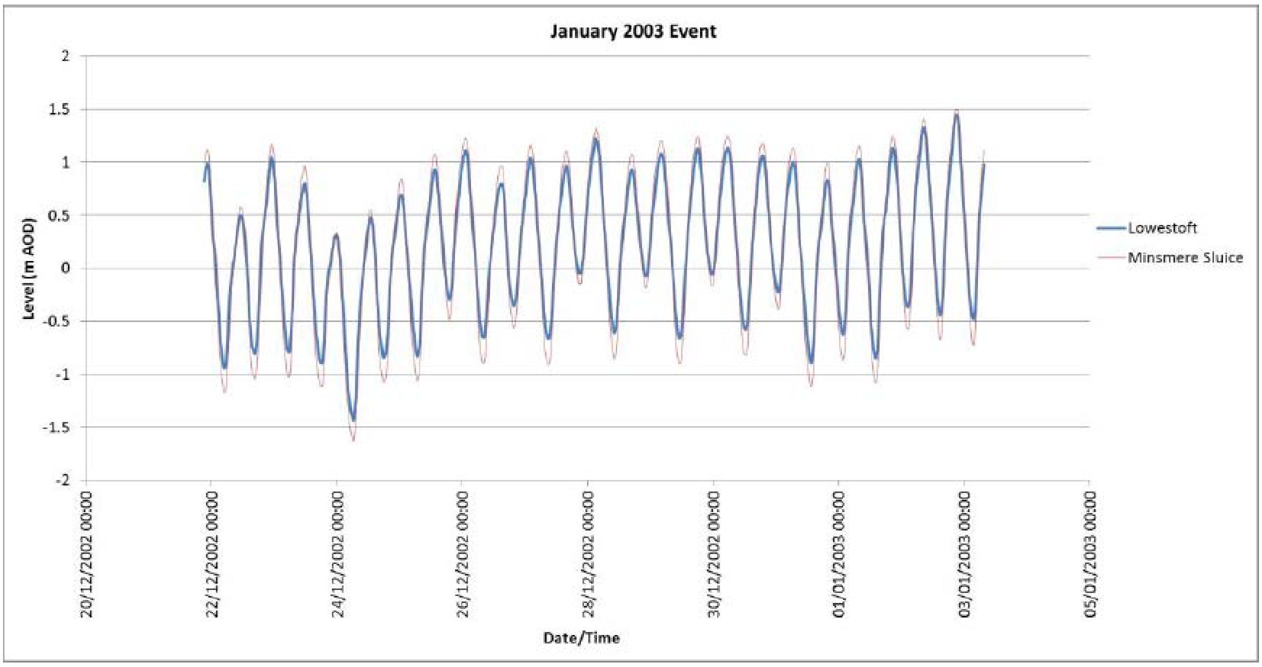

Figure 2.11. January 2003 Calibration Event Tide Data

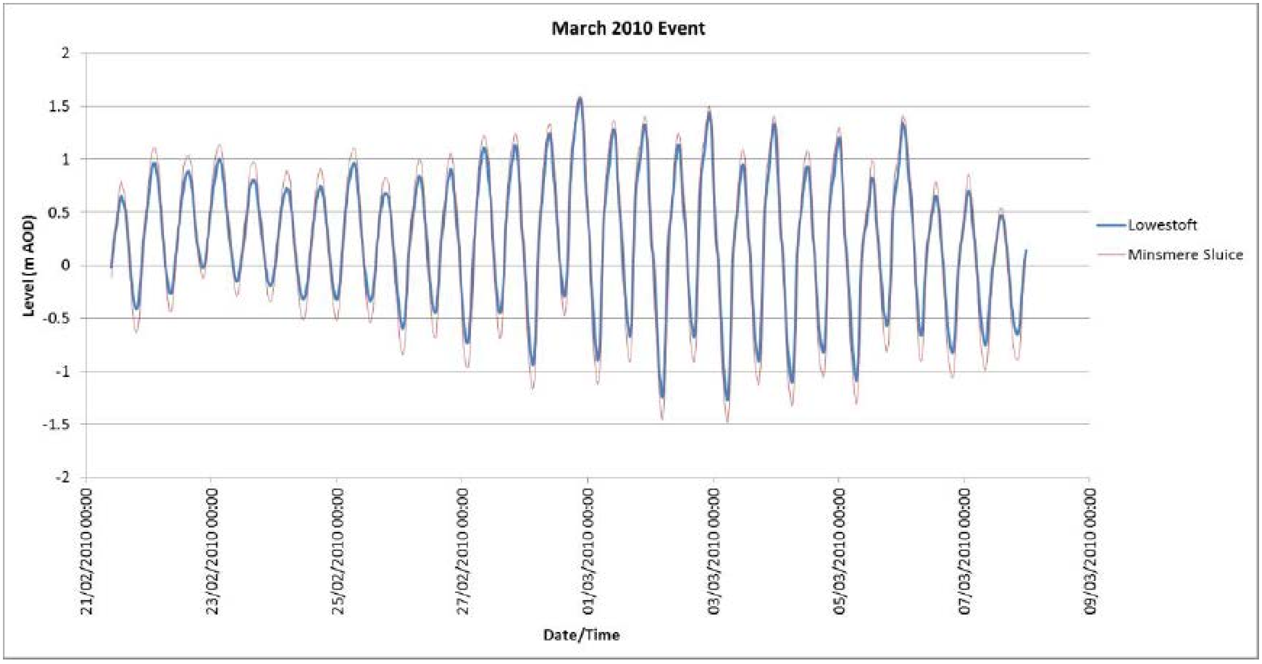

Figure 2.12. March 2010 Calibration Event Tide Data

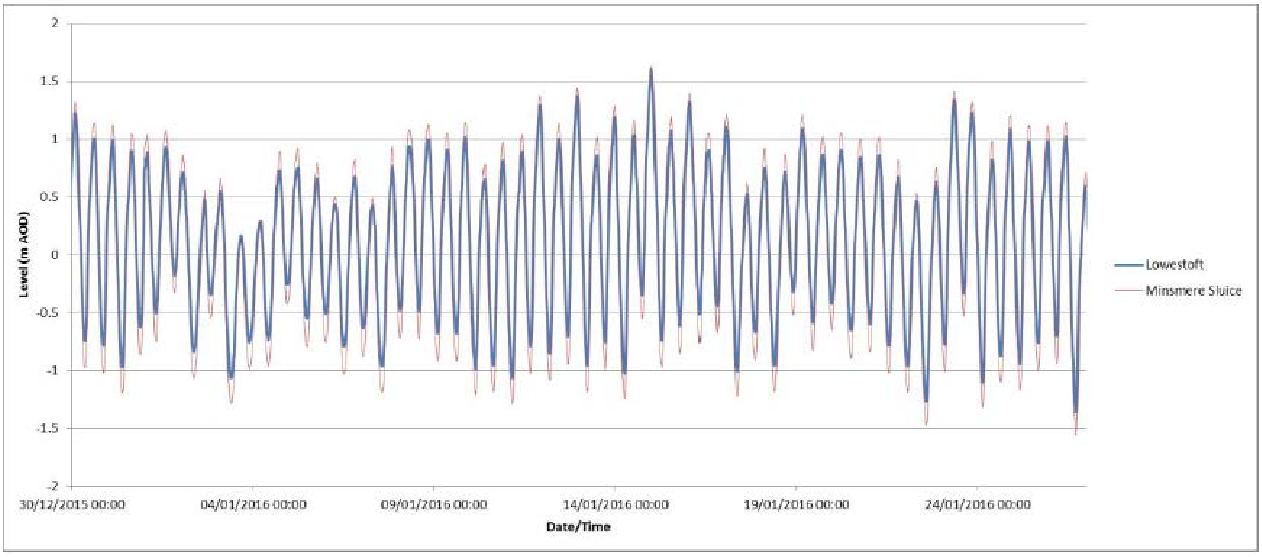

Figure 2.13. January 2016 Calibration Event Tide Data

The overall conclusion from the model calibration process is that calibration confidence is somewhat limited due to fluvial data availability and data quality. However, the model shows reasonable correlation with the available data and visual observations. The model is considered representative for the Minsmere and Leiston water systems at the rising limb of the flood hydrograph and is considered suitable for testing the relative impacts associated with the Sizewell C scheme. It is acknowledged that during peak flood conditions the model shows an overestimation of the water level for both Leiston and Minsmere systems and that results for gauge G5 near Leiston shows that the simulated flood levels are slightly underestimated.

Further details on model calibration and validation are available in the “Sizewell-C Fluvial Modelling Calibration” Report (Ref 14).

2.5 Minsmere Sluice Data

For a short period of time between 2005 and 2006 there is a limited dataset available for Minsmere Sluice comprising water levels near the sluice (i.e. at Minsmere New Cut and Scotts Hall Drain). **Figure 2.14** shows the water levels at Middleton, which is a good indicator for the amount of water flowing through the Minsmere New Cut and arriving at Minsmere Sluice. It also shows the selected calibration events (blue circles). The period of data available for Minsmere Sluice (2005-2006) is shown by the yellow box.

Peaks for the selected calibration events are higher than the peak level within the period of data availability for Minsmere Sluice. It is anticipated that the peak level in the period 2005 - 2006 is below bank full. The data period is prior to the renovation of Minsmere Sluice. Therefore, it was concluded that this data is not particularly suitable for calibration of the hydraulic model as there is no overlap with other gauging station data, comprises a relatively short time period and is thought to represent only in-bank flow conditions.

The Sizewell C project was placed on hold between late-2016 and mid-2018. The collection of hydrological data was carried out prior to this and there was no extended dataset upon which the current hydrological report can be updated.

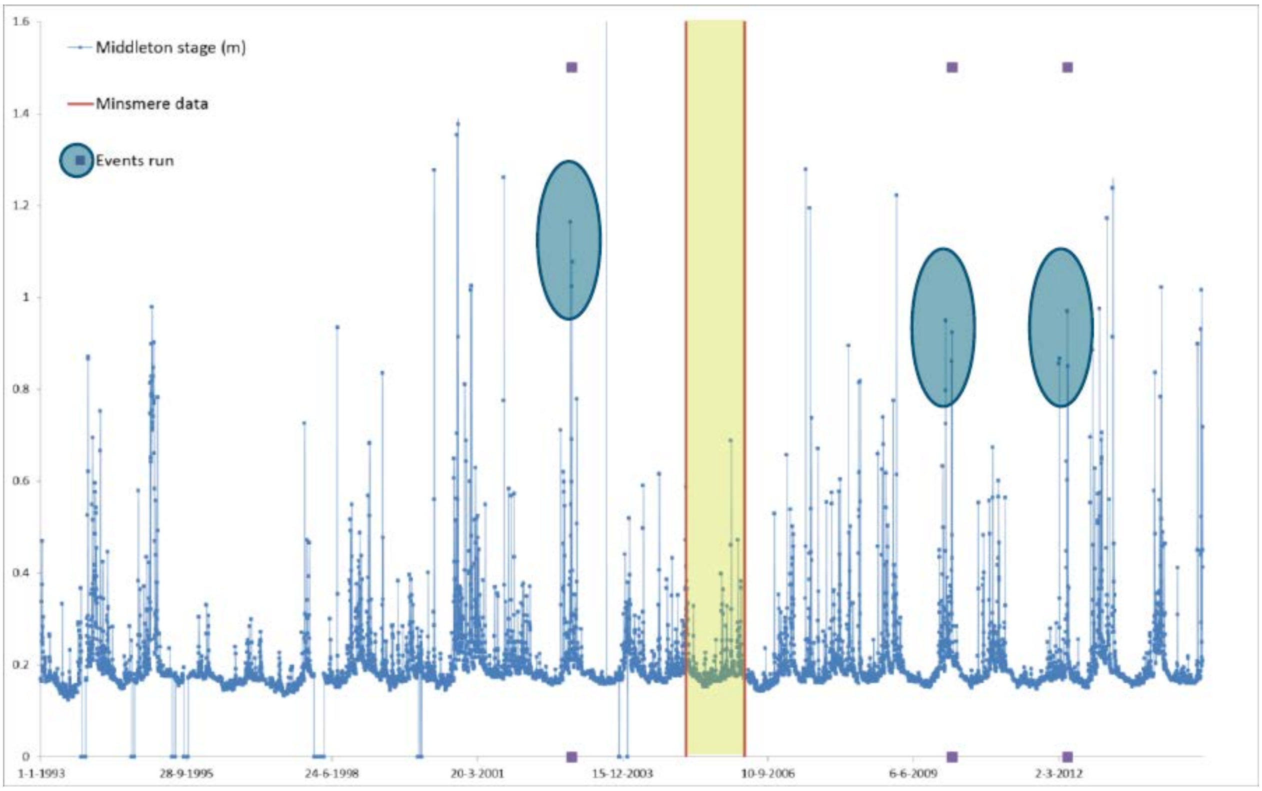

Figure 2.14.Time series of the water level at Middleton, showing the period in which Minsmere Sluice data is available

3 Hydrological Approach

In Section 1.2 it was identified that the original approach proposed for deriving the hydrological inputs to the hydraulic model was presented in Section 3 of the "Sizewell C Hydrology Review and Proposed Methodology for the EDF SZC FRA", (Ref 5). This was then revised in line with comments received at the Stakeholders meeting as outlined in Technical Note: "Sizewell FRA - Hydrology Update" (Ref 6).

This section describes how the fluvial system of River Minsmere and Leiston Drain catchments was represented in the hydraulic model, provides a description on the proposed methodologies and the adopted approach to derive hydrological design conditions. It also summarises the derived climate change allowances for potential increase in peak river flows and rainfall intensity, as well as the sea level rise that was included in the model boundaries for the future epochs representing different phases of the Sizewell C development.

3.1 Design Event Peak Flow Estimation

3.1.1 Representation of the Fluvial System

Considering the characteristics of the River Minsmere and Leiston Drain catchments, the hydrological representation of the system differs for Upland and Lowland components and are defined as follows:

- **Upland Inflows** – Areas where ground level is greater than 3m AOD flows have been included in the form of hydrological boundary units in 1D Flood Modeller Pro (former ISIS) and as point inflows in the 2D TUFLOW model. Further details on the selection of the boundary unit type and derivation of the Upland Inflows are given in following sub-sections;
- **Lowland inflows** – Areas where ground level is below 3m AOD inflows are added in the form of Direct Rainfall to the 2D TUFLOW element of the hydraulic model. Further details on derivation of the Lowland Inflows are given in Section 3.2.3.

The upland system consists of the Minsmere Gauge catchment, Leiston Drain catchment defined at the G5 gauge and other upland sub-catchments for which the G5 gauge was used as a 'donor site' for the parameters adopted in the hydrological boundaries in the model, as outlined in Section 2.3.

Figure 3.1 shows the extents of adopted upland inflow catchments and the lowland direct rainfall area.

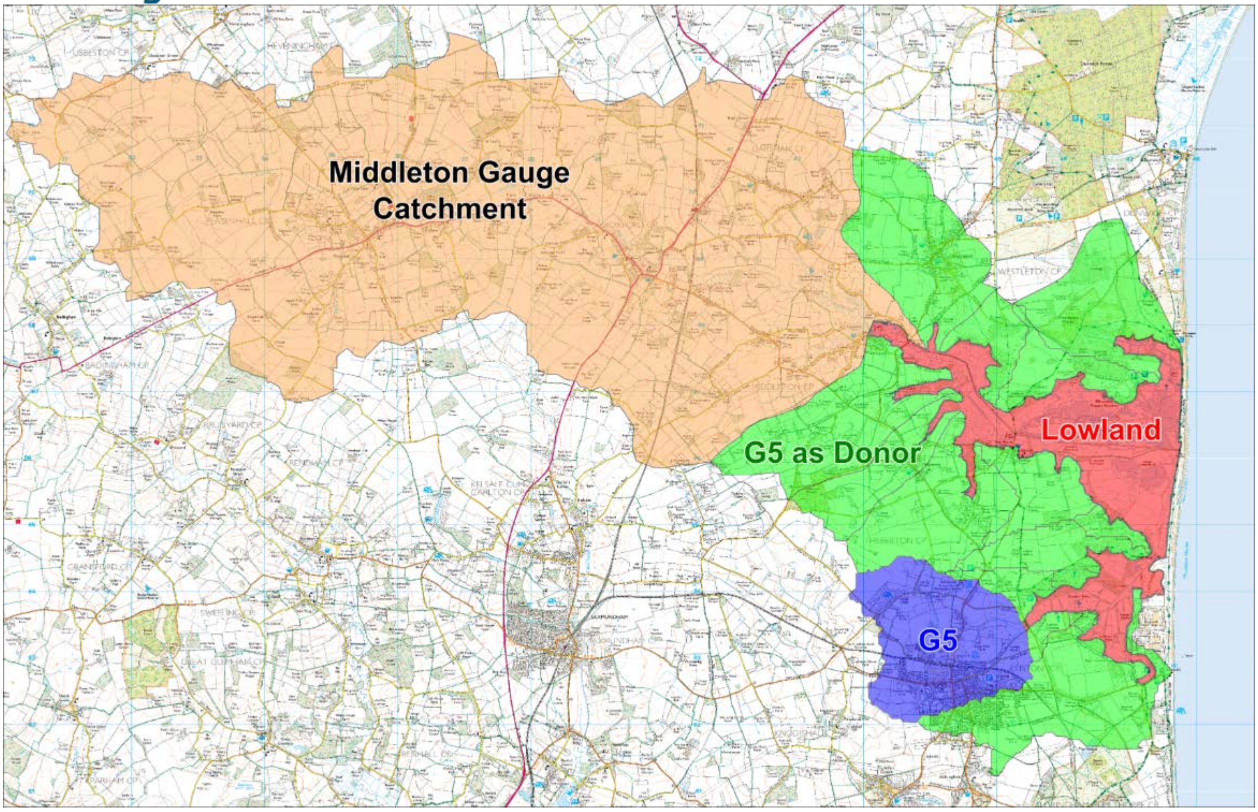

Figure 3.1. Extents of adopted Upland Inflow Catchments and Lowland Direct Rainfall Area

3.1.2 FEH Catchment Descriptors

FEH catchment descriptors (cd's) have been extracted from the FEH CD-ROM Version 3. These have been taken for the catchment as a whole and also the main inflow locations to the model from the relevant sub-catchments (i.e. the lateral upland inflows), as illustrated in **Figure 3.2**. The cd's were obtained up to the point where the hydraulic model utilised the Lowland Inflow approach. This is defined by the 3m AOD contour and the cd's are therefore considered to be representative of the upland catchment, and not affected by the flatter gradient within the lower catchment.

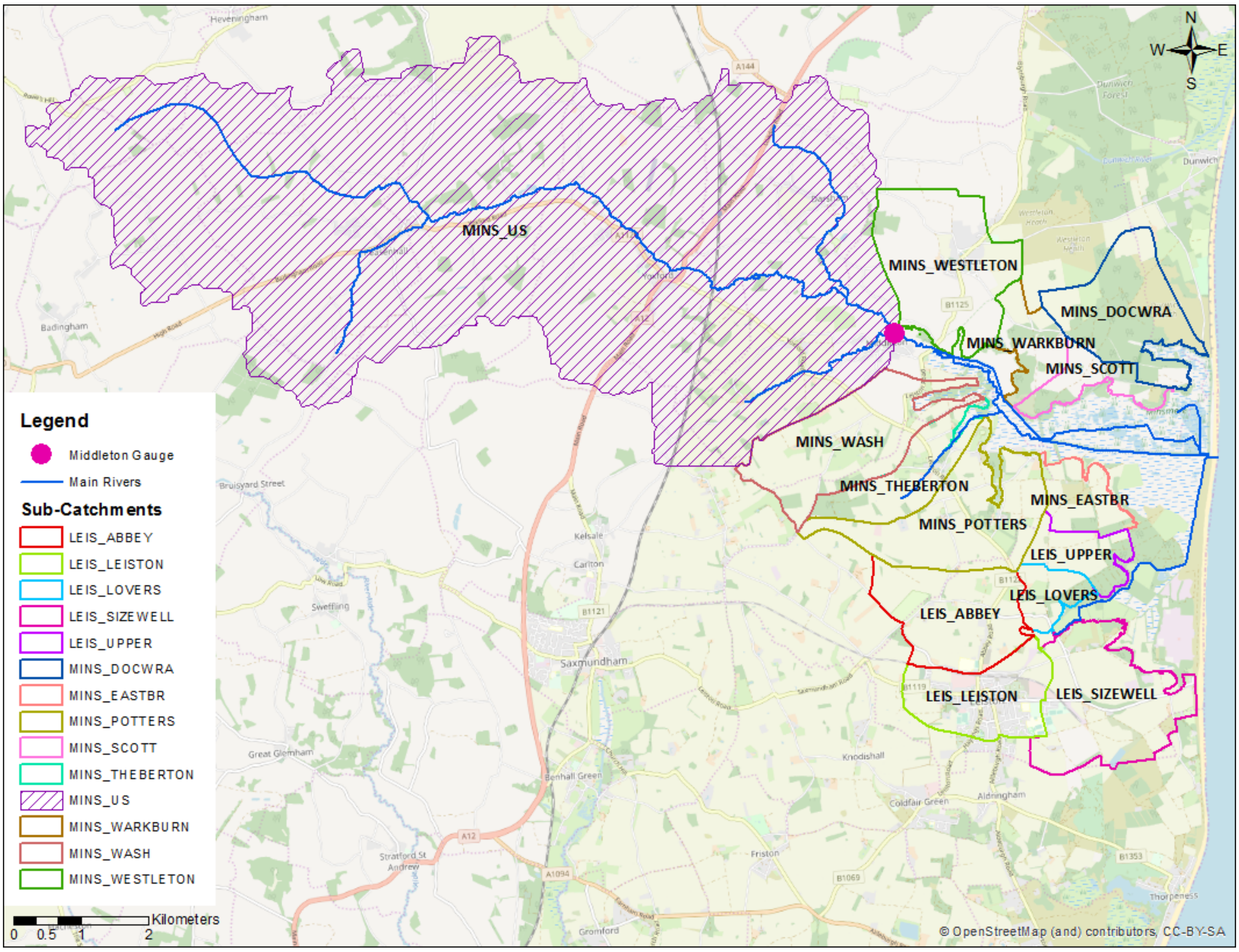

Figure 3.2. Location and extent of Upland Sub-Catchments

Due to the relatively flat topography within the study area, the AREA values provided in the FEH CD-ROM Version 3 were checked and amended using the more accurate data provided by LiDAR and Ordnance Survey mapping. In a number of places, the intervening areas have been added to the sub-catchment AREA, such that the sum of the sub-catchment AREA's equals the total AREA to the outfall.

It is acknowledged that other catchment descriptors, such as BFIHOST and SPRHOST could also be checked taking into consideration variation in soil characteristics between sub-catchments. Although it has been considered at the time of the study, more detailed checks and validation of adopted descriptors values could be carried out at the later stages of the Sizewell C project.

Table 3.1 presents key FEH catchment descriptors obtained from the FEH CD-ROM 3 for each of the sub-catchments. The full FEH catchment descriptors are available in Appendix B.

Table 3.1. Key FEH Catchment Descriptors for all the Sub-Catchments considered in the Sizewell C FRA Study

Sub-Catchment	AREA (km^2)	AREA adjusted (km^2)	BFIHOST	URBEXT1990	SAAR (mm)	SPRHOST
MINS_US	46.00	50.16	0.375	0.0084	594	41.05
MINS_EASTBR	1.02	1.23	0.881	0.0037	590	13.29
MINS_DOCWRA	2.39	2.86	0.898	0.0000	585	14.39
MINS_POTTERS	2.99	3.49	0.724	0.0050	595	25.33
MINS_SCOTT	1.58	0.98	0.886	0.0000	586	13.54
MINS_THEBERTON	1.95	1.71	0.630	0.0083	597	29.54
MINS_WARKBARN	0.83	1.09	0.893	0.0000	590	14.01
MINS_WASH	2.87	3.62	0.578	0.0000	598	32.44
MINS_WESTLETON	3.47	3.84	0.787	0.0209	595	21.94
LEIS_ABBEY	2.43	2.83	0.630	0.0072	594	27.83
LEIS_LEISTON	1.85	2.21	0.817	0.1671	592	20.71
LEIS_LOVERS	0.59	0.854	0.840	0.0000	591	18.21
LEIS_UPPER	0.72	1.229	0.855	0.0000	590	17.05
LEIS_SIZEWELL	3.92	3.644	0.890	0.0540	586	14.52

3.1.3 Strip-UH

The observed rainfall data and rated flow data at Middleton was used to derive improved parameters for Tp, PR and UH shape, using in-house developed software Strip_UH, that performs the FEH standard process of extracting a Unit Hydrograph from observed flow and rainfall data (“Strip” out a UH). There is limited hydrometric data available to supersede the use of catchment descriptors although adjustments have been applied to the hydrology, where possible.

Analysis of rainfall (various gauges) and flow data (Middleton) using the updated rating curve (see Section 2.2) indicated that for the events in January 2003, March 2010 and January 2016 the percentage runoff to the gauge was approximately 50%. Therefore, for calibration and validation of the model, the upland inflows within the model have been run with PR set to 50%. It is recognised that the recommended number of events used to derive the PR should be more than 3 (best case to use 8 events), however due to the very limited observable data in the catchment only 3 reliable events were identified at the time of the study. Additional events may be available since, but further assessment could only be carried out at the later stage of the Sizewell C project

Further sensitivity testing may aid in providing further understanding regarding the relative influence of the adopted PR values on the modelling outputs. However, due to the limited availability of appropriate data against which the model can be compared or verified, it would not provide greater confidence in the model results. Therefore, sensitivity testing was not carried out at this stage of works but would be recommended for future stages, subject to further appropriate data being available against which it can be compared and verified.

"Strip_UH" was also used to derive event specific UH's for the temporary gauge G5 used in the model calibration process. This provided a "real" UH against which to compare the accuracy of the "theoretical" UH produced from catchment descriptors in the FEH or ReFH. It was found that the Strip_UH analysis from temporary gauge G5 was very similar in shape to the ReFH UH. The ReFH UH for each of the upland inflow sub-catchments was therefore obtained and used in an ISIS FEH boundary unit for each sub-catchment. This is with the exception of the main upstream inflow for River Minsmere (MINS_US), where the UH was derived using Strip_UH based on the Middleton rating flows. The Strip_UH analysis of the temporary gauge data also showed that a PR value of 50% was "typical" for flood conditions and was therefore used within all the sub-catchment boundary units for the design events.

Derived Unit Hydrographs for the River Minsmere at Middleton Gauge and the upland inflow sub-catchments are presented in **Figure 3.3** and **Figure 3.4** respectively.

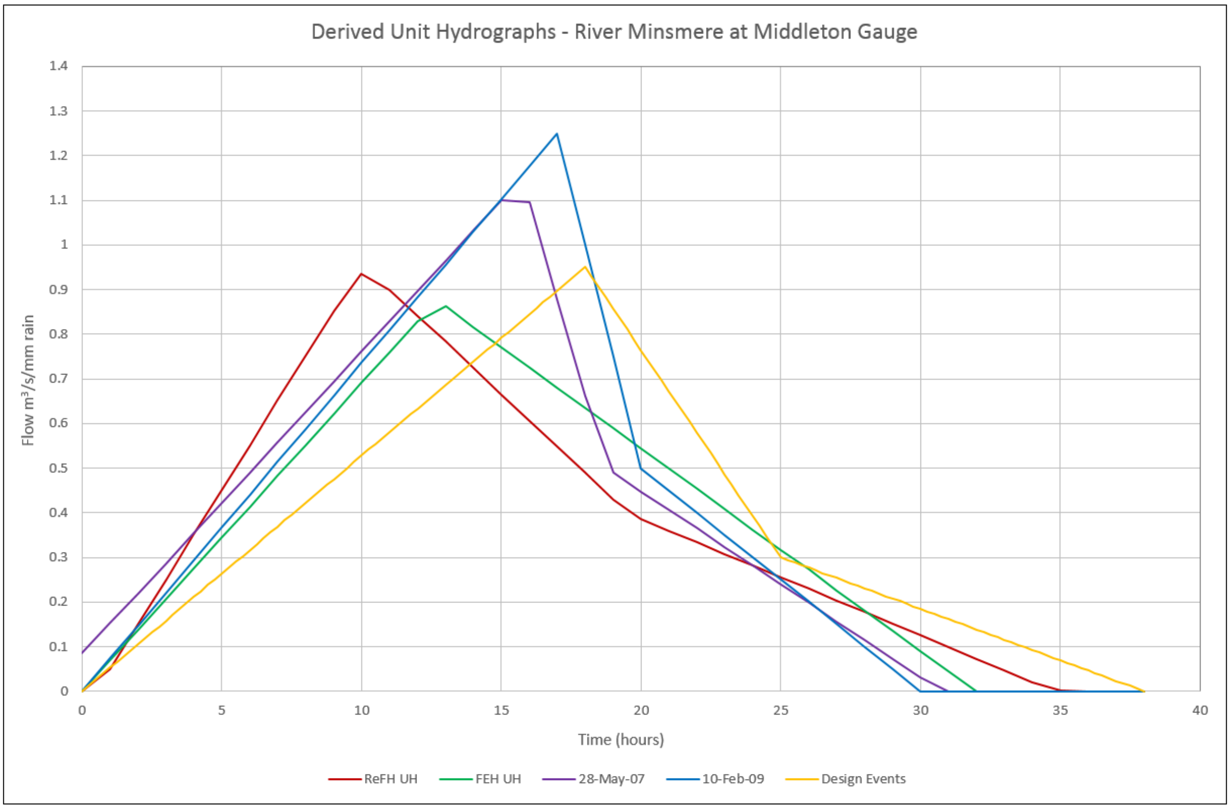

Figure 3.3. Comparison of derived Unit Hydrographs for River Minsmere at Middleton Gauge

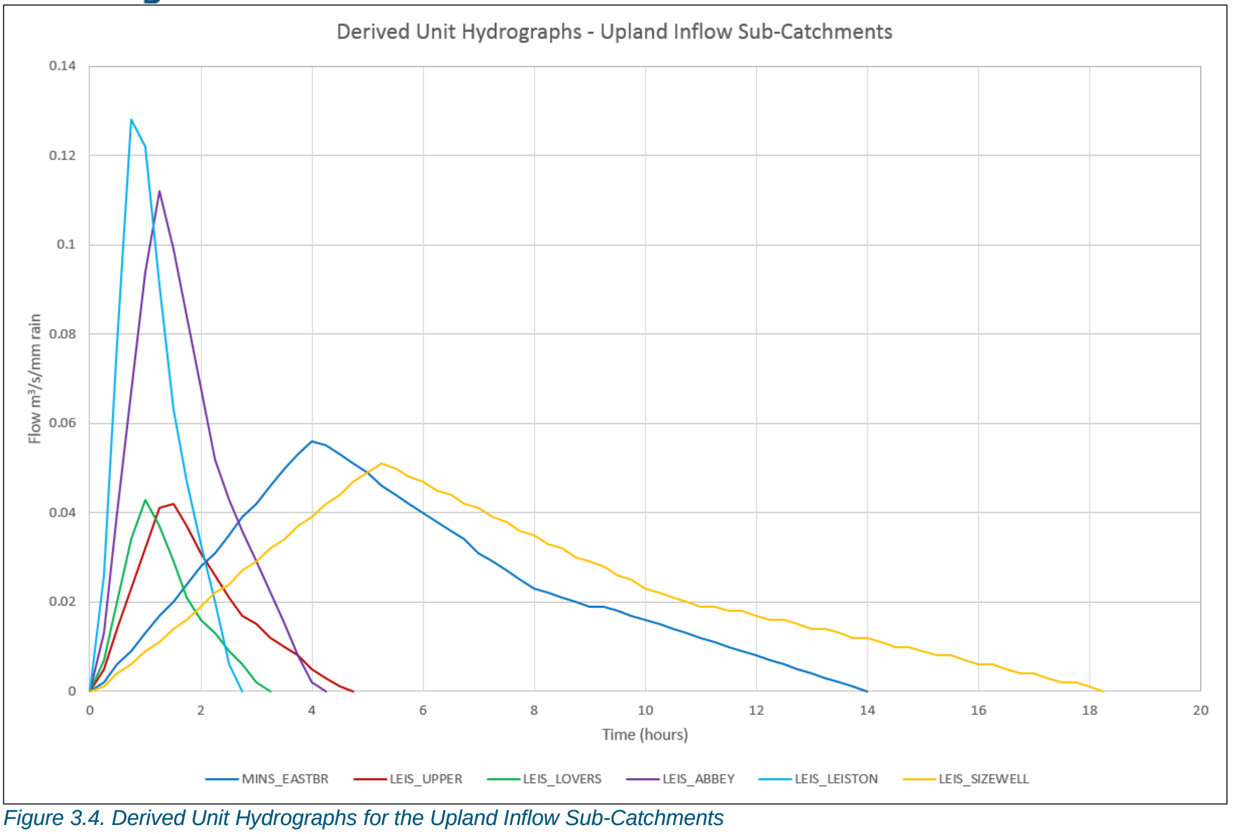

Figure 3.4. Derived Unit Hydrographs for the Upland Inflow Sub-Catchments

3.2 Selection of Methodology for Design Event Peak Estimation

Different methods of flow estimation have been considered for the Sizewell C study in line with the Environment Agency's Flood Estimation Guidelines (Ref 3) which was current at the time of the assessment. Although the site has a gauge at the upstream reach (on the River Minsmere at Middleton), the flow data may not be not reliable due to bypassing of the gauge, as described in Section 2.2. Therefore, techniques available to estimate flows at an ungauged site have been considered, such as statistical analysis, but single site statistical analysis using the Middleton gauging station was considered to have low confidence.

Three different types of techniques have been identified and considered and are further described in the following sub-sections:

- Rainfall-Runoff Methods;
- Statistical Method; and
- Lowland Hydrology for the downstream reaches.

Four reference sites have been selected to compare the flows estimated with the proposed hydrological methods, except the 'Lowland' hydrology where a different approach has been adopted, as described in Section 3.2.3. Nodes representing the reference sites have the largest contributing area and give a good spatial representation of the catchment. The selected nodes are as follows:

- Upstream of Minsmere, inflow node MINs01_6151
- Downstream of Minsmere inflow node MINs01_2628
- Upstream of Leiston inflow node LEIS_4265
- Upstream of Sizewell inflow node SIZE01_1768

To provide a comparison with the analysis of observed data, flows were also calculated for the Middleton gauge site, making a total of five reference sites.

The following sub-sections provide further details on the methodologies considered, their obtained results and observations.

3.2.1 Rainfall-Runoff Analysis

Catchment descriptors were taken from the FEH CD-ROM (Version 3), inserted into Flood Modeller (formerly ISIS) FEH, ReFH and ReFH2 boundary units and used to generate flows for the five reference sites.

Only the critical storm duration (i.e. that which yields the highest peak flows) for each of the catchments was considered at this point. However, it is important to appreciate that flood risk in catchments such as those at Sizewell, are often governed by the volume of the flood hydrograph rather than the peak inflow. This is because the low-lying areas of the catchment serve in effect as a “reservoir”, storing and attenuating the flood. Flood risk in areas affected by tide-locking (i.e. where the fluvial water is unable to discharge as a result of high sea levels) is governed by the volume of the flood event, duration of the time of tide-locking and the capacity of the outfall.

As part of the hydraulic analysis, a matrix of storm duration events was run, in order to establish the storm duration which results in the highest modelled water levels. The storm durations were checked at two locations within the model; the main development platform and the Minsmere Sluice. The assessment included consideration of the impact of climate change scenarios, since the period of tide-locking may be longer in future due to sea level rise and increased fluvial runoff.

The initial flows for the 1 in 100-year return period flood (using catchment descriptors obtained from the FEH CD_ROM 3 without any adjustments) are presented in **Table 3.2**. The purpose of this table is to present comparison of flows derived using the three rainfall-runoff methods.

Table 3.2. Initial 1 in 100-year Flows Derived from the Rainfall Runoff FEH/ReFH/ReFH2 Methods

Reference Site	Critical Storm Duration (hours)	FEH (m^3/s)	ReFH (m^3/s)	ReFH2 (m^3/s)
MINS01_6151	22	24.1	19.3	24.5
MINS01_2628	15	3.9	2.1	2.7
LEIS_4265	9	2.8	1.1	4.0
SIZE01_1768	11	1.1	0.3	0.4
MIDDLETON GS	22	26.6	21.2	27.1

Table 3.2 shows that the FEH gives higher peak flows than the ReFH estimates. This is due to the modified UH in the ReFH method which has a steeper / shorter rising limb to allow the kinked and longer receding limb. It is also evident that ReFH2 gives flows more similar to the FEH method than the ReFH method.

It is important to appreciate that the design events considered within the FRA will cover a number of storm durations, from the very intense short duration storm to more prolonged events lasting several days. This is to test the sensitivity of the system to different flood volumes, in order to determine the most severe in terms of flood risk. The various hydrological methodologies assessed have been developed in order to derive hydrographs for the "critical storm" i.e. the storm duration that yields the highest peak water levels. In the case of the ReFH, this methodology begins to fail when the storm duration is longer than the critical storm, due to over-exaggeration of the baseflow component, leading to unrealistic storm volumes. This is exaggerated increasingly as the storm duration is increased.

Since the critical storm duration of the smaller sub-catchments is short (e.g. 3 to 5 hours), in line with Environment Agency guidance (Ref 3) the ReFH and ReFH2 methods are not generally considered appropriate for the much longer storm durations required for the FRA. **Table 3.3** below shows the performance of the ReFH method for the Sizewell_1768 reference site for events of differing storm duration. Flows are derived for the 1 in 100-year return period, but with PR lowered to 7.3% to be consistent with the ReFH software (hence different to **Table 3.2** above.)

Table 3.3. Peak Flow and Flood Volume Variation with Storm Duration for the Sizewell_1768 reference site

Storm Duration (hours)	Rainfall Depth (mm)	Peak Flow (m^3/s)		Flood Volume (m^3)	
		FEH	ReFH	FEH	ReFH
5	75.8	0.61	0.19	23,000	7,000
11	89.8	0.65	0.25	27,000	12,000
21	101.9	0.59	0.23	31,000	17,000
49	119.8	0.42	0.26	38,000	28,000

It can be seen that the ReFH flood volumes increase four-fold from the 5hr to the 49hr storm, whereas the rainfall increases by just 50%. This indicates that the ReFH method is not appropriate for representing the sub-catchment inflows in this study.

In addition, the ReFH2 method provides similar peak flow values to the FEH rainfall runoff method and due to remaining concerns over the application of the ReFH2 method for longer length critical storm durations instead, it is recommended that the FEH rainfall runoff method is used, with appropriate modifications made to the hydrological parameters, such as PR values and derived event specific unit hydrographs as described in Section 3.1.3.

3.2.2 Statistical Analysis

Statistical Pooling group analysis is widely recommended for an ungauged site, which is considered to be the case for this site due to low confidence in the data collected from Middleton gauge. The pooling group analysis was undertaken at the four locations (reference sites) as discussed previously, with catchment descriptors extracted at the four locations using the FEH CD-ROM Version 3 (see Section 3.1.2). The latest (at the time of the analysis) available NRFA Peak Flow dataset (formerly HiFlows, dataset version 3.3.4) was used to generate the pooling group analysis. It is acknowledged that there have been more recent updates to the NRFA dataset, however this was the latest information available at the time of the assessment. Further assessment could be carried out in the later stages of the Sizewell C project.

As documented in the original Sizewell Study (Ref 1), the Middleton gauge was identified as the only potential donor site from which to derive QMED and higher flows. Due to the very large distances between the catchment centroids, the "Data Transfer" method set out within the Flood Estimation Guidelines (Environment Agency, 2012), which includes the term "a" to account for geographical distance, led to very low adjustment factors, making the adjustment process unreliable, and values were retained the same as the FEH rainfall runoff method based on FEH derived cd's.

The superseded method of 'Data Transfer', based on "hydrological similarity" and analogue catchments was also considered however this resulted in a considerable and seemingly unrealistic increase in the derived flows.

Pooling groups were derived for the reference sites and resulting flows are presented in **Table 3.4**. Further details of the pooling groups for the subject sites can be provided upon request.

Table 3.4. Initial Flows Derived from the Statistical Method

Reference Site	QMED / 2 year (m^3/s)	10 year (m^3/s)	100 year (m^3/s)
MINS01_6151	See Middleton Gauge		
MINS01_2628	0.6	1.2	2.2
LEIS_4265	0.3	0.7	1.3
SIZE01_1768	No suitable donor found		
MIDDLETON GS	6.1	10.6	16.7
MIDDLETON GS (old method of Data Transfer)	10.5	18.6	29.7

Due to the absence of reliable data it is considered that the Statistical Method (single site or enhanced single site) is not preferred for this study and therefore was not considered further in this hydrological assessment. This aligns with conclusions contained within the JBA 2103 Study related to the use of pooling groups and the Statistical Method.

3.2.3 Lowland Hydrology

The downstream reaches of the catchment, around Sizewell C, the Sizewell belts and Minsmere Levels are low-lying and flat and have a large attenuation effect on flows in the lower catchment. The impact of these waterbodies is reflected by the low value of FARL (FEH index of flood attenuation due to reservoirs and lakes which is 0.80 at the downstream of the site) and the URBEXT, with some of the sub-catchments being completely rural (see **Table 3.1** in Section 3.2). Therefore, it was prudent to also consider methods that have been developed particularly for lowland catchments, such as the pumped catchment method as stated in the Environment Agency's Pumped Catchment Guide (Ref 4).

Following this guidance, a trapezoidal Unit Hydrograph was derived and used to generate flows to represent the pumped lowland system. This technique however, is intended for lowland systems considerably larger than the Sizewell study. Furthermore, it is ideally recommended for catchments that have a means of calibration, using either records of pump rate or outflow rate. As there is no data record for outfall rates through the Minsmere Sluice, it is not possible to calibrate and adjust the flood volumes produced using the Pumped Catchment Method. Little confidence could therefore be placed in the flows generated by this method.

In order to improve the representation of inflows to the lowland system, a Direct Rainfall method was applied to the hydraulic model for areas located below the 3m AOD contour line. This approach is preferred over using the "Lowland Unit Hydrographs", as it avoids the potential for double counting of storage and attenuation within the hydraulic model and the hydrological approach. Instead, it allows the 2D model domain to route the water through the system, better representing flood mechanisms and accounting for storage within the system.

The Lowland Catchment during any significant out of bank events (i.e. 1 in 5-year or greater), would mean that most of the marshes are fully inundated and the surface depressions are likely to be a minimal issue. However, the event being modelled is a long duration winter storm with a relatively high initial level of saturation, although some of the rainfall would infiltrate to ground even when the marshes are underwater. Therefore, the sensitivity to infiltration is likely to be lower than would be the case for short duration summer floods.

3.2.4 Adopted Approach

Conclusions derived from the analysis using different methods indicate that the ReFH and ReFH2 approaches are not appropriate for this study as flooding within the system is dominated by storm durations considerably longer than the critical storm duration, in which case the ReFH methods are known to significantly overestimate the flood volume.

Sensitivity testing has been carried out to understand the relative difference between FEH, ReFH and ReFH2. These methods, in particular ReFH, are not recommended for permeable catchments. It is acknowledged within the Technical Guidance Document: ReFH2.2 (Ref 15) that permeable catchments (with BFIHOST greater than 0.65) have more complex hydrology.

ReFH2 is known to have substantially improved the hydraulic modelling performance for permeable catchments. When comparing the Factorial Standard Error applied in the permeable ReFH2 method it is still slightly higher than the impermeable ReFH and ReFH2 for the FEH99 and the FEH13 datasets. While there is a significant improvement, this represents a continuing uncertainty of the predictions in permeable catchments. The majority of the sub-catchments in this study have BFIHOST values well above 0.65, see **Table 3.1** in Section 3.1.2. A review of sensitivity testing results showed that the FEH rainfall run-off results produced were similar to the ReFH2. Further sensitivity testing and comparison with ReFH2 could be carried out at the later stage of the Sizewell C project.

With regard to the Statistical Method, it was not possible to find appropriate donor gauges with reliable data to undertake a reliable statistical analysis or obtain a representative pooling group.

Within the Lowland Catchment, the Pumped Catchment method was considered but based on a review of the Environment Agency guidance it was considered inappropriate with no ability to calibrate the outputs.

At this stage, the focus of the hydrology is to support the development of an appropriate hydraulic model to demonstrate the impact of fluvial flooding and the relative difference the proposed development would make throughout its lifetime. Therefore, the FEH parameters derived in this analysis would be used to derive flood flows for a series of events. However, no adjustment will be applied to PR or PROPWET (proportion of time the catchment soils are wet).

The relative impact of the application of ReFH2 was assessed with sensitivity testing. However, there is insufficient data for calibration or verification within the catchment to fully determine whether a specific hydrological method is more accurate than another. There is limited hydrometric data available to supersede the use of catchment descriptors, although adjustments have been applied to the hydrology, where possible. A calibration has been conducted and the available level data confirms the underlying hydrological inputs are plausible. This is considered sufficient to assess the relative impact of the project on fluvial flooding.

A hybrid approach is recommended as the most appropriate approach for deriving main inflows and sub-catchment inflows. These are as follows:

- Main inflow (Minsmere upstream) – FEH boundary unit with UH shape derived from data from the Middleton gauge using ‘Strip-UH’ and a PR value based on observed event analysis;
- Remaining “Upland” inflows – FEH boundary units with parameters and ‘Strip-UH’ shape adjusted based on analysis of the flows at the G5 gauge; and
- Lowland inflows – Direct rainfall method on all areas where the ground level is located below the 3m AOD contour line.

Some of the sub-catchment inflows were applied in the 1D model as FEH boundary units, others have been added as inflow points to the 2D TuFLOW model, with hydrographs derived from the FEH boundary units. **Figure 3.5** illustrates the locations of all hydrological inflows into the hydraulic model adopted for the Sizewell C FRA study.

Further details of the Flood estimation calculation records are provided in Appendix C.

A series of sensitivity tests have been carried out using the hydraulic model to determine the critical storm duration, for a 1 in 100-year return period event (as considered critical for the FRA study). The rainfall data from two gauges were analysed and used within the model calibration process to understand the rainfall profile. A review of a variety of storm durations were carried out to assess the critical storm duration for the catchment. The critical storm duration for the lowland catchment is 121 hours, which will be used for all other events and return periods. Whilst it is acknowledged that this is a long storm duration, it has been appropriately derived and further information is available in the fluvial modelling report.

Initially for the assessment of fluvial flood risk, a representation of Mean High Water Spring (MHWS) was applied at the downstream boundary of the model. However, this approach was revised in accordance with Environment Agency comments that requested the application of the joint probability approach from FD2308 – Use of Joint Probability Methods in Flood Management (Ref 16).

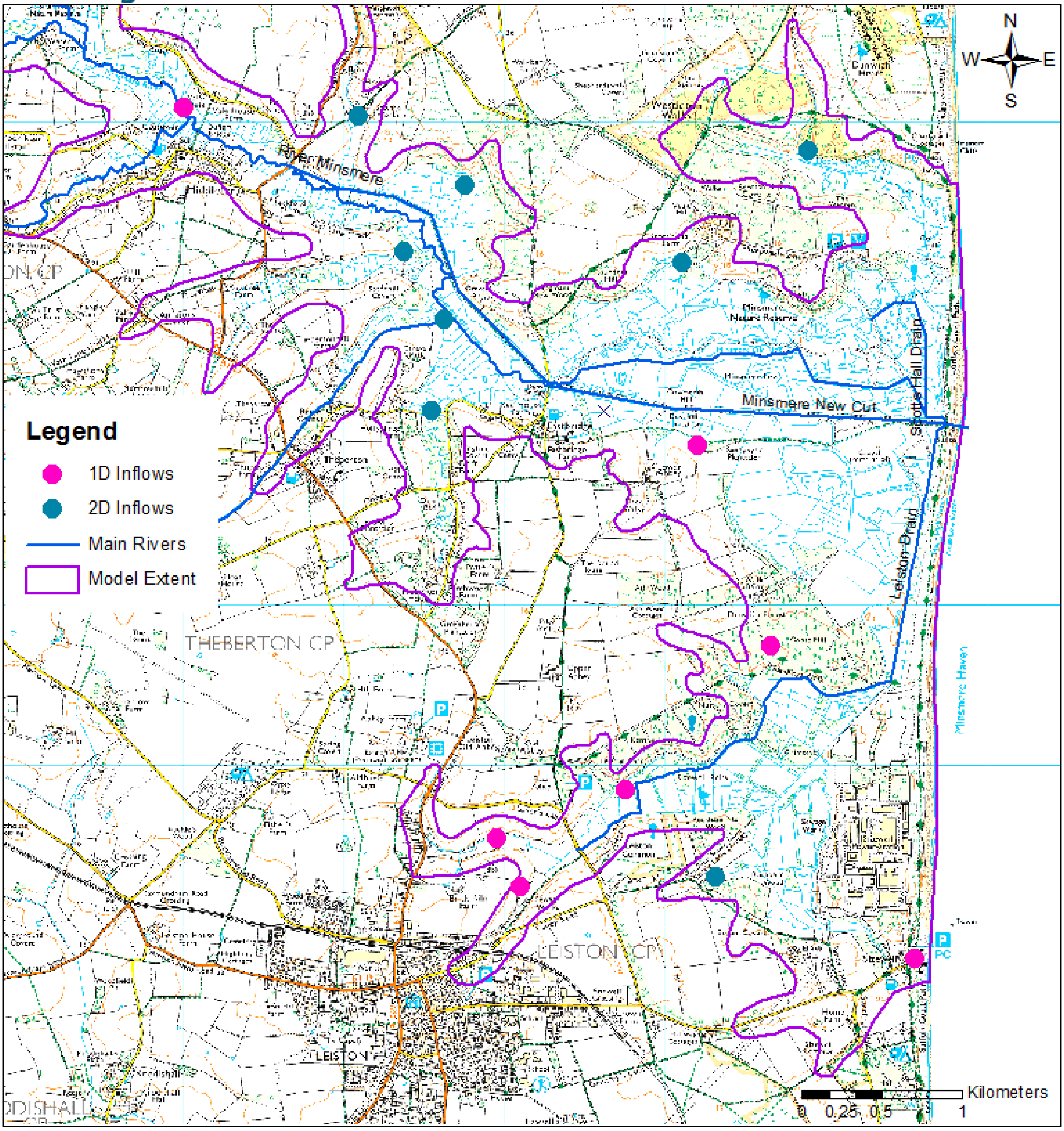

Figure 3.5. Locations of Hydrological Inflows adopted in the hydraulic model.

3.3 Tidal Boundary

To derive the time series of water levels for the tidal model boundary the following steps were undertaken:

1) Derivation of time series of astronomical tide levels based on harmonic constituents for Lowestoft, adopted from Admiralty Tide Tables for 2017 (Ref 17) and then transforming them to Minsmere using the same transformation method as described in Section 2.4;
2) Selection of a donor surge shape profile. The surge donor profile for Lowestoft was obtained from the Environment Agency Coastal Flood Boundary Conditions Database (CFBD), (Ref 18);

3) Obtaining the extreme water levels for required return period events based on recently updated UK Coastal Flood Boundary Dataset (2018) for point at chainage 4192 with base year of 2017 (Ref 19);
4) Derivation of the design tide curve by scaling the surge shape so that when combined with the tide data, the peak tide equalled the required extreme water level for each return period. Peak of the surge was timed so that it coincides with the highest predicted astronomical tide in 2017 (1.31m AOD at Lowestoft, 29/03/2017).

As set out in point 1 above, tidal levels at Minsmere were derived based on harmonic data for Lowestoft and applying a suitable relationship for transformation of tide levels from Lowestoft to Minsmere Sluice discussed in Section 2.4. Since the transformation was derived based on high and low tide data for Lowestoft and Sizewell B stations from February 2009 to December 2012 (supplied by CEFAS on behalf of EDF), a sensitivity check was carried out to confirm that the transformation is still valid considering more recent years or recorded tide levels.

For that purpose, further records from the Sizewell B gauge station for the period between July 2016 and December 2018 were supplied by EDF and the Lowestoft records covering the same period were obtained from open source dataset available from British Oceanographic Data Centre (Ref 20). **Figure 3.6** presents the relationship between the Lowestoft and Sizewell B tide levels from all collected data. The comparison shows very good fit of the recent records to the older records and therefore it is concluded that the derived transformation is valid.

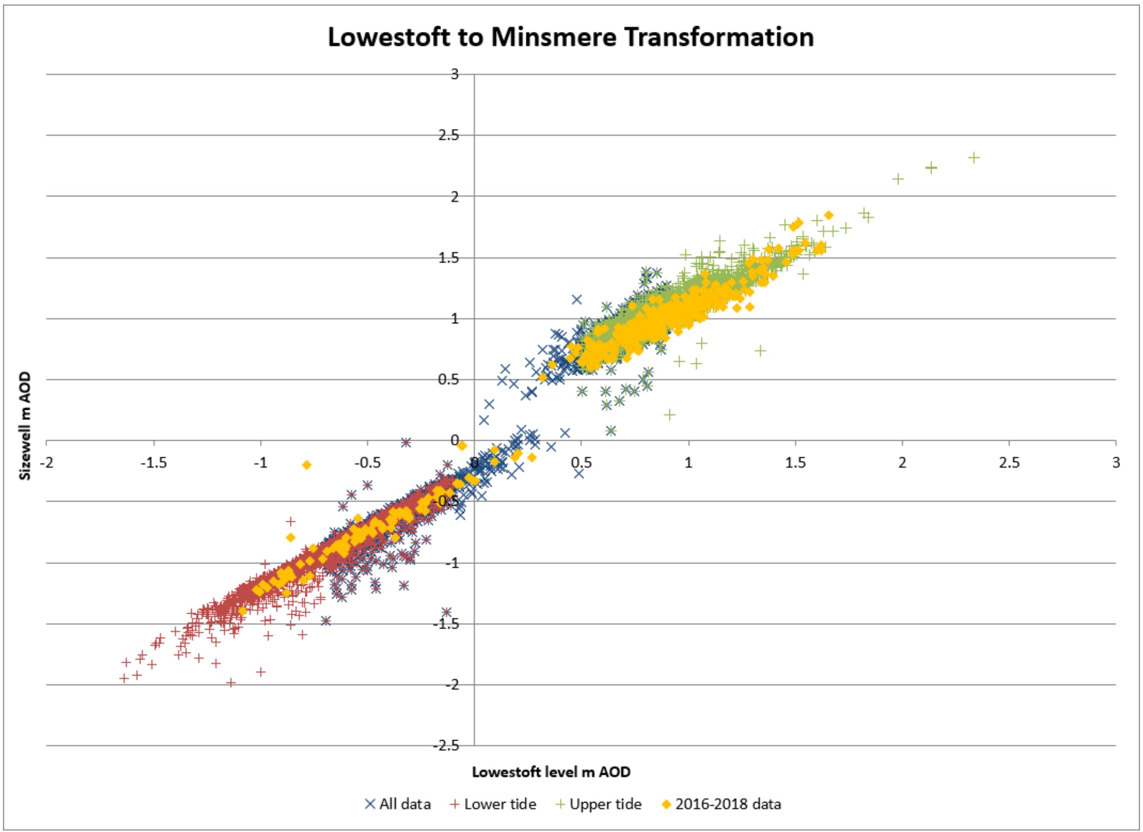

Figure 3.6. Lowestoft and Sizewell B Tide Relationship Check

The extreme tide levels for considered return period events were obtained from Environment Agency Coastal Flood Boundary Dataset (CFBD) for UK updated in 2018 for chainage 4192 (**Figure 3.7**), including confidence interval (Ref 19) and are presented in **Table 3.5** .

Table 3.5. Extreme Water Levels from UK CFBD (2018) for chainage 4192

Return Period	Extreme Water Level (m AOD)
1 year	2.08
2 year	2.25
5 year	2.51
10 year	2.72
20 year	2.94
50 year	3.23
75 year	3.38
100 year	3.47
200 year	3.72
1,000 year	4.37
10,000 year	5.51

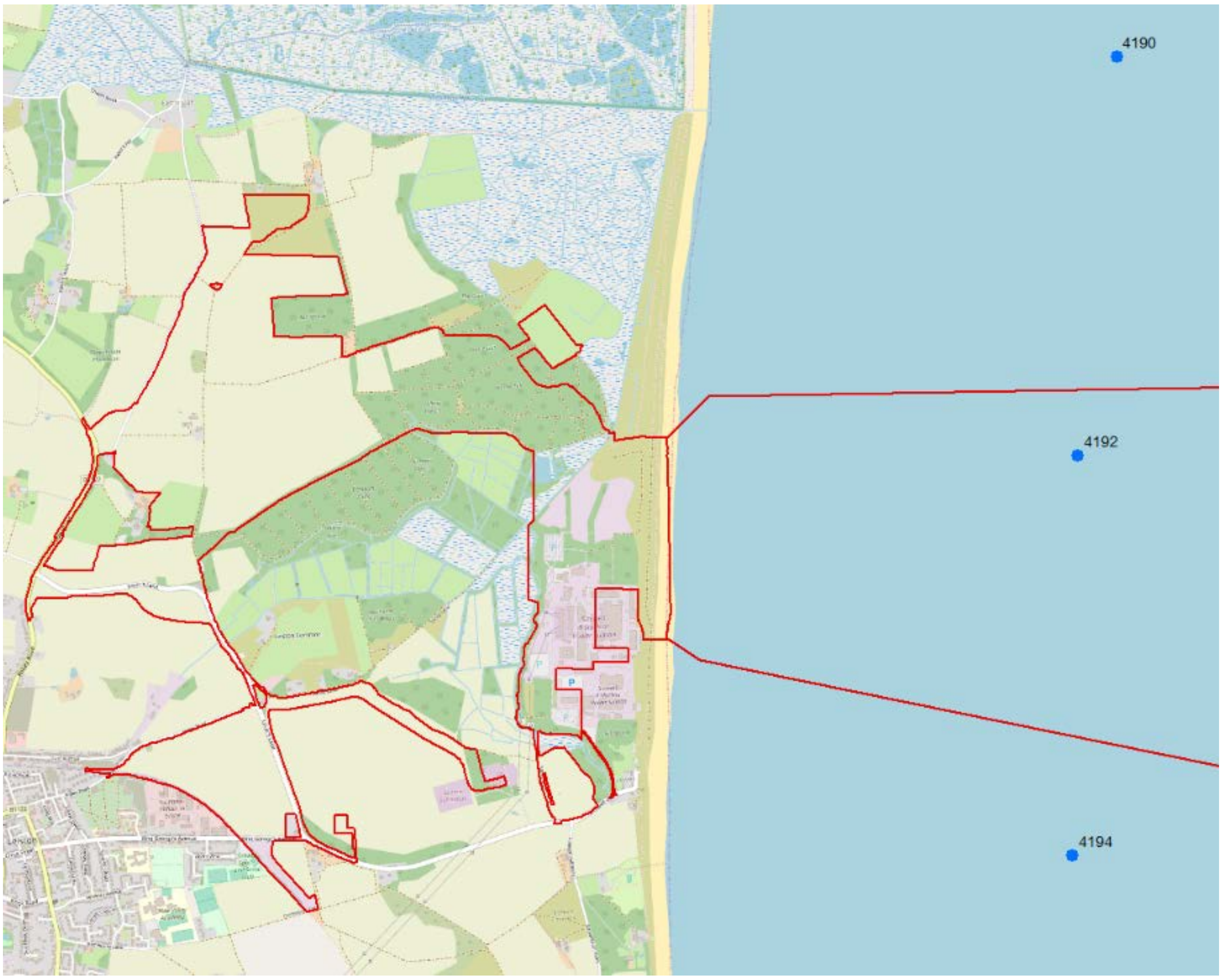

Figure 3.7. Location of the UK Coastal Flood Boundary Dataset Point in front of Sizewell C development

To derive the design tidal boundary, joint probability of fluvial flows and tide levels was carried out, discussed in the following Section 3.4, and the resulting tide levels were then used to scale the surge event.

3.4 Joint Probability

A joint probability of fluvial flows and tide levels was conducted with the focus of extreme fluvial flows in line with the Environment Agency guidance on 'Use of Joint Probability Methods in Flood Management' (Ref 16).

For the joint probability assessment, the derived extreme water levels for different return period events were used, together with corresponding flows at Minsmere outfall. The flows at Minsmere outfall were taken as combined peak flows from all sub-catchments based on the adopted approach for peak flow estimation discussed in Section 3.2.4. The dependence information for river flow and surge was adopted from the Environment Agency guidance (**Figure 3.8**) which is recommended for the most immediate use for derivation of combinations of sea level and river flow for use in modelling of individual rivers. For River Minsmere catchment the suggested dependence coefficient chi is between 0.03 and 0.06, which gives rho value of 0.5 (as per Table 3.9 of the Environment Agency R&D Technical Report FD2308/TR2) that has been adopted for this study.

Derived joint exceedance return period events are presented in **Table 3.6**.

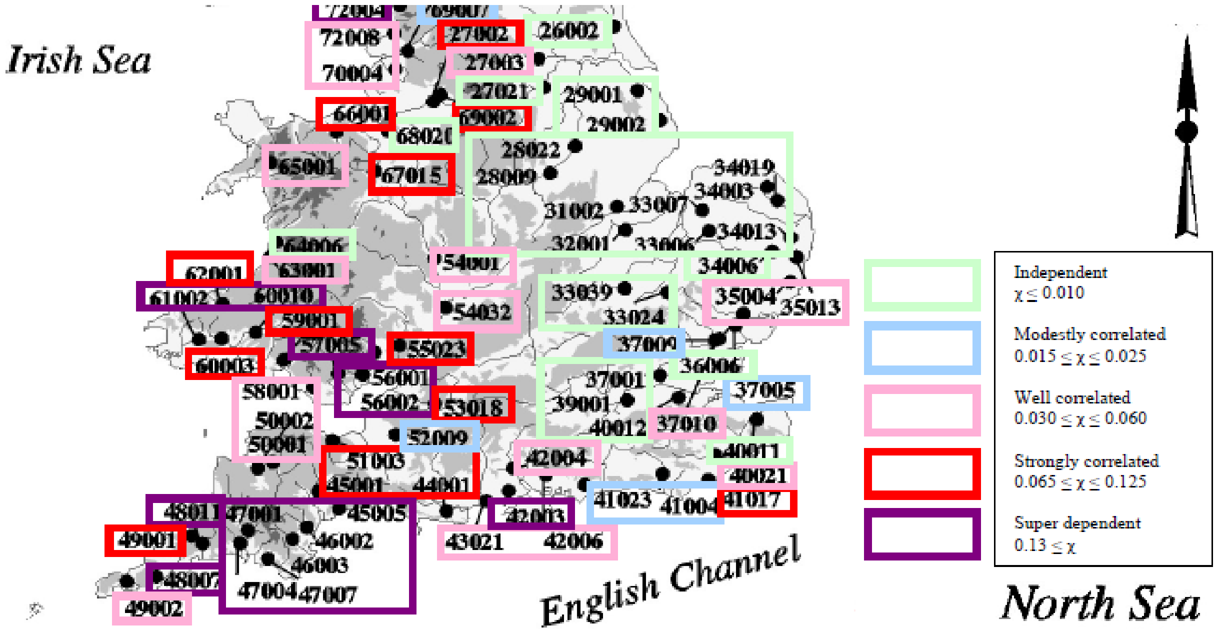

Figure 3.8. Extract from Environment Agency R&D Technical Report FD2308/TR2: Figure 2 Summary dependence information for river flow and surge

As the main focus of the fluvial modelling was to assess fluvial flood risk on-site and off-site, the joint probability combinations were chosen so that the extreme fluvial flow for relevant return period event was selected with lower return period giving the joint exceedance return period of the fluvial flow event. For example, for a 1 in 100-year joint exceedance event, a 1 in 100-year fluvial flow was used with the corresponding tide level, in this case 0.2-year return period tide. The assessment showed the marginal return periods for tide events between 2 years and 50 years are the same and therefore only three extreme tide levels were used for the tidal boundary corresponding to a range of fluvial flow return period events, as presented in **Table** 3.7.

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Table 3.6. Derived joint exceedance return period events for fluvial flows and extreme water levels

Marginal return period (years) for Fluvial Flow (m³/s)	Joint exceedance return period (years): 2	5	10	20	50	100	200	1,000
	Marginal return period (years) for Water Level (m AOD)							
0.01	2.0	5.0	10.0	20.0	50.0	100.0	200.0	1,000.0
0.02	2.0	5.0	10.0	20.0	50.0	100.0	200.0	1,000.0
0.05	1.8	5.0	10.0	20.0	50.0	100.0	200.0	1,000.0
0.1	0.9	3.1	8.1	20.0	50.0	100.0	200.0	1,000.0
0.2	0.5	1.6	4.0	10.3	35.9	91.9	200.0	1,000.0
0.5	0.2	0.6	1.6	4.1	14.3	36.8	94.2	838.3
1	0.1	0.3	0.8	2.1	7.2	18.4	47.1	419.1
2	0.05	0.2	0.4	1.0	3.6	9.2	23.6	209.6
5	#N/A	0.1	0.2	0.4	1.4	3.7	9.4	83.8
10	#N/A	#N/A	0.1	0.2	0.7	1.8	4.7	41.9
20	#N/A	#N/A	#N/A	0.1	0.4	0.9	2.4	21.0
50	#N/A	#N/A	#N/A	#N/A	0.1	0.4	0.9	8.4
100	#N/A	#N/A	#N/A	#N/A	#N/A	0.2	0.5	4.2
200	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	0.2	2.1
1,000	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	0.4

Table 3.7. Derived joint exceedance return period tide levels

Joint exceedance return period event	Extreme Tide Level Return Period	Extrapolated Tide Level (m AOD) from UKCFB data (base year 2017)
1 in 2, 5,10, 20, 50-year	0.1-year	1.52
1 in 100-year	0.2-year	1.69
1 in 1,000-year	0.5-year	1.91

The derived extreme tide events were then used to scale surge event and produce timeseries of tide levels to be applied at the downstream model boundary, as illustrated in **Figure 3.9**.

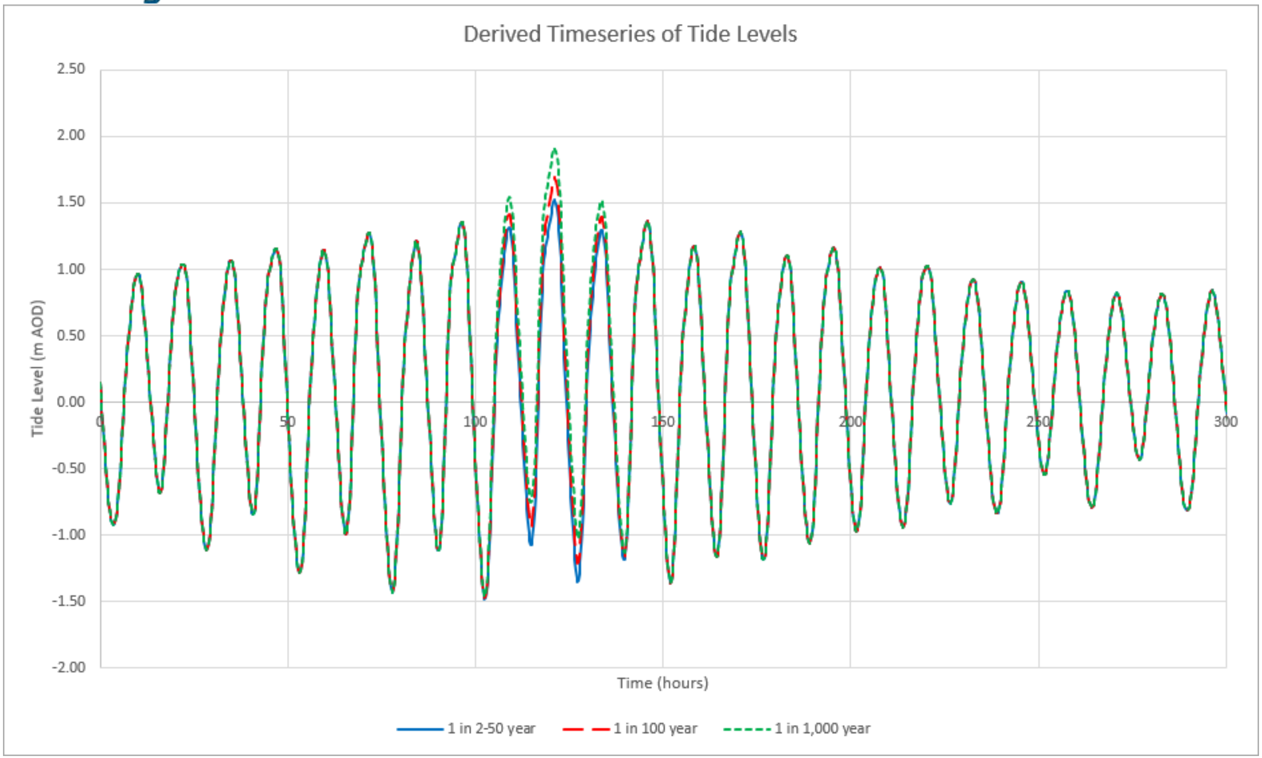

Figure 3.9. Derived tidal model boundary

3.5 Climate Change

Climate change allowances have been considered for the FRA study comprising an increase in peak river flows, increase in rainfall intensity and sea level rise for the tidal boundary. The following sub-sections summarise the adopted allowances for key phases of the Sizewell C development.

Full details on derivation of the climate change allowances are available in a technical note 'UK Climate Change Projections 2018 - Review and Proposed Response' (Ref 21).

3.5.1 Peak River Flow and Rainfall Intensity

The climate change allowances for peak river flows and rainfall intensity were derived in line with the National Planning Policy Framework (Ref 22) and the Environment Agency guidance (Ref 23), including Advice for Flood and Coastal Erosion Risk Management Authorities (Ref 24).

The UK Climate Projections published in 2018 provided updated information on climate change allowances for rapid response rainfall in small catchments (<5km^2) but the Environment Agency is in process of updating fluvial allowances for larger catchments.

Following the above 2016 fluvial guidance, it was recommended that the peak river flow allowances for the Anglian River Basin District are applied, which are summarised as the Upper End percentiles and H++ scenarios respectively:

- 2020's: +25% and +25%;
- 2050's: +35% and +40%;
- 2080's: +65% and +80%.

The Environment Agency guidance for Risk Management Authorities also suggests that for areas >5km^2, the fluvial climate change allowance should be used as allowance for increase in rainfall intensity. In the hydraulic model developed for the FRA study, the 'Lowland' area where the direct rainfall is applied is larger than 5km^2, therefore the same allowances for rainfall as per the fluvial allowances were used.

Further discussions with the Environment Agency led to their request for the Higher Central and Upper End allowances. During technical consultation, the Environment Agency have confirmed their requirement for the 35% and 65% climate change allowances to be applied. **Table 3.8** presents the recommended climate change allowances for the Sizewell C FRA study.

Table 3.8. Recommended Climate Change Allowances to use in Sizewell C FRA for assessment of pluvial and fluvial flood risk

Development Phase	Year	Climate Change Scenario	Climate Change Allowance
End of Construction / Commissioning	2030	Upper End	+25%
End of Operation	2090	Higher Central	+35%
Interim Spent Fuel Store Decommissioned	2140	Upper End	+65%
Theoretical Maximum Site Lifetime	2190	H++ Scenario	+80%

For epochs beyond 2115 (2080s) no extrapolation was applied. The 35%, 65% and 80% allowances were used in accordance with the 'Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities' guidance (Ref 24) stating: 'For changes beyond the 2080s, it is recommended that the 2080s changes are used'.

The Higher Central and Upper End allowances were used for assessment of the reasonably foreseeable climate change scenario and the H++ for the credible maximum scenario.

3.5.2 Sea Level Rise

For application to the tidal boundary in the fluvial hydraulic model, it was considered appropriate to apply sea level rise allowance adopting the UKCP18 RCP8.5 allowances at 95%ile in line with the ONR and Environment Agency advice on 'Use of UK Climate Projections 2018 (UKCP18) by GB Nuclear Industry' (Ref *25*).

Following review of the initial of the 'UKCP18 Review and Proposed Response' technical note (issued 13 March 2019) the Environment Agency provided comments and advice on 'How to extrapolate the UKCP18 dataset for sea level rise allowances beyond 2100' (Ref *26*). In accordance with this advice, the UKCP18 21st century projections were extrapolated up to 2125. For allowances beyond 2125, the exploratory projections were used.

Derived cumulative sea level rise allowances (relative to 2017 base year) were applied to the tide curve for the considered climate change epochs/ key points in time for the Sizewell C development as follows:

- 2030: +0.094m;
- 2090: +0.867m;
- 2140: +1.761m; and
- 2190: +2.591m.

3.6 Safety Case

The nuclear safety case would involve studying the impact of fluvial, pluvial and coastal events for the 1 in 10,000-year as basis of design and 1 in 100,000-year return period as sensitivity testing. The focus of the safety is to assess flood risk to the development itself and demonstrate resilience of the design to very extreme low probability events. For that purpose, three separate analyses would be carried out for each flood risk source.

For the pluvial flood risk, extreme rainfall events for the 1 in 10,000-year return period were analysed. Derivation of extreme rainfall events and associated climate change allowances is discussed in detail in the Extreme Rainfall Assessment report (Ref 27).

Fluvial flood risk would be assessed for both 1 in 10,000-year and 1 in 100,000-year events, considering joint probability of fluvial flows and surge levels. The boundary conditions would be derived adopting the same approach as described in Section 3.2.4 for fluvial flows and Sections 3.3 and 3.4 for the joint probability with tide levels. Further details and final derived fluvial flows and sea levels are discussed in the Sizewell C Safety Case Modelling report (Ref 28).

For the coastal flood risk assessment, more conservative extreme sea levels would be considered as discussed in the UKCP18 Review report (Ref 21). The climate change allowances for sea level rise would be considered based on the UKCP18 projections for the reasonably foreseeable scenario and the more conservative, BECC Upper projections for the credible maximum scenario. Further details and final derived sea levels are discussed in the Sizewell C Safety Case Modelling report (Ref 28).

4 Final Design Model Boundaries

Design flows have been derived for the main inflow into the system which is represented in the hydraulic model (River Minsmere at Middleton gauge) and all considered sub-catchments. **Table 4.1** presents derived peak flows for all the inflow boundaries for a series of return period events. These are the peak flows for the 121-hour storm duration event, as testing of the hydraulic model for the 1 in 100-year event indicates this is the critical storm duration for the system.

Figure 4.1 and **Figure 4.2** show derived design hydrographs for a 1 in 100-year return period event with 121-hour storm duration for the main inflow into River Minsmere (MINS_US) and the Leiston Drain (LEIS_LEISTON) respectively. **Figure 4.3** shows design hyetographs for the 'Lowland' area for all considered return period events with 121-hour storm duration.

Table 4.1. Derived Peak Flows for the 'Upland' Inflows for all Sub-Catchments for the 121-hour storm duration

Sub-Catchment	Peak Flow (m^3/s) for Return Period Event						
	1 in 2y	1 in 5y	1 in 10y	1 in 20y	1 in 50y	1 in 100y	1 in 1000y
MINS_US	7.717	10.257	12.046	14.003	16.655	18.643	27.952
MINS_EASTBR	0.187	0.251	0.296	0.346	0.414	0.465	0.705
MINS_DOCWRA	0.456	0.613	0.724	0.847	1.014	1.140	1.736
MINS_POTTERS	0.525	0.704	0.830	0.969	1.158	1.301	1.973
MINS_SCOTT	0.150	0.202	0.239	0.280	0.335	0.377	0.575
MINS_THEBERTON	0.263	0.352	0.415	0.485	0.579	0.650	0.984
MINS_WARKBARN	0.176	0.236	0.279	0.326	0.389	0.437	0.664
MINS_WASH	0.556	0.745	0.878	1.025	1.225	1.375	2.084
MINS_WESTLETON	0.615	0.821	0.968	1.128	1.346	1.510	2.282
LEIS_ABBEY	0.118	0.155	0.182	0.211	0.250	0.280	0.419
LEIS_LEISTON	0.092	0.108	0.125	0.145	0.175	0.202	0.327
LEIS_LOVERS	0.035	0.046	0.054	0.062	0.074	0.083	0.125
LEIS_UPPER	0.051	0.067	0.079	0.092	0.109	0.122	0.183
LEIS_SIZEWELL	0.849	1.148	1.362	1.597	1.917	2.160	3.309

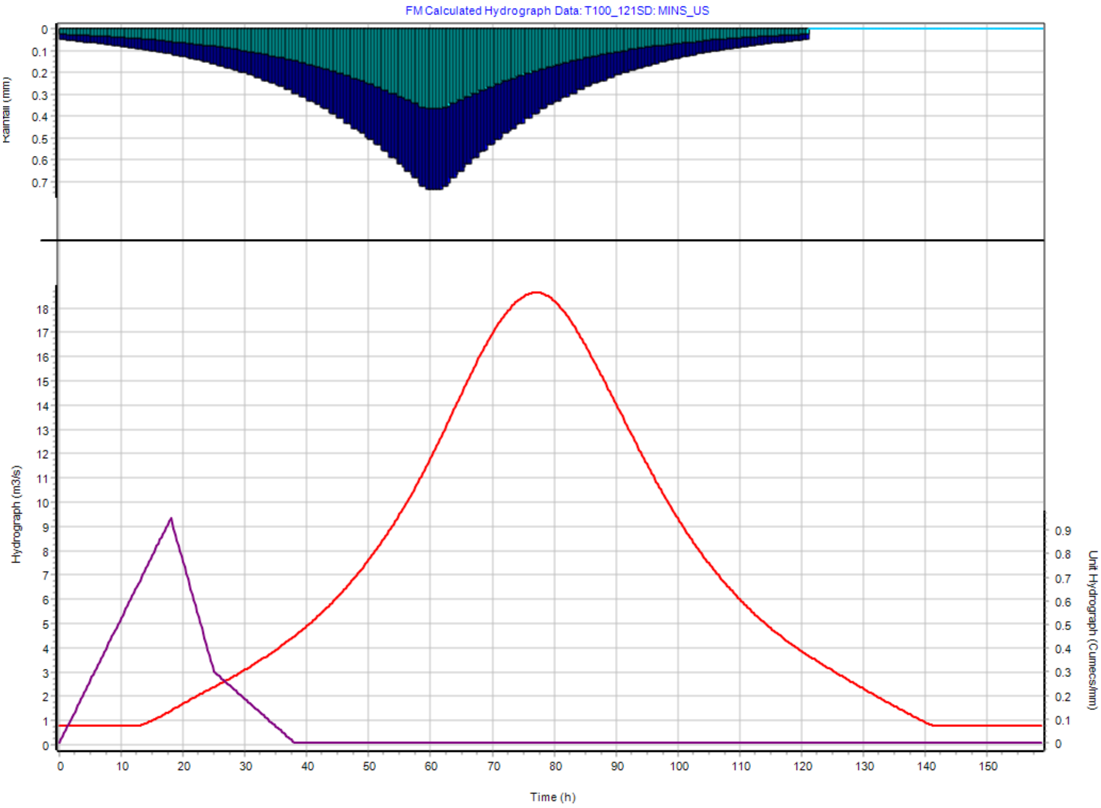

Figure 4.1. Derived Hydrograph for 1 in 100y return period event with 121-hour storm duration – MINS_US

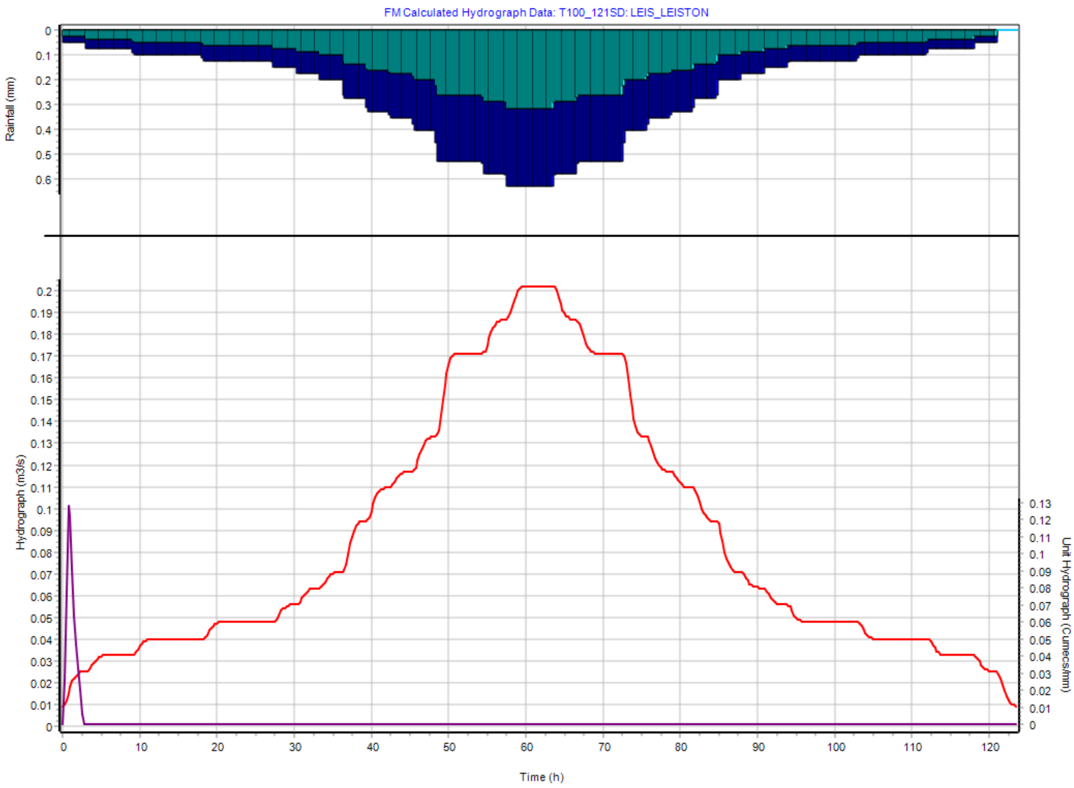

Figure 4.2. Derived Hydrograph for 1 in 100y return period event with 121-hour storm duration – LEIS_LEISTON

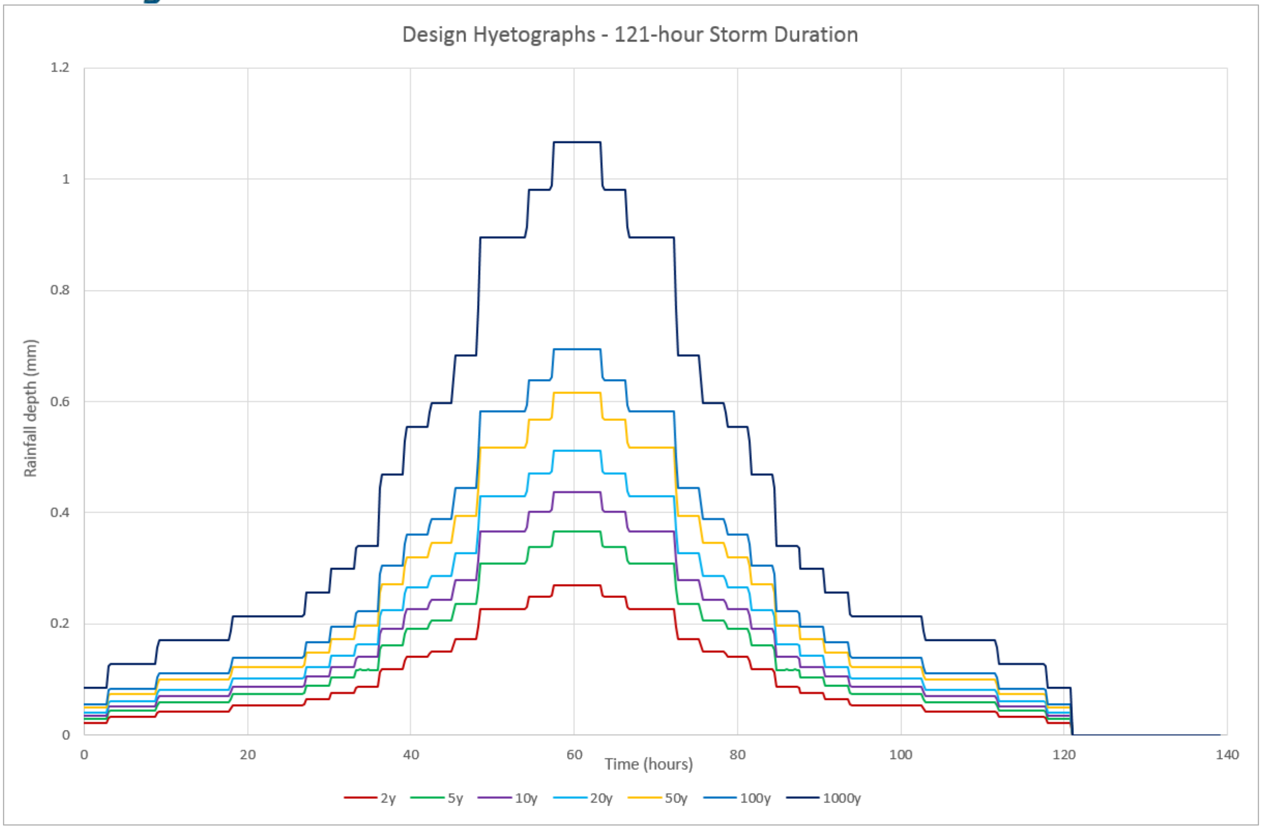

Figure 4.3. Derived Hyetograph for the 1 in 100y return period event with 121-hour storm duration

Appendix D provides the derived design hydrographs for the main 'Upland' inflow sub-catchment. i.e. MINS_US, for the 1 in 100-year return period event and various considered storm durations. Derived design hydrographs for the main 'Upland' inflow sub-catchment, i.e. MINS_US, for all considered return period events with 121-hour storm duration are provided in Appendix E.

As discussed in Section 2.2, there are very limited high flow records available for the Middleton gauging station and there is a level of uncertainty relating to the rating curve. There are also limited observable flows for the Leiston gauge (G5) and observable rainfall data is only available for gauges some distance from the site. Taking into account all of the above, it is acknowledged that there is an overall degree of uncertainty related to the derived design flows due to the limited data / information available.

Hydrological boundaries derived for all 'Upland' inflow sub-catchments for all return period events considered in the Sizewell C study are available as FEH boundary units in the hydraulic model, supplied as a part of the modelling pack.

The tidal model boundary was adopted as discussed in Section 3.3 and Section 3.4.

5 Conclusions and Recommendations

A hydrological assessment has been carried out to derive boundary conditions for incorporation into the hydraulic modelling which is required to assess flood risk in the area and to inform the Flood Risk Assessment for the proposed Sizewell C development.

The main inflows into the system are the River Minsmere and Leiston Drain catchments. Data for the stage gauge station on River Minsmere at Middleton was collected. A new rating curve was developed using the hydraulic model, although confidence remains relatively low due to bypassing at higher flows and uncertainty regarding backwater effects in the absence of a downstream water level recorder.

For a short period between 2005 and 2006 there was also data available for Minsmere Sluice comprising water levels near the sluice. However, it was found not suitable to use in calibration as the limited recorded peak levels are expected to be below bank full. In addition, the data has been recorded prior to renovation of the sluice, and therefore it was concluded that this data is not suitable for use in this assessment.

To derive the hydrological boundaries, the approach recommended for ungauged catchments was adopted, where both statistical and rainfall-runoff methods were considered. The statistical pooling group method was carried out with Middleton gauge as the donor site. However, due to the absence of reliable data at the gauge it was considered that the statistical analysis is not preferred for this study.

Three rainfall-runoff methods were sensitivity tested; FEH, ReFH and ReFH2. The catchment descriptors were taken from the FEH CD-ROM 3. The AREA parameter was adjusted based on Lidar data with finer resolution than that used in the FEH software. This analysis suggested the ReFH method was not appropriate for this study various reasons and the ReFH2 method gave very similar results to the FEH.

The flooding in the catchment is dominated by storm durations that are considerably longer than the critical storm duration. This means the ReFH methods grossly overestimate the flood volume in such instances.

Furthermore, these methods, in particular the ReFH method, are not recommended for permeable catchments with BFIHOST greater than 0.65. The majority of the sub-catchments in this study have BFIHOST well above 0.65. Therefore, the FEH rainfall runoff method was recommended and used to derive the hydrological boundaries for the hydraulic modelling, for all upland catchments. Percentage runoff value was determined based on observed rainfall data and set as 50% for all sub-catchments. Also, event specific unit hydrographs for each boundary unit was derived using in-house tool ‘Strip-UH’ that performs the FEH standard process of extracting a Unit Hydrograph from observed flow and rainfall data. Direct Rainfall is applied to the model for the lowland catchment below the 3m AOD contour.

Due to very limited observable flow/stage records for the Middleton and Leiston gauging stations and rainfall data being only available for gauges some distance from the site, there is an overall degree of uncertainty with the derived design flows.

It is acknowledged that some of the analysis was undertaken in 2015 and the recorded 2016 event was primarily used for model calibration. However, despite this and other uncertainties, the derived hydrological inflows to the fluvial model are considered appropriate for testing the order of magnitude of potential impacts on the fluvial system from the proposed Sizewell C platform. If the fluvial impacts were found to be significant, then further work on the hydrology may be warranted. However, preliminary results from the hydraulic model indicated impacts of less than 15mm. Even substantial changes to the hydrology are unlikely to dramatically change this finding.

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Appendix A

Velocity Index and Rating Report (December 2014)

Atkins
Sizewell Monitoring

J3328

Velocity Index and Ratings

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QUALITY MANAGEMENT REPORT

STAFF INVOLVEMENT

Task	Personnel	Position
Management	[redacted]	Project Manager
Rating Analysis		Hydrometric Engineer
Site visits		Hydrologist
Reporting		Hydrologist

DOCUMENT REVISION

Revision	Issue date	Scope of revision
0	8th December 2014	Final Report

DOCUMENT CONTROL

Revision	Number	Issued to
0	1	Atkins
	2	Hydro-Logic, Bromyard
	3	Hydro-Logic, Reading

Limitation of liability and use

1. INTRODUCTION

1.1 Velocity index rating

A velocity-Index relationship is used to calibrate the velocity measured by an instrument so that it represents the mean velocity at the measuring cross-section. The result is calibrated mean velocity, which can be multiplied by the cross-sectional area to give calibrated flow for the site. The velocity data recorded by the instrumentation cannot be taken at face value as it only represents the velocity at the location of the instrument, in this case the middle of the channel.

The simplest form of the Velocity-Index relationship is one that is developed between the velocity measured by the instrumentation (Index Velocity), in this case a Nivus PCM4, and the mean velocity at the measuring cross-section. The mean velocity comes from the calibration gaugings undertaken at the site, where the gauged flow is divided by the area which is derived from the stage/area relationship.

A more complex version, is to include stage as a variable, this is to aid with any site where velocity and flow does not always necessarily increase with level. All of the sites have used stage as a variable due to the impact of weeds and backing up both affecting the velocity and water levels at the sites.

During each calibration gauging, the Nivus PCM4 was set to record every minute, thus giving a very detailed picture of the variation in index velocity during the calibration gauging. When the gauging is complete, the average index velocity is calculated and compared to the mean velocity obtained from the calibration gauging. Mean velocity is calculated in accordance with the relevant British and International Standard, in the case of current meter gauging. When several calibration gaugings, over a range of flows, have been completed the results can be plotted and a trend line fitted to them.

1.2 Stage Discharge

At sites G3 and G4, a thin plate weir was installed on site, therefore the flows have been calculated based on a rating created by the consultant’s software Hydrolog. Hydrolog has been set up to create ratings for structures based on the international standards.

As the weirs are not to international standard, check gaugings have been undertaken at both sites to confirm the rating.

2. SITE REFERENCE NO. G1

2.1 Introduction

The Nivus and Pressure transmitter were installed within the channel. The Nivus was attached to a stainless bracket, which was pushed into the silt to sit the Nivus above the bed level. The pressure transmitter was housed within a metal stilling tube attached to the gaugeboard near the bank. The Instrumentation was cabled back to a cabinet located on top of the bank within a small enclosed fenced area.

Figure 1: Enclosed Cabinet and Gaugeboard installation (Site 1)

Figure 2: General Overview of Site 1

2.2 Velocity Index Rating

A velocity index relationship is used to convert velocity measured by the instrument (Index Velocity) in to mean velocity to give calibrated flow values for the site. A review of the velocity index rating has been completed using the gaugings completed up to November 2014, using the consultant's Rating Manager (RatMan) software.

Both linear and polynomial relationships were reviewed. The polynomial was not used as it is likely to overestimate higher velocities when extrapolated. The linear equation provides a better relationship although there is still some scatter within the data points.

This Stage-Area relationship can be described as the equation below:

Area = $0.8354*h^4-2.6273*h^3+4.0584*h^2-1.3652*h-0.0587$

Where h = Stage (m)

The velocity index rating can be described as below:

Velocity Index rating = (1.024*Vi)-(0.0110*h)+0.009

Where: Vi = Index Velocity (m/s)
h = Stage (m)

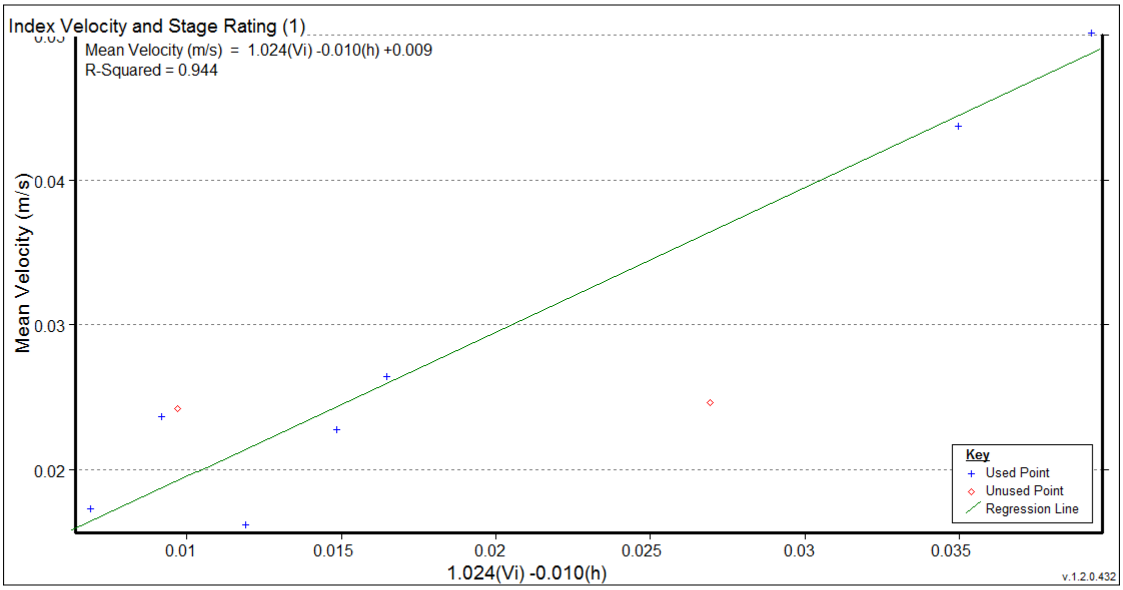


Figure 3: Velocity Index Rating for Site 1

As shown in the graph above, there is a reasonable relationship between the index velocity and the mean velocity. To improve this relationship the last two gaugings that were undertaken were removed.

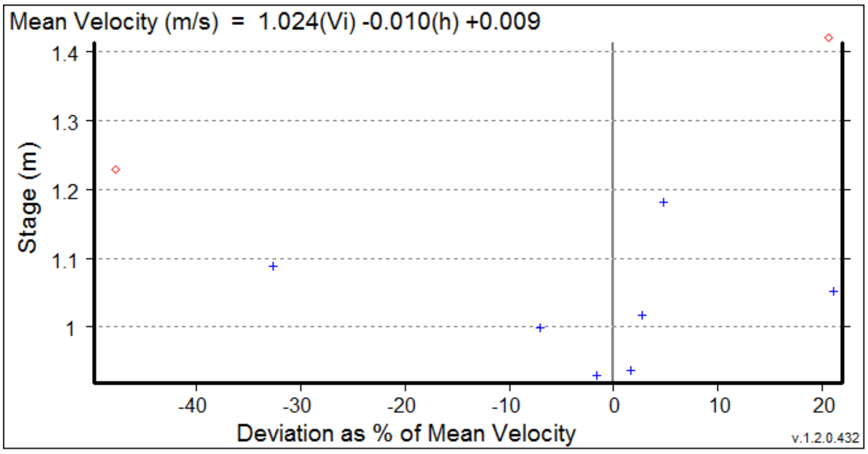


Figure 4: Deviation Plot for Site 1 Rating

2.3 Results

The table below summarises the percentage difference between the gauged and the calibrated flow. Please note that due to the nature of this channel, continuous accurate flow measurement is difficult to undertake and therefore there will be some high percentage variations. The key issues at this site are the continually altering of the channel area due to the silt deposition. Also the extreme low velocities which seem to occur for the majority of the monitoring period.

Data/Time	Stage	Index Velocity	Gauged Flow	Area	Mean Velocity	Calibrated Flow	% Difference
05/02/2014 14:07	1.182	0.018	0.078	4.52	0.0144	0.0652	16.44
06/03/2012 10:31	1.015	0.048	0.183	3.65	0.0470	0.1714	6.35
13/05/2014 15:26	0.929	0.043	0.141	3.23	0.0428	0.1382	1.99
18/06/2014 08:37	0.936	0.025	0.086	3.26	0.0243	0.0793	7.83
23/07/2014 09:46	1.053	0.019	0.091	3.84	0.0169	0.0648	28.83
20/08/2014 08:57	1.088	0.022	0.065	4.02	0.0196	0.0786	-20.90
17/09/2014 08:48	0.997	0.024	0.081	3.56	0.0226	0.0804	0.68
15/10/2014 08:07	1.229	0.038	0.118	4.78	0.0344	0.1643	-39.26
10/11/2014 15:56	1.419	0.023	0.143	5.93	0.0169	0.1005	29.73

Table 1: Summary of gaugings for Site 1

The velocity index rating has limitations, which are due to the gaugings, only being completed within a certain range. These ranges are stated below, and the data outside of this range needs to be treated with caution.

	Minimum Value	Maximum Value
Gauged Flow	0.065	0.183
Index Velocity	0.018	0.048
Stage	0.929	1.419
Stage (Area)	-	1.54

Table 2: Summary of velocity index range (site 1)

The stage (area) value states the bank height at which AMEC completed the survey too. Therefore for stages above this value, the site is out of bank and the area is unknown.

3. SITE REFERENCE NO. G3

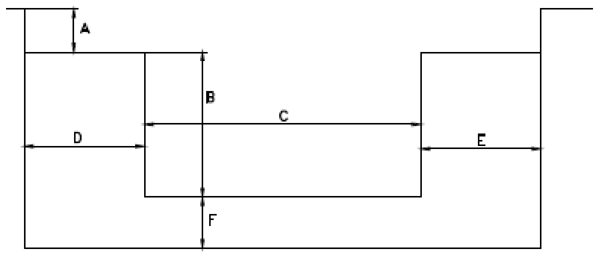
3.1 General Details

The site has a rectangular weir plate installed with an upstream pressure transmitter measuring the water level.

Figure 5: Photos of Site 3 installation

3.2 Stage Discharge Rating

A stage discharge relationship has been establish based on the intertional standard for thin plate weirs.

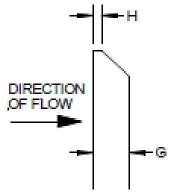


Measure	Location	mm
A	Top of weir to top of chamber	
B	Depth of Weir	500
C1	Top Width of Weir	1135
C2	Mid Width of Weir	1135
C3	Bottom Width of Weir	1135
D	Weir to left bank of chamber	1200
E	Weir to left bank of chamber	1200
F1	Bottom of weir to base of chamber u/s	300
F2	Bottom of weir to base of chamber d/s	500
G	Crest Thickness	2
H	Plate Thickness	2
	Location of Chamfer (if any)	N/A

All dimensions in mm

Channel Width	3000
Uninterrupted Approach Length	At least 5000

WEIR PLATE DETAIL



Initial Theorectical Rating

Max Stage

0.07 Q = 1.9899*(h-0.001)^1.5005

0.5 Q = 2.0260*(h-0.002)^1.5145

$Q = C(h-a)^B$

Where:

Q = Discharge (in cumecs)

h = stage (in metres)

a, B and C = Constants

Rating Table

Stage (m)	Flow (m^3/s)
0.06	0.0285
0.1	0.0601
0.125	0.0848
0.15	0.1122
0.175	0.1421
0.2	0.1744
0.225	0.2088
0.25	0.2452
0.275	0.2836
0.3	0.3238
0.325	0.3659
0.35	0.4096
0.375	0.4550
0.4	0.5020
0.425	0.5505
0.45	0.6005
0.475	0.6520
0.5	0.7048

3.3 Results

Data/Time	Stage	Average Velocity	Average Flow	Rated Flow	% Diff
05/02/2014 16:37	0.08	0.004	0.043	0.043	0
06/03/2014 15:29	0.057	0.049	0.028	0.025	-12
13/05/2014 10:00	-0.087	0	0	0	0
18/0602014 15:31	-0.013	0	0	0	0
22/07/2014 13:07	-0.104	0	0	0	0
19/08/2014 11:30	-0.1	0	0	0	0
17/09/2014 11:20	-0.123	0	0	0	0
14/10/2014 10:51	-0.092	0	0	0	0
11/11/2014 11:40	-0.04	0	0	0	0

Table 3: Summary of gaugings for Site 3

During the second period of monitoring, due to the lowering of the weir plate at site 4, this site is typically not flowing, therefore no gaugings have been undertaken since March 2014. This site does flow but only during high flow events.

On the 9th January 2014, channel modifications were undertaken by the wildlife trust which causes the water level to drop below the weir plate, for a brief time period.

Only two gaugings have been undertaken during the monitoring period, and both of this have a reasonable agreement with the rating and therefore the rating has not been altered.

4. SITE REFERENCE G4

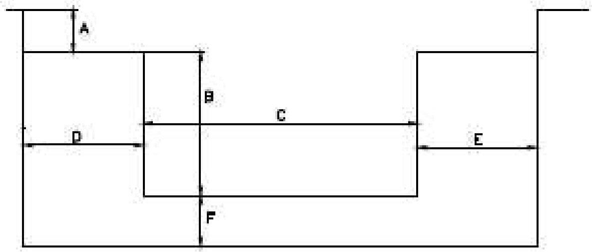
4.1 General Details

The site has a rectangular weir plate installed with an upstream and downstream pressure transmitter measuring the water level.

Figure 6: Photos of Site 4, pre and post weir plate movement.

4.2 Stage Discharge Rating

A stage discharge relationship has been establish based on the intertional standard for thin plate weirs.



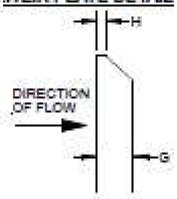
Measure	Location	mm
A	Top of weir to top of chamber	
B	Depth of Weir	500
C1	Top Width of Weir	960
C2	Mid Width of Weir	960
C3	Bottom Width of Weir	960
D	Weir to left bank of chamber	1320
E	Weir to left bank of chamber	1320
F1	Bottom of weir to base of chamber u/s	300*
F2	Bottom of weir to base of chamber d/s	800*
G	Crest Thickness	2
H	Plate Thickness	2
	Location of Chamfer (if any)	N/A

All dimensions in mm

* approx measurements due to silt build up

Channel Width	3600
Uninterrupted Approach Length	At least 5000

WEIR PLATE DETAIL



Initial Theorectical Rating

$Q = 1.6761*(h-0.001)^{1.5}$

$Q = C(h-a)^B$

Where:

Q = Discharge (in cumecs)

h = stage (in metres)

a, B and C = Constants

Rating Table

Stage (m)	Flow (m^3/s)
0.06	0.0240
0.1	0.0522
0.125	0.0732
0.15	0.0964
0.175	0.1217
0.2	0.1488
0.225	0.1777
0.25	0.2083
0.275	0.2404
0.3	0.2740
0.325	0.3091
0.35	0.3456
0.375	0.3834
0.4	0.4224
0.425	0.4628
0.45	0.5043
0.475	0.5470
0.5	0.5908

In April, the weir plate was moved down to reduce the water levels in the channels for the livestock entering the field for the summer.

4.3 Results

Data/Time	Stage	Average Velocity	Average Flow	Rated Flow	% Diff	Comment
06/02/2014 09:49	0.064	0.021	0.028	0.027	-3.70	
06/03/2014 14:26	0.061	0.021	0.031	0.025	-24.00	
13/05/2014 10:11	0.076	0.035	0.033	0.034	2.94	
18/06/2014 15:13	0.067	0.038	0.03	0.028	-7.14	
22/07/2014 13:57	0.078	0.037	0.038	0.036	-5.56	
20/08/2014 11:51	0.085	0.049	0.033	0.041	19.51	
17/09/2014 12:17	0.078	0.05	0.027	0.036	25.00	
15/10/2014 11:41	0.11	0.054	0.038	0.0389	2.36	Drowned - adjusted rating
11/11/2014 12:14	0.15	0.05	0.046	0.034	-35.29	Drowned - adjusted rating

Table 4: Summary of gaugings for Site 4

Please note that since the weir plate was moved, the water over the weir will hit the metal bar at 0.145 m, which means the data will be suspect, as the rating will no longer apply. Also due to lowering of the weir, it is often getting drowned from the downstream water level, therefore for stages around 0.1 and above a drowned rating has been applied for the more recent time period.

On the 9th January 2014, channel modifications were undertaken by the wildlife trust which causes the water level to drop below the weir plate, for a brief time period.

The majority of the gaugings have a reasonable agreement with the rating and therefore the rating has not been altered. The last two gaugings did not have a good agreement, due to the structure being drowned. A different rating was applied to the structure for when it is drowned, this rating was based on the international standard.

5. SITE REFERENCE NO. G5

5.1 General Details

The Nivus and Pressure transmitter were installed within the channel. The Nivus was attached to a paving slab, which was surrounded by a number of slabs to reduce silt levels. The width of the channel was also reduced at this site to improve the velocities at the Nivus location. The pressure transmitter was housed within a metal stilling tube attached to the gaugeboard near the bank. The Instrumentation was cabled back to a cabinet located on top of the bank within a small enclosed fenced area.

Figure 7: Photos of the installation at site 6

5.2 Velocity Index Rating

A velocity index relationship is used to convert velocity measured by the instrument (Index Velocity) in to mean velocity to give calibrated flow values for the site. A review of the velocity index rating has been completed using the gaugings completed up to November 2014, using the consultant's Rating Manager (RatMan) software.

Both linear and polynomial relationships were reviewed. The polynomial was not used as it is likely to overestimate higher velocities when extrapolated. The linear equation provides a better relationship although there is still some scatter within the data points.

This Stage-Area relationship can be described as the equation below:

$\text{Area} = 1.5207*h^4-5.3247*h^3+7.6867*h^2-0.5866*h-0.0048$

Where h = Stage (m)

The velocity index rating can be described as below:

Velocity Index rating = (1.397*Vi)-(0.0191*h)-0.027

Where: Vi = Index Velocity (m/s)
h = Stage (m)

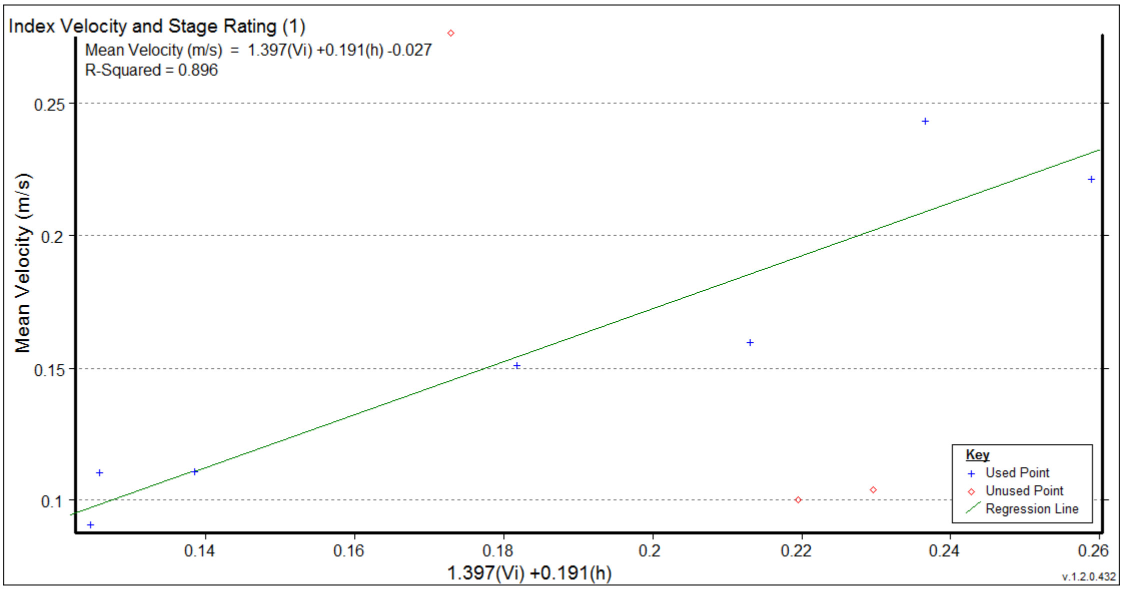


Figure 8: Velocity Index Rating Site 5

As shown in the graph above, there is a reasonable relationship between the index velocity and the mean velocity. To improve this relationship three gaugings that were undertaken were removed.

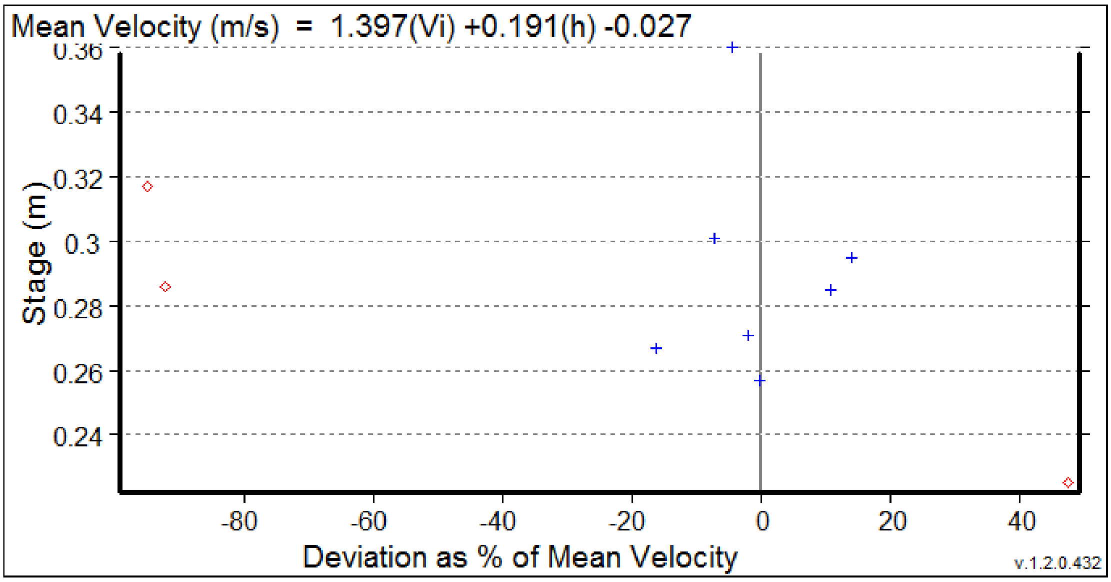


Figure 9: Deviation Plot for Site 5

5.3 Results

The table below summarises the percentage difference between the gauged and the calibrated flow. Please note that due to the nature of this channel, continuous accurate flow measurement is difficult to undertake and therefore there will be some high percentage variations. The key issues at this site are the continually altering of the channel area due to the silt deposition. Also the extreme low velocities which seem to occur for the majority of the monitoring period.

Data/Time	Stage	Index Velocity	Gauged Flow	Area	Mean Velocity	Calibrated Flow	% Difference
03/01/2014 10:06	0.295	0.129	0.084	0.38	0.2096	0.0787	6.32
05/02/2014 08:59	0.36	0.136	0.123	0.57	0.2318	0.1314	-6.82
05/03/2014 15:45	0.271	0.093	0.043	0.31	0.1547	0.0484	-12.44
13/05/2014 08:29	0.225	0.093	0.052	0.21	0.1459	0.0299	42.43
18/06/2014 13:36	0.285	0.051	0.035	0.35	0.0987	0.0344	1.67
22/07/2014 15:31	0.257	0.064	0.028	0.28	0.1115	0.0310	-10.70
20/08/2014 10:37	0.286	0.118	0.032	0.35	0.1925	0.0676	-111.35
17/09/2014 10:23	0.301	0.048	0.033	0.39	0.0975	0.0382	-15.85
15/10/2014 16:03	0.317	0.121	0.043	0.44	0.2026	0.0885	-105.89
11/11/2014 10:32	0.267	0.116	0.044	0.30	0.1860	0.0563	-27.92

Table 5: Summary of Results for Site 5

The velocity index rating has limitations, which are due to the gaugings, only being completed within a certain range. These ranges are stated below, and the data outside of this range needs to be treated with caution.

The current rating, does cause extreme high flow values, which are suspect as they are two high for this channel and also based on the other sites. Therefore the high flow data for this site needs to be treated as suspect.

	Minimum Value	Maximum Value
Gauged Flow	0.028	0.123
Index Velocity	0.048	0.136
Stage	0.225	0.36
Stage (Area)	-	1.74

Table 6: Summary of rating range (Site 5)

The stage (area) value states the bank height at which AMEC completed the survey too. Therefore for stages above this value, the site is out of bank and the area is unknown.

6. SITE REFERENCE NO. G6A

6.1 General Details

The Nivus and Pressure transmitter were installed within the channel. The Nivus was attached to a stainless bracket, which was pushed into the silt to sit the Nivus above the bed level. The pressure transmitter was housed within a metal stilling tube attached to the gaugeboard near the bank. The Instrumentation was cabled back to a cabinet located on top of the bank within a small enclosed fenced area.

Figure 10: Photo of the enclosure (Site 6)

Figure 11: Photo of gaugeboard and Stilling Well

6.2 Velocity Index

A velocity index relationship is used to convert velocity measured by the instrument (Index Velocity) in to mean velocity to give calibrated flow values for the site. A review of the velocity index rating has been completed using the gaugings completed up to November 2014, using the consultant's Rating Manager (RatMan) software.

Both linear and polynomial relationships were reviewed. The polynomial was not used as it is likely to overestimate higher velocities when extrapolated. The linear equation provides a better relationship although there is still some scatter within the data points.

This Stage-Area relationship can be described as the equation below:

Area = $-1.1002*h^3+4.0148*h^2+0.8568*h-0.0167$

Where h = Stage (m)

The velocity index rating can be described as below:

Velocity Index rating = (0.452*Vi)-(0.060*h)+0.061

Where: Vi = Index Velocity (m/s)
h = Stage (m)

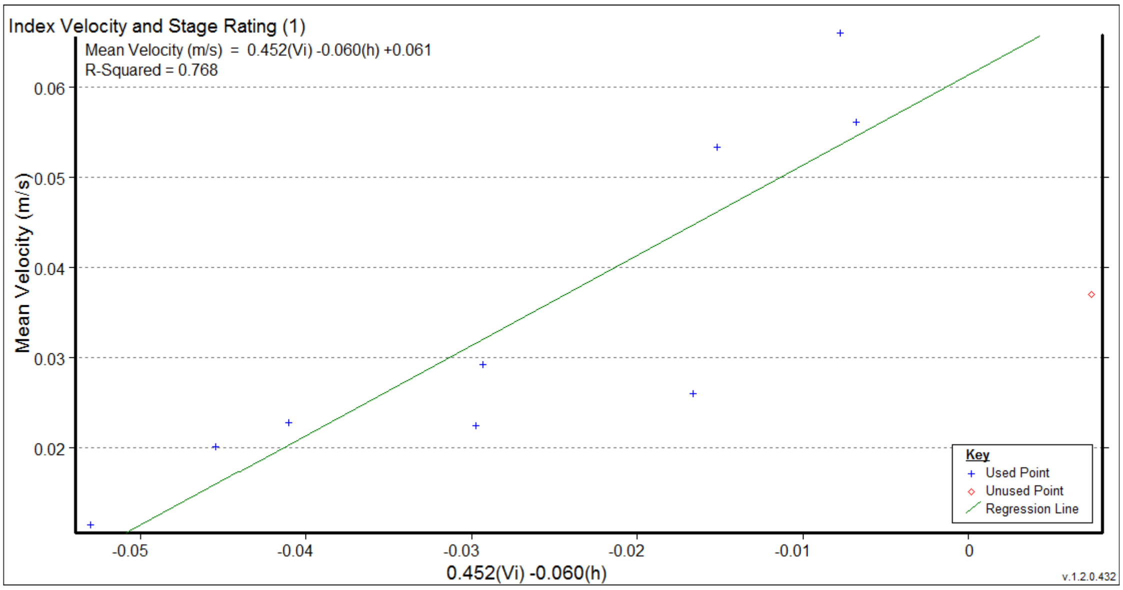


Figure 12: Velocity Index Rating for Site 6

As shown in the graph above, there is a reasonable relationship between the index velocity and the mean velocity. To improve this relationship one gauging that was undertaken was removed.

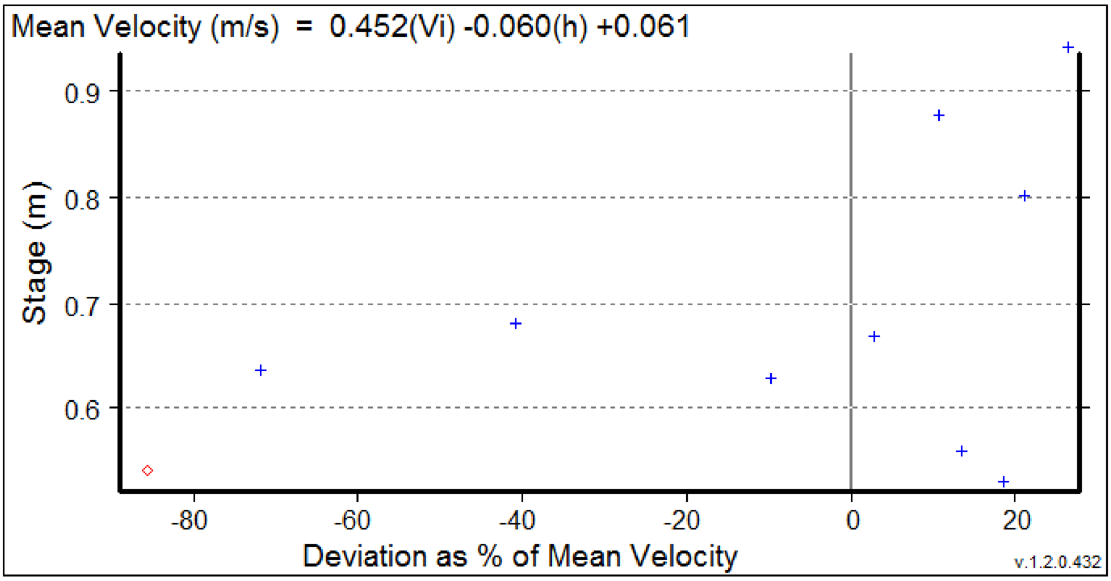


Figure 13: Devation Plot of rating (site 6)

6.3 Results

The table below summarises the percentage difference between the gauged and the calibrated flow. Please note that due to the nature of this channel, continuous accurate flow measurement is difficult to undertake and therefore there will be some high percentage variations. The key issues at this site are the continually altering of the channel area due to the silt deposition. Also the extreme low velocities which seem to occur for the majority of the monitoring period.

Data/Time	Stage	Index Velocity	Gauged Flow	Area	Mean Velocity	Calibrated Flow	% Difference
03/01/2014 12:16	0.878	0.026	0.07	3.09	0.0201	0.0619	11.52
05/02/2014 10:50	0.669	0.074	0.114	2.02	0.0543	0.1099	3.58
06/03/2014 11:55	0.56	0.0409	0.082	1.53	0.0459	0.0702	14.44
12/05/2014 15:51	0.531	0.0536	0.093	1.41	0.0534	0.0750	19.34
18/06/2014 11:31	0.542	0.0885	0.054	1.45	0.0685	0.0994	-84.13
22/07/2014 13:43	0.637	0.048	0.049	1.87	0.0445	0.0833	-70.08
20/08/2014 14:03	0.682	0.025	0.047	2.09	0.0314	0.0655	-39.27
16/09/2014 13:19	0.63	0.019	0.054	1.84	0.0318	0.0585	-8.40
14/10/2014 14:37	0.801	0.006	0.054	2.68	0.0157	0.0419	22.32
10/11/2014 14:47	0.941	0.008	0.039	3.43	0.0082	0.0280	28.31

Table 7: Results of gauging site 6

The velocity index rating has limitations, which are due to the gaugings, only being completed within a certain range. These ranges are stated below, and the data outside of this range needs to be treated with caution.

	Minimum Value	Maximum Value
Gauged Flow	0.039	0.114
Index Velocity	0.006	0.0885
Stage	0.531	0.941
Stage (Area)	-	1.05

Table 8: Summary of rating range (site 6)

The stage (area) value states the bank height at which AMEC completed the survey too. Therefore for stages above this value, the site is out of bank and the area is unknown.

7. SITE REFERENCE NO. 7A
7.1 Installation Report

The Nivus and Pressure transmitter were installed within the channel. The Nivus was attached to a stainless bracket, which was pushed into the silt to sit the Nivus above the bed level. The pressure transmitter was housed within a metal stilling tube attached to the gaugeboard near the bank. The Instrumentation was cabled back to a cabinet located on top of the bank within a small enclosed fenced area.

Figure 14: Photos of the installation at site 7

7.2 Velocity Indexing

A velocity index relationship is used to convert velocity measured by the instrument (Index Velocity) in to mean velocity to give calibrated flow values for the site. A review of the velocity index rating has been completed using the gaugings completed up to November 2014, using the consultant's Rating Manager (RatMan) software.

Both linear and polynomial relationships were reviewed. The polynomial was not used as it is likely to overestimate higher velocities when extrapolated. The linear equation provides a better relationship although there is still some scatter within the data points.

This Stage-Area relationship can be described as the equation below:

Area = $5.747*h^4-8.0523*h^3+6.5995*h^2+0.0942*h-0.014$

Where h = Stage (m)

The velocity index rating can be described as below:

Velocity Index rating = (0.811*Vi)-(0.002*h)+0.002

Where: Vi = Index Velocity (m/s)
h = Stage (m)

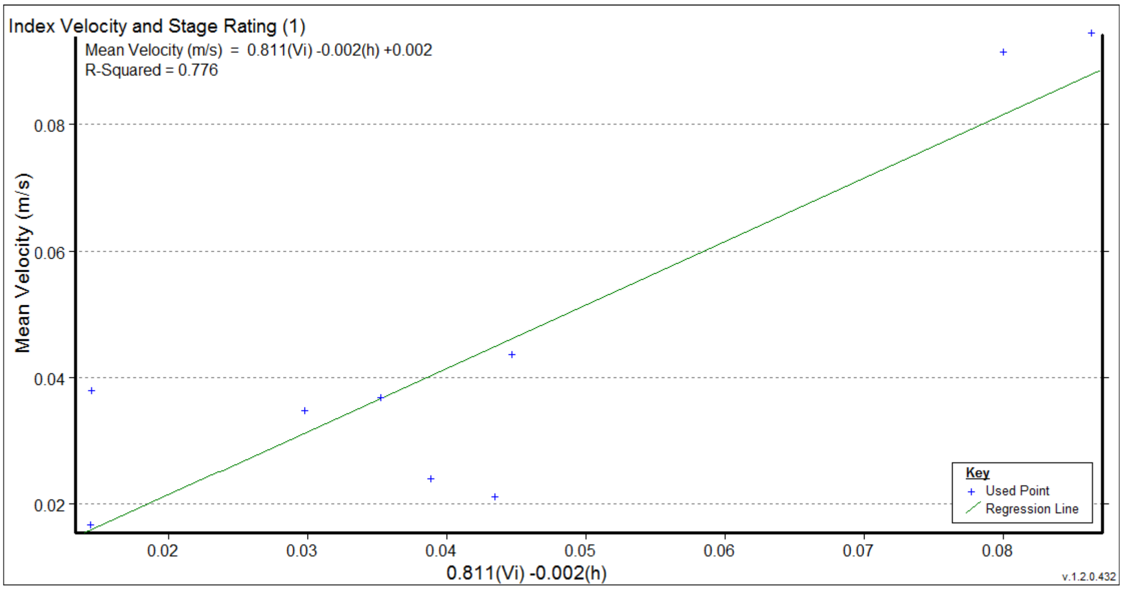


Figure 15: Velocity Index Rating site 7

As shown in the graph above, there is a reasonable relationship between the index velocity and the mean velocity. All gaugings were used in the creation of the relationship.

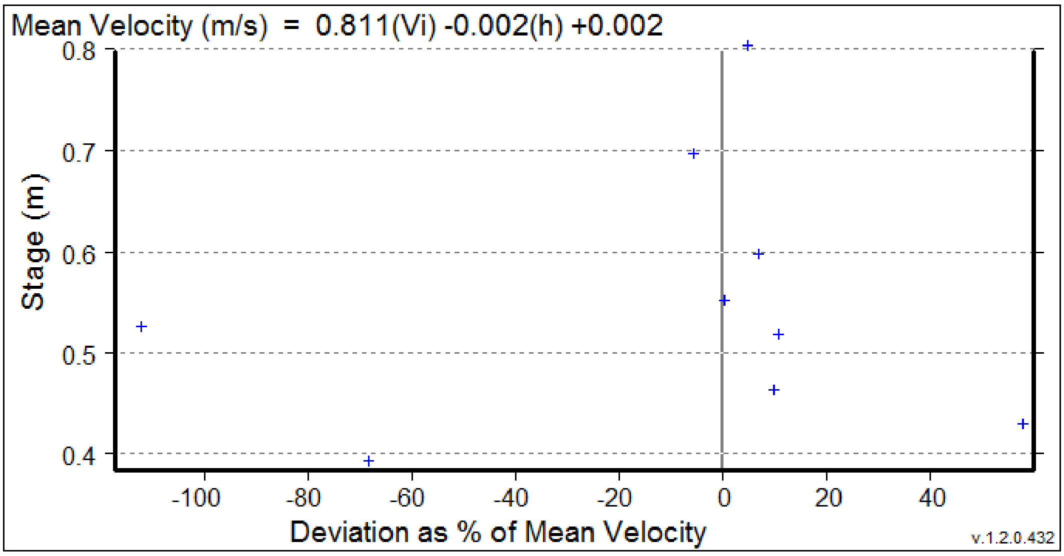


Figure 16: Deviation Plot for rating (Site 7)

7.3 Results

The table below summarises the percentage difference between the gauged and the calibrated flow. Please note that due to the nature of this channel, continuous accurate flow measurement is difficult to undertake and therefore there will be some high percentage variations. The key issues at this site are the continually altering of the channel area due to the silt deposition. Also the extreme low velocities which seem to occur for the majority of the monitoring period.

Data/Time	Stage	Index Velocity	Gauged Flow	Area	Mean Velocity	Calibrated Flow	% Difference
03/01/2014 13:39	0.803	0.02	0.043	2.54	0.0166	0.0422	1.97
05/02/2014 12:47	0.598	0.087	0.131	1.42	0.0714	0.1010	22.90
06/03/2014 08:45	0.519	0.072	0.101	1.10	0.0594	0.0655	35.13
13/05/2014 12:56	0.429	0.03	0.031	0.80	0.0255	0.0204	34.27
18/06/2014 10:14	0.393	0.019	0.017	0.69	0.0166	0.0115	32.47
23/07/2014 12:14	0.527	0.02	0.024	1.13	0.0172	0.0195	18.94
19/08/2014 15:21	0.552	0.029	0.045	1.23	0.0244	0.0300	33.37
16/09/2014 14:52	0.462	0.029	0.032	0.91	0.0246	0.0223	30.37
15/10/2014 09:32	0.697	0.051	0.082	1.89	0.0420	0.0792	3.40

Table 9: Summary of gaugings for Site 7

The velocity index rating has limitations, which are due to the gaugings, only being completed within a certain range. These ranges are stated below, and the data outside of this range needs to be treated with caution.

	Minimum Value	Maximum Value
Gauged Flow	0.017	0.131
Index Velocity	0.019	0.108
Stage	0.393	0.803
Stage (Area)	-	0.95

Table 10: Summary of range (Site 7)

The stage (area) value states the bank height at which AMEC completed the survey too. Therefore for stages above this value, the site is out of bank and the area is unknown.

Appendix B

Derived Full Catchment Descriptors

Catchment Name	LEIS_ABBEY	LEIS_LEISTON	LEIS_LOVERS	LEIS_SIZEWELL	LEIS_UPPER	MINS_DOCWRA	MINS_EASTBR
CATCHMENT NGR	TM 44950 63550	TM 45100 63250	TM 45750 63850	TM 47550 62800	TM 46650 64750	TM 46800 67750	TM 46200 66000
CATCHMENT E/N	644950, 263550	645100, 263250	645750, 263850	647550, 262800	646650, 264750	646800, 267750	646200, 266000
AREA	2.43	1.85	0.59	3.92	0.72	2.39	1.02
ALTBAR	17	16	12	12	12	17	10
ASPBAR	95	64	127	51	98	145	35
ASPVAR	0.47	0.46	0.54	0.44	0.59	0.32	0.73
BFIHOST	0.63	0.817	0.84	0.89	0.855	0.898	0.881
DPLBAR	1.52	1.55	0.82	2.27	1.43	1.24	0.98
DPSBAR	11	10	8	8.7	6.7	25	6.7
FARL	1	1	1	1	1	1	1
LDP	2.98	2.99	1.92	4.98	2.75	2.43	1.87
PROPWET	0.26	0.26	0.26	0.26	0.26	0.26	0.26
RMED-1H	10.9	10.9	10.9	11	11	11	11
RMED-1D	29.7	29.2	29.4	30.2	30	31.2	30.3
RMED-2D	37.6	37.3	37.2	38.6	37.7	38.6	38.1
SAAR	594	592	591	586	590	585	590
SAAR4170	596	596	595	593	594	591	593
SPRHOST	27.83	20.71	18.21	14.52	17.05	14.39	13.29
URBCONC1990	0.222	0.737	-999999	0.677	-999999	-999999	-999999
URBEXT1990	0.0072	0.1671	0	0.054	0	0	0.0037
URBLOC1990	0.692	0.874	-999999	1.452	-999999	-999999	-999999
C	-0.02012	-0.01931	-0.02	-0.019	-0.02	-0.01982	-0.02
D1	0.29619	0.28897	0.29722	0.29642	0.30078	0.31709	0.30561
D2	0.27589	0.27802	0.26877	0.27952	0.26924	0.27027	0.26852
D3	0.22919	0.22195	0.22338	0.21276	0.22282	0.23446	0.22978
E	0.31048	0.30931	0.31033	0.30893	0.3109	0.31	0.30946
F	2.5034	2.50384	2.49504	2.51222	2.5014	2.50924	2.50509
C (1km)	-0.02	-0.019	-0.02	-999999	-0.02	-0.019	-0.02
D1 (1km)	0.295	0.29	0.297	-999999	0.299	0.32	0.307
D2 (1km)	0.268	0.272	0.267	-999999	0.272	0.27	0.267
D3 (1km)	0.223	0.223	0.22	-999999	0.215	0.223	0.231
E (1km)	0.31	0.309	0.311	-999999	0.311	0.309	0.31
F (1km)	2.493	2.5	2.489	-999999	2.506	2.517	2.504

Note: Extracted from FEH CD_ROM Version 3 on 18th September 2014

Catchment Name	MINS_MIDDLETON	MINS_POTTERS	MINS_SCOTT	MINS_THERBERTON	MINS_US	MINS_WARKBARN	MINS_WASH	MINS_WESTLETON
CATCHMENT NGR	TM 43000 68000	TM 44550 66200	TM 47250 67050	TM 44200 66650	TM 43000 68100	TM 44700 67450	TM 44100 67000	TM 43900 67850
CATCHMENT Easting, Northing	643000, 268000	644550, 266200	647250, 267050	644200, 266650	643000, 268100	644700, 267450	644100, 267000	643900, 267850
AREA	4.48	2.99	1.58	1.95	46	0.83	2.87	3.47
ALTBAR	23	13	12	17	33	17	17	19
ASPBAR	72	30	163	70	94	202	70	203
ASPVAR	0.41	0.44	0.64	0.48	0.23	0.5	0.4	0.38
BFIHOST	0.391	0.724	0.886	0.63	0.375	0.893	0.578	0.787
DPLBAR	2.77	1.7	1.92	1.77	8.54	0.69	2.32	2.11
DPSBAR	22.4	11.1	14.4	15.1	24.6	34.3	16.8	14.8
FARL	1	1	0.929	1	0.973	1	1	1
LDP	5	3.38	3.44	3.45	17.1	1.47	4.55	4.13
PROPWET	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
RMED-1H	10.9	10.9	11	10.9	10.9	10.9	10.9	11
RMED-1D	30.3	30.3	30.5	30.3	29.4	30.8	30.3	31.2
RMED-2D	38.2	38.1	38.2	38.1	37.4	38.4	38.1	38.9
SAAR	596	595	586	597	594	590	598	595
SAAR4170	600	598	588	599	601	596	600	600
SPRHOST	41.2	25.33	13.54	29.54	41.05	14.01	32.44	21.94
URBCONC1990	-999999	0.111	-999999	0.091	0.445	-999999	-999999	0.254
URBEXT1990	0.002	0.005	0	0.0083	0.0084	0	0	0.0209
URBLOC1990	-999999	0.505	-999999	0.576	0.808	-999999	-999999	0.698
C	-0.02042	-0.02002	-0.01942	-0.02	-0.02177	-0.02	-0.02	-0.02085
D1	0.30686	0.30436	0.30946	0.30222	0.30266	0.31232	0.30496	0.32004
D2	0.27695	0.27181	0.27285	0.27387	0.29503	0.27227	0.27213	0.27705
D3	0.25212	0.23576	0.23111	0.24122	0.24766	0.24168	0.24695	0.24254
E	0.31004	0.30972	0.30903	0.309	0.31215	0.3098	0.30946	0.31164
F	2.5044	2.50504	2.5055	2.5092	2.49204	2.50868	2.50747	2.50682
C (1km)	-0.021	-0.02	-0.019	-0.02	-0.021	-0.02	-0.02	-0.02
D1 (1km)	0.317	0.306	0.308	0.311	0.317	0.308	0.311	0.314
D2 (1km)	0.278	0.269	0.275	0.263	0.278	0.276	0.263	0.273
D3 (1km)	0.25	0.233	0.228	0.248	0.25	0.234	0.248	0.242
E (1km)	0.312	0.308	0.309	0.309	0.312	0.309	0.309	0.31
F (1km)	2.5	2.506	2.504	2.508	2.5	2.504	2.508	2.509

Note: Extracted from FEH CD_ROM Version 3 on 18th September 2014

Appendix C

Flood estimation calculation record

Flood estimation calculation record

Introduction

This document is a supporting document to the Environment Agency's flood estimation guidelines. It provides a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This version of the record is for studies where flood estimates are needed at multiple locations.

Contents

Approval

	Signature	Name and qualifications	For Environment Agency staff: Competence level (see below)
Calculations prepared by:	[redacted]	[redacted] *Sc*	Level 1
Calculations checked by:		[redacted] *BSc MSc*	Level 3
Calculations approved by:			Level 3

Environment Agency competence levels are covered in Section 2.1 of the flood estimation guidelines:

- Level 1 – Hydrologist with minimum approved experience in flood estimation
- Level 2 – Senior Hydrologist
- Level 3 – Senior Hydrologist with extensive experience of flood estimation

ABBREVIATIONS

AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

1 Method statement

1.1 Overview of requirements for flood estimates

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	The hydrological assessment is required to review and, where necessary, update the design event hydrology set out in the Flood Study Report undertaken by JBA Consulting in 2013 on behalf of the Environment Agency. **Previous work** JBA (2013) Study – Flood Study of River Minsmere and Leiston Drain: commissioned by EA. RHDHV (2014) – Review of above JBA Flood Study (2013): commissioned by client EDF Energy. The RHDHV (2014) review found that only the ReFH method was used to calculate flood flow estimations. The statistical method was considered in the JBA Study (2013) but it was found that there were no suitable donors for the Middleton stage gauge so the approach was rejected. The RHDHV review found constraints to the use of the ReFH and ReFH2 approaches. They are not the most appropriate methodology for this study as flooding in this hydrological system is dominated by storm durations considerably longer than the critical storm duration of 121 hours. This can mean that ReFH and ReFH2 methods grossly overestimate the flood volume in such instances. The RHDHV (2014) review suggested the application and testing of alternative methods. The estimated design flows and hydrographs generated from this assessment will serve as input data to an updated version of the JBA 2013 hydraulic model. This modelling exercise informs a comprehensive Flood Risk Assessment (FRA) being prepared in support of a Development Consent Order (DCO) application. In summary, the hydraulic model will be used to assess the flood risk to and from the proposed Sizewell C (SZC) power station development. The return periods of key interest are 1 in 20 year, 1 in 100 year and 1 in 1,000 year. In addition, a number of climate change scenarios were considered. The Environment Agency Flood estimation guidelines, Operational Instruction 097_08, Issued 26/06/2012 were used in the hydrology assessment as they were the most current guidelines at the time of the assessment.

1.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	For further details of the catchment refer to SZC FRA Hydrology Review and Design Event Methodology Report Chapter 1.1 Overview of hydrological features. In summary, the River Minsmere begins life as the River Yox watercourse, its source rising near the village of Peasenhall, Suffolk before changing name at the village of Yoxford. It is approximately 13km long flowing in a south easterly

direction through the 1,000ha coastal marshland habitat of Minsmere RSPB Nature Reserve where it ultimately discharges into the North Sea. Throughout the catchment land use is dominated by rural occupancy and agricultural use.

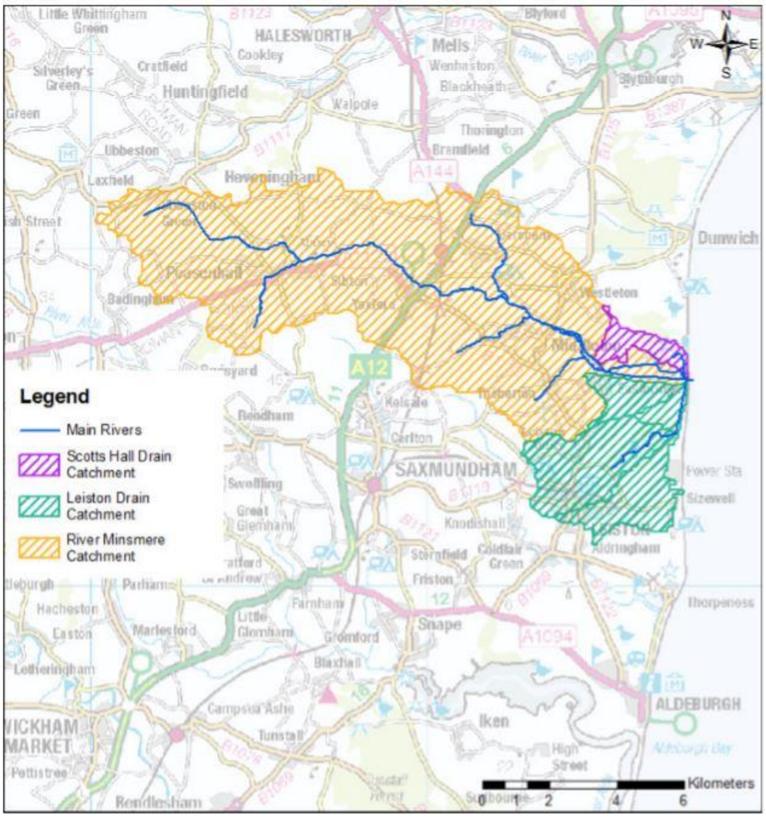


Figure 1. River Minsmere Catchment Area and Drainage Network

The overlying soils of the catchment are highly permeable and are formed mostly of sedimentary layers of sand, gravel, alluvial clay and silt and peat.

The overall catchment area is primarily considered to be a lowland catchment.

1.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	The stage data record from the Middleton gauge is considered by the Environment Agency to be too unreliable for inclusion in the National River Flow Archive (NRFA) UK Peak Flow (previously HiFlows) dataset. Although the station has a long record that runs from September 1976 onwards the Environment Agency have low confidence in the rating curve for the gauge due to drowning and bypassing of the weir at higher flows.

1.4 Gauging stations (flow or level)

(at the sites of flood estimates or nearby at potential donor sites)

Watercourse	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catchment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
River Minsmere	Middleton	Environment Agency	35022	643097 267952	50.53 km²	Stage only	01/01/1993 to 18/11/2014

1.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Middleton	N/A	N/A	N/A	N/A	N/A	N/A
Give link/reference to any further data quality checks carried out			High level review of rating curve, provided by the Environment Agency, was carried out.			

1.6 Rating equations

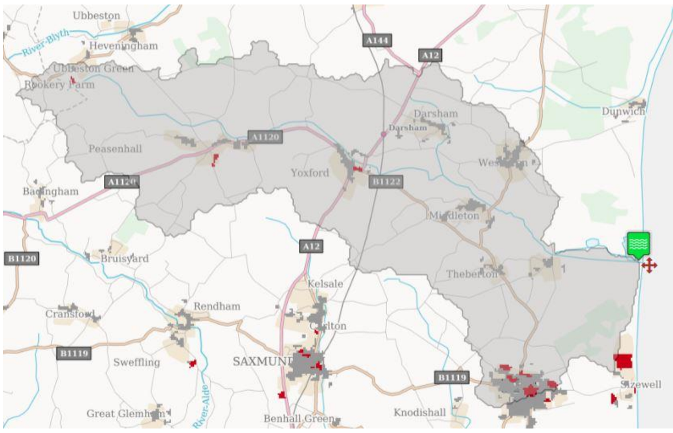
Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Middleton	-	High level review	Environment Agency expressed concerns related to function of rating curve. Understood that existing rating curve functions only for low flows and is seen as unreliable for out of bank flows.
Give link/reference to any rating reviews carried out		Summarised in SZC Hydrology Report Section 2.2	

1.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)	A review of the existing rating curve was carried out, but the Middleton gauge is Stage only with no Flow records to match or improve the rating curve. In addition, there are concerns that the rating curve is only reliable during low flow conditions. In summary, the 1D-2D linked hydraulic model was used to generate a new rating curve which included floodplain flows.				
Historic flood data – give link to historic review if carried out.	A review of Historic Flood Events is provided in SZC Hydrology Report Section 2.1				
Flow data for events	None available				
Rainfall data for events	N/A				
Potential evaporation data	None available				
Results from previous studies	Review of JBA (2013) Study has formed an integral part of the assessment				
Other data or information (e.g. groundwater, tides)	None available				

1.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	The FEH methodologies are considered appropriate to use for the upland catchment hydrology calculations. However, the lowland area does not lend itself for FEH methodologies and an alternative method is required. None of the catchments considered as part of this study are smaller than 0.5km^2. However, the area under investigation is hugely complex due to the myriad of drainage channels and interconnected watercourses that are a legacy of past land use around this area. The hydrological nature of the lowland area requires some caution in the application of FEH methods due to the large volumes of water stored during high flow events and the low BFIHOST values associated with the catchments. The upland system has been divided into a number of sub-catchments with FEH catchment descriptors for each obtained from the FEH CD ROM Version 3. These sub-catchments represent the inflows to the lowland system.
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Flooding is likely and does occur in the lowland area due to the large volumes of water that are stored across the area. The Environment Agency's Flood Map for Planning shows a significant area of floodplain in the lowland section of the catchment.  Figure 2. Environment Agency's Flood Map for Planning showing Flood Zone 3
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland	A number of the tributary sub-catchments at Sizewell have a BFIHOST value >0.65, as such this would suggest that ReFH method is unsuitable for this location. This is due to the fact the area is predominantly formed of sand and is highly permeable. The hydrological representation of the system differs for Upland and Lowland components and are defined as follows: • The Upland system consists of the Minsmere Gauge catchment, Leiston Drain catchment and

catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	other upland sub-catchments. • Along the upstream reaches the ground level is above 3mAOD and as such is considered appropriate for the FEH methodology and inflows have been derived in the form of hydrological boundary units. • The Lowland system comprises the downstream lowland reaches of the catchment, known as the Sizewell Belts and Minsmere Levels, the ground level is below 3mAOD. Due to this variation in catchment the direct rainfall approach was applied within Flood Modeller for these areas.  Figure 3. FEH Web Service Urban Extent 2000 in Minsmere catchment area (grey - suburban; red – urban)
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Initially it was proposed to explore the potential to use standard FEH methods of flood estimation: Statistical Method / FEH Rainfall Runoff Method / ReFH / ReFH2 The overall catchment has been split into smaller sub-catchments due to the complexity of the hydrology in this area.
Software to be used (with version numbers)	FEH CD-ROM v3.0[1] WINFAP-FEH v3.0.002[2] / ReFH spreadsheet / ReFH2 / FLOOD MODELLER (formerly ISIS)

[1] FEH CD-ROM v3.0 © NERC (CEH). © Crown copyright. © AA. 2009. All rights reserved.

[2] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

2 Locations where flood estimates required

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

2.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
MINS_US	Minsmere River		643000	268100	46.000	50.160
MINS_WESTLE	Drain	Westleton Common	643900	267850	3.470	3.840
MINS_WARKBAR	Drain		644700	267450	0.830	1.090
MINS_WASH	Drain	The Drift	644100	267000	2.870	3.620
MINS_THEBERT	Drain	Theberton Hall Farm	644200	266650	1.950	1.710
MINS_POTTERS	Drain	Holly Tree Farm	644550	266200	2.990	3.490
MINS_SCOTT			647250	267050	1.580	0.980
MINS_DOCWRA			646800	267750	2.390	2.860
MINS_EASTBR			646200	266000	1.020	1.230
LEIS_UPPER			646650	264750	0.720	1.229
LEIS_LOVERS			645750	263850	0.590	0.854
LEIS_ABBEY			644950	263550	2.430	2.830
LEIS_LEISTON			645100	263250	1.850	2.210
LEIS_SIZEWELL			647550	262800	3.920	3.644
Reasons for choosing above locations		These sites represent key inflow points along the Minsmere River, and/or main sites of interest.				

2.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
MINS_US	0.973	0.26	0.375	8.54	24.6	594	41.05	0.0084	*
MINS_WESTLE	1.000	0.26	0.787	2.11	14.8	595	21.94	0.0209	*
MINS_WARKBAR	1.000	0.26	0.893	0.69	34.3	590	14.01	0.0000	*
MINS_WASH	1.000	0.26	0.578	2.32	16.8	598	32.44	0.0000	*
MINS_THEBERT	1.000	0.26	0.630	1.77	15.1	597	29.54	0.0083	*
MINS_POTTERS	1.000	0.26	0.724	1.70	11.1	595	25.33	0.0050	*
MINS_SCOTT	0.929	0.26	0.886	1.92	14.4	586	13.54	0.0000	*
MINS_DOCWRA	1.000	0.26	0.898	1.24	25.0	585	14.39	0.0000	*
MINS_EASTBR	1.000	0.26	0.881	0.98	6.7	590	13.29	0.0037	*
LEIS_UPPER	1.000	0.26	0.855	1.43	6.7	590	17.05	0.0000	*
LEIS_LOVERS	1.000	0.26	0.840	0.82	8.0	591	18.21	0.0000	*
LEIS_ABBEY	1.000	0.26	0.630	1.52	11.0	594	27.83	0.0072	*
LEIS_LEISTON	1.000	0.26	0.630	1.55	10.0	592	27.83	0.1671	*
LEIS_SIZEWELL	1.000	0.26	0.890	2.27	8.7	586	14.52	0.054	*

2.3 Checking catchment descriptors

<table>
<tr><td>Record how catchment boundary was checked and describe any changes (refer to maps if needed)</td><td>Catchment area boundaries (AREAS) were checked against LiDAR data and Ordnance Survey mapping and adjusted accordingly to a finer resolution to better represent the delineation lines of the sub-catchments. In a number of places, the intervening areas have been added to the sub-catchment AREA, such that the sum of the sub-catchment AREA's equals the total AREA to the outfall.

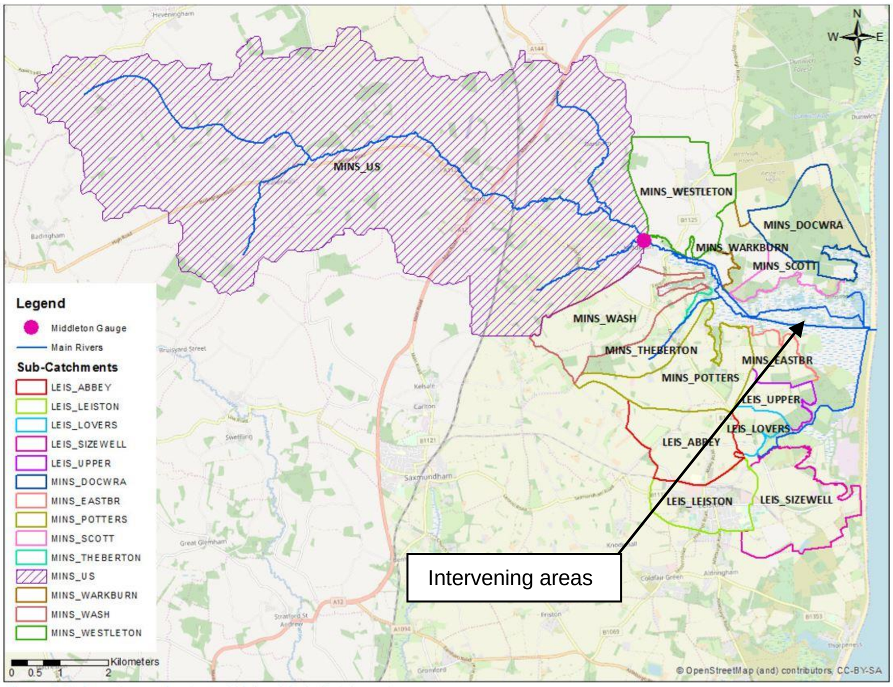


Figure 4. Location and extent of Upland Sub-catchments.</td></tr>
<tr><td>Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.</td><td></td></tr>
<tr><td>Source of URBEXT</td><td>FEH CD Rom Version 3 & $URBEXT_{1990}$</td></tr>
<tr><td>Method for updating of URBEXT</td><td>N/A</td></tr>
</table>

3 Statistical method

3.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites Mention: • Number of potential donor sites available • Distances from subject site • Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors • Quality of flood peak data Include a map if necessary. Note that donor catchments should usually be rural.	It should be noted that this analysis was undertaken in 2014 and was based on the limited data available at the time. The Middleton gauge was identified as the only potential donor site from which to derive QMED and higher flows. Due to the large distances between the catchment centroids and potential donor site, the "Data Transfer" method set out within the Flood Estimation Guidelines (Environment Agency, 2012), which includes the term "a" to account for geographical distance, led to very low adjustment factors, making the adjustment process unreliable, and values were retained the same as the FEH rainfall runoff method based on FEH derived cd's. In addition, the Statistical Method was found not to be appropriate for this study as no other appropriate donor gauges were found with reliable data to undertake the statistical analysis and comparison.

3.2 Donor sites chosen and QMED adjustment factors – N/A

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjust-ment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)

Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).	n/a

3.3 Overview of estimation of QMED at each subject site – N/A

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer				If more than one donor		Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	Weight	Weighted average adjustment factor	

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	

Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?	
Which version of the urban adjustment was used for QMED, and why?	

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050Error! Bookmark not defined. should be used. If the original FEH equation has been used, say so and give the reason why.

The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.

The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial estimate from catchment descriptors.

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

3.4 Derivation of pooling groups – Initial pooling groups were assessed for 6 reference sites

The composition of the initial pooling groups can be provided upon request; however, this method was not progressed in the hydrological assessment due to the lack of available pooling group gauging stations.

Name of group	**Site code from whose descriptors group was derived**	**Subject site treated as gauged?** (enhanced single site analysis)	**Changes made to default pooling group, with reasons** Note also any sites that were investigated but retained in the group.	**Weighted average L-moments**, L-CV and L-skew, (before urban adjustment)

Notes

Pooling groups were derived using the revised procedures from Science Report SC050050 (2008).

The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.

3.5 Derivation of flood growth curves at subject sites

Site code	**Method** (SS, P, ESS, J)	**If P, ESS or J, name of pooling group (3.4)**	**Distribution used and reason for choice**	**Note any urban adjustment or permeable adjustment**	**Parameters of distribution** (location, scale and shape) after adjustments	**Growth factor for 100-year return period**

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (3.4)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustments	Growth factor for 100-year return period

Notes

Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis

A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.

Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010).

Growth curves were derived using the revised procedures from Science Report SC050050 (2008)..

3.6 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2								

4 Revitalised flood hydrograph (ReFH) method

4.1 Parameters for ReFH model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible, so it is not essential to enter them in the table.

Site code	**Method:** OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	**Tp (hours)** Time to peak	**C_{max} (mm)** Maximum storage capacity	**BL (hours)** Baseflow lag	**BR** Baseflow recharge
MINS01_6151	CD				
MINS01_2628	CD				
LEIS_4265	CD				
SIZE01_1768	CD				
MIDDLETON GS	CD				
Brief description of any flood event analysis carried out (further details should be given below or in a project report)					

4.2 Design events for ReFH method

Site code	**Urban or rural**	**Season of design event (summer or winter)**	**Storm duration (hours)**	**Storm area for ARF** (if not catchment area)
MINS01_6151			22	
MINS01_2628			15	
LEIS_4265			9	
SIZE01_1768			11	
MIDDLETON GS			22	
			22	
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?				

4.3 Flood estimates from the ReFH method

Site code	**Flood peak (m^3/s) for the following return periods (in years)**								
	2	**100**							
MINS01_6151		19.3							
MINS01_2628		2.1							
LEIS_4265		1.1							
SIZE01_1768		0.3							
MIDDLETON GS		21.2							

5 FEH rainfall-runoff method

5.1 Parameters for FEH rainfall-runoff model

Methods: FEA : Flood event analysis
LAG : Catchment lag
DT : Catchment descriptors with data transfer from donor catchment
CD : Catchment descriptors alone
BFI : SPR derived from baseflow index calculated from flow data

Site code	Rural (R) or urban (U)	Tp(0): method	Tp(0): value (hours)	SPR: method	SPR: value (%)	BF: method	BF: value (m^3/s)	If DT, numbers of donor sites used (see Section 5.2) and reasons
MINS_US		Calculated	12.531	Observed	50	Calculated	0.226	n/a
MINS_WESTLE		Calculated	6.613	Observed	50	Calculated	0.018	n/a
MINS_WARKBAR		Calculated	3.104	Observed	50	Calculated	0.004	n/a
MINS_WASH		Calculated	7.486	Observed	50	Calculated	0.018	n/a
MINS_THEBERT		Calculated	6.420	Observed	50	Calculated	0.008	n/a
MINS_POTTERS		Calculated	7.116	Observed	50	Calculated	0.016	n/a
MINS_SCOTT		Calculated	4.140	Observed	50	Calculated	0.004	n/a
MINS_DOCWRA		Calculated	4.692	Observed	50	Calculated	0.010	n/a
MINS_EASTBR		Calculated	6.367	Observed	50	Calculated	0.005	n/a
LEIS_UPPER		Calculated	7.945	Observed	50	Calculated	0.005	n/a
LEIS_LOVERS		Calculated	5.568	Observed	50	Calculated	0.004	n/a
LEIS_ABBEY		Calculated	6.644	Observed	50	Calculated	0.013	n/a
LEIS_LEISTON		Calculated	3.036	Observed	50	Calculated	0.009	n/a
LEIS_SIZEWELL		Calculated		Observed		Calculated		

-

5.2 Donor sites for FEH rainfall-runoff parameters – N/A

No.	Watercourse	Station	Tp(0) from data (A)	Tp(0) from CDs (B)	Adjustment ratio for Tp(0) (A/B)	SPR from data (C)	SPR from CDs (D)	Adjust-ment ratio for SPR (C/D)

5.3 Inputs to and outputs from FEH rainfall-runoff model

Site code	Storm duration (hours)	Storm area for ARF (if not catchment area)	Flood peaks (m^3/s) for the following return periods (in years)							
			2	5	10	20	50	100	100yr +CC	1000
MINS_US	121	n/a	7.717	10.257	12.046	14.003	16.655	18.643		27.952
MINS_WESTLE	121	n/a	0.615	0.821	0.968	1.128	1.346	1.510		2.282
MINS_WARKBAR	121	n/a	0.176	0.236	0.279	0.326	0.389	0.437		0.664
MINS_WASH	121	n/a	0.556	0.745	0.878	1.025	1.225	1.375		2.084
MINS_THEBERT	121	n/a	0.263	0.352	0.415	0.485	0.579	0.650		0.984
MINS_POTTERS	121	n/a	0.525	0.704	0.830	0.969	1.158	1.301		1.973

Site code	Storm duration (hours)	Storm area for ARF (if not catchment area)	Flood peaks (m³/s) for the following return periods (in years)							
			2	**5**	**10**	**20**	**50**	**100**	**100yr +CC**	**1000**
MINS_SCOTT	121	n/a	0.150	0.202	0.239	0.280	0.335	0.377		0.575
MINS_DOCWRA	121	n/a	0.456	0.613	0.724	0.847	1.014	1.140		1.736
MINS_EASTBR	121	n/a	0.187	0.251	0.296	0.346	0.414	0.465		0.705
LEIS_UPPER	121	n/a	0.051	0.067	0.079	0.092	0.109	0.122		0.183
LEIS_LOVERS	121	n/a	0.035	0.046	0.054	0.062	0.074	0.083		0.125
LEIS_ABBEY	121	n/a	0.118	0.155	0.182	0.211	0.250	0.280		0.419
LEIS_LEISTON	121	n/a	0.092	0.108	0.125	0.145	0.175	0.202		0.327
LEIS_SIZEWELL	121	n/a	0.849	1.148	1.362	1.597	1.917	2.160		3.309
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?					No, testing of the hydraulic model for the 1 in 100 year event identified that the 121 hour storm duration event is the worst case critical storm duration at the key area of interest.					

6 Discussion and summary of results

6.1 Comparison of results from different methods

This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods. Blank cells indicate that results for a particular site were not calculated using that method.

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	ReFH	Other method	Other method	ReFH	Other method	Other method

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The FEH Rainfall Runoff method was considered the most appropriate technique to use for deriving the upland sub-catchments inflows. A unique Unit Hydrograph generated from observed river and rainfall data using the RHDHV in-house software model 'Strip_UK' was the best approach to improve the hydrological input parameters of percentage runoff (PR), time to peak (Tp) and Unit Hydrograph (UH) shape to the FEH rainfall runoff method. These parameters have been used to improve and modify the sub-catchment inflows. This technique performs the same FEH standard process of extracting a Unit Hydrograph but does so from local observed river and rainfall data. At the main upland inflow point (MINS_US) the unit hydrograph is based on the re-rated Middleton river gauge rating curve. The Middleton gauge was re-rated by RHDHV (2014) to provide greater confidence in higher flows. The ReFH derived upland inflow for the River Minsmere (MINS_US) UH was found to be very similar to the Strip_UH analysis based on the updated and re-rated Middleton gauge rating curve. The statistical method was carried out with the Middleton gauge serving as the donor site to allow a comparison. However, due to the absence of reliable data at the gauge it was considered that the statistical analysis method is not appropriate for this study. A hybrid approach to the design hydrology estimations has been used to derive the inflows to the total catchment area as this was considered reasonable due to the variable nature of the catchment. The downstream reaches around the proposed SZC development are low and flat (<3mAOD) and have a large attenuation effect in response to rainfall over the lower catchment. It was considered that the Direct Rainfall method applied in Flood Modeller was the most appropriate approach to use for the inflows derived around this area.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The main inflows into the system are the River Minsmere and the Leiston Drain catchments. The catchment area is formed of consolidated sand and alluvial deposits of gravel and silt and is therefore recognised as being highly permeable in nature. Highly permeable catchments are unsuitable for the ReFH method. Alternative methods therefore must be considered.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	ReFH is considered unsuitable to use for generating reliable return period events greater than 1 in 150 year events. The ReFH method is not appropriate for catchments with BFIHOST values >0.65.
Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH **3** 12.5 or the factorial standard error from Science Report SC050050 (2008).	N/A - due to the variation in the approaches adopted and limited availability of data for calibration there are known uncertainties in the design flows
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	It should be emphasised that the results of this assessment should be considered in the context of the needs of this study and should be reassessed in any future studies considered.
Give any other comments on the study, for example suggestions for additional work.	Limited availability of observed data means that calibration data is sparse.

6.4 Checks

Are the results consistent, for example at confluences?	There is limited data with which to calibrate the hydrological assessment; however, checks have been carried out within the hydraulic modelling process and these indicate that the results are in accordance with observed data.
What do the results imply regarding the return periods of floods during the period of record?	Where possible they have been compared with anecdotal evidence to ensure that the results seem to be appropriate.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	N/A for this study to the range of approaches adopted for various sub-catchments.
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	N/A for this study to the range of approaches adopted for various sub-catchments.
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	N/A to this study
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	A comparison with the JBA 2013 study has found that similar issues with data derivation has been experienced. The current hydrological assessment built on the preceding JBA 2013 study and therefore should be used in preference to the previous work.
Are the results compatible with the longer-term flood history?	Flood history: anecdotal evidence collected from interviews with RSPB conservation wardens for the area have provided a reliable understanding of the hydrological behaviour of the system during high

	flow events. Key points on flooding within the Reserve: • Flooding tends to occur due to closure of the Minsmere sluice gates at high tide increasing backwater depth and causing bank overspill. • As the water depths increase upstream of the Minsmere control the bank overspill flows into the North and South Levels over low points in both banks of the Minsmere River. • Main concerns over flooding are due to the backwater effect and downstream structure blockages.
Describe any other checks on the results	Checks against anecdotal information provided by the RSPB has been carried out.

6.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)							
	2	5	10	20	50	100	100+CC	1000
MINS_US	7.717	10.257	12.046	14.003	16.655	18.643		27.952
MINS_WESTLE	0.615	0.821	0.968	1.128	1.346	1.510		2.282
MINS_WARKBAR	0.176	0.236	0.279	0.326	0.389	0.437		0.664
MINS_WASH	0.556	0.745	0.878	1.025	1.225	1.375		2.084
MINS_THEBERT	0.263	0.352	0.415	0.485	0.579	0.650		0.984
MINS_POTTERS	0.525	0.704	0.830	0.969	1.158	1.301		1.973
MINS_SCOTT	0.150	0.202	0.239	0.280	0.335	0.377		0.575
MINS_DOCWRA	0.456	0.613	0.724	0.847	1.014	1.140		1.736
MINS_EASTBR	0.187	0.251	0.296	0.346	0.414	0.465		0.705
LEIS_UPPER	0.051	0.067	0.079	0.092	0.109	0.122		0.183
LEIS_LOVERS	0.035	0.046	0.054	0.062	0.074	0.083		0.125
LEIS_ABBEY	0.118	0.155	0.182	0.211	0.250	0.280		0.419
LEIS_LEISTON	0.092	0.108	0.125	0.145	0.175	0.202		0.327
LEIS_SIZEWELL								

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	

7 Annex - supporting information

7.1 Additional supporting information

Appendix D

Derived Sub-Catchment Hydrographs – All Storm Durations

Figure A-6.1 – Figure A-6.8 present hydrographs derived for the main inflow sub-catchment, i.e. MINS_US for the 1 in 100y return period event and various storm durations.

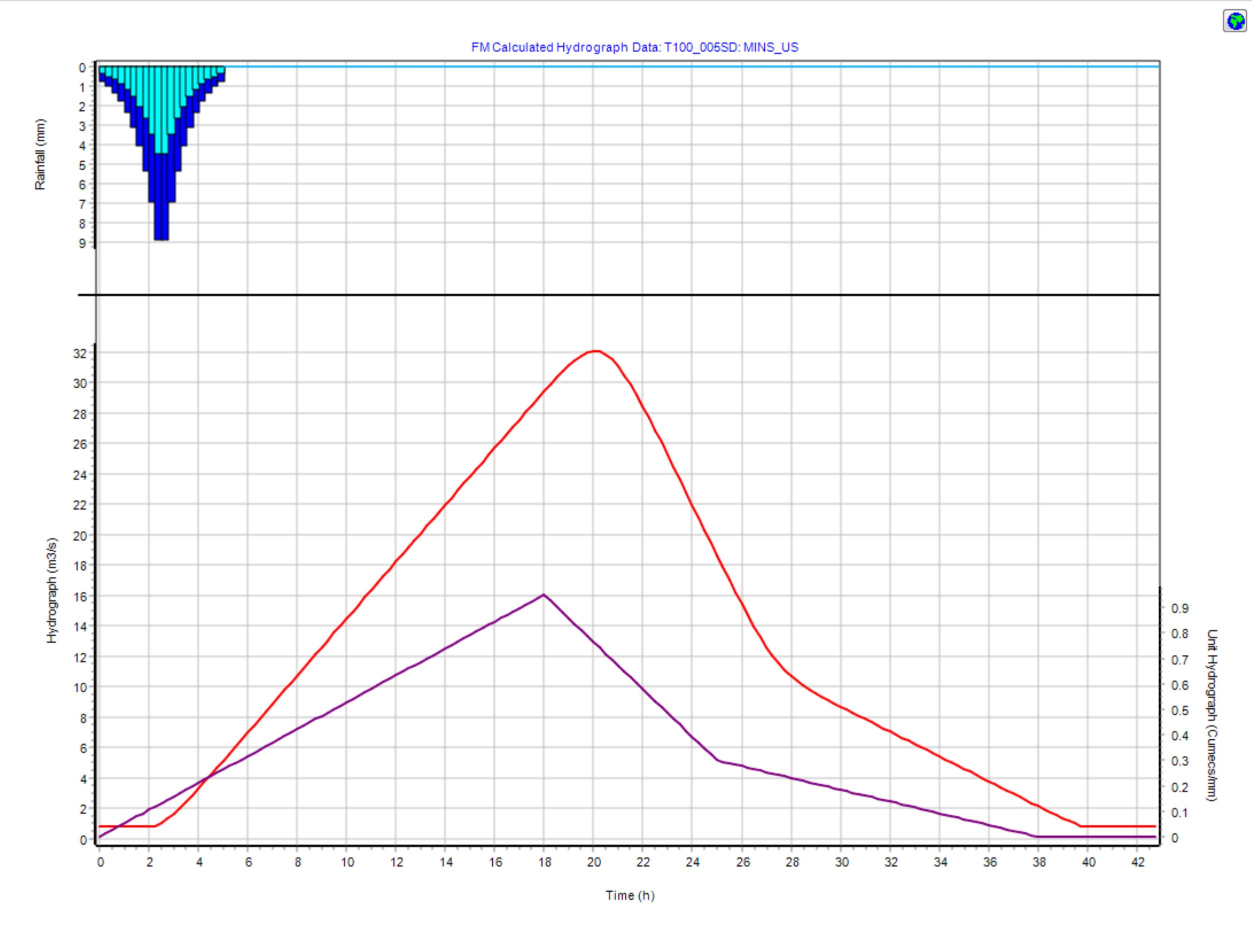


Figure A-6.1. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 5-hour storm duration

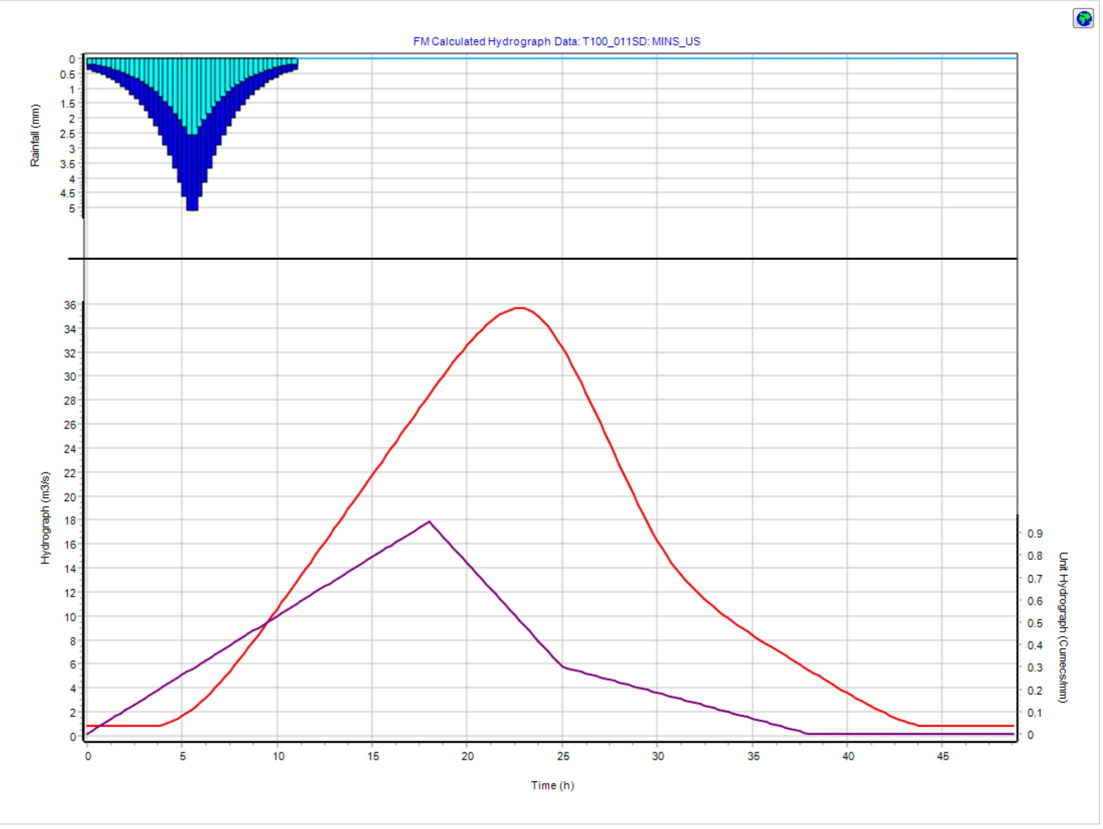


Figure A-6.2. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 11-hour storm duration

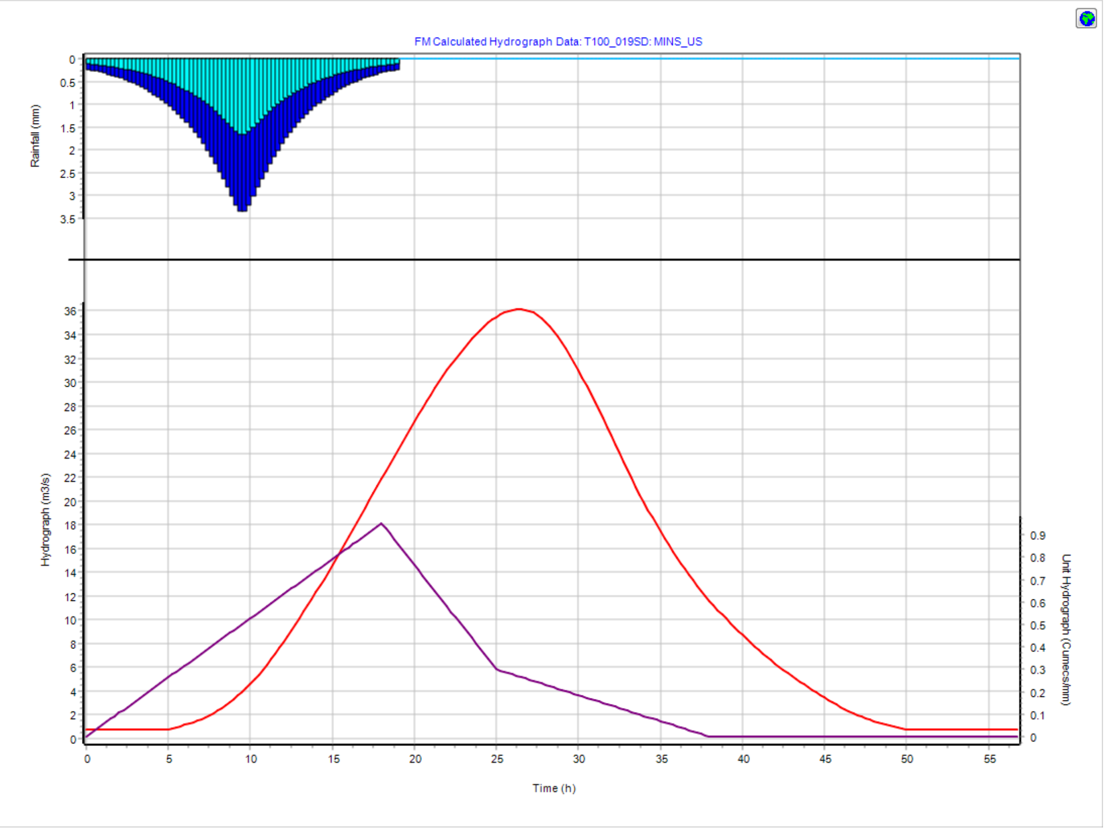


Figure A-3. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 19-hour storm duration

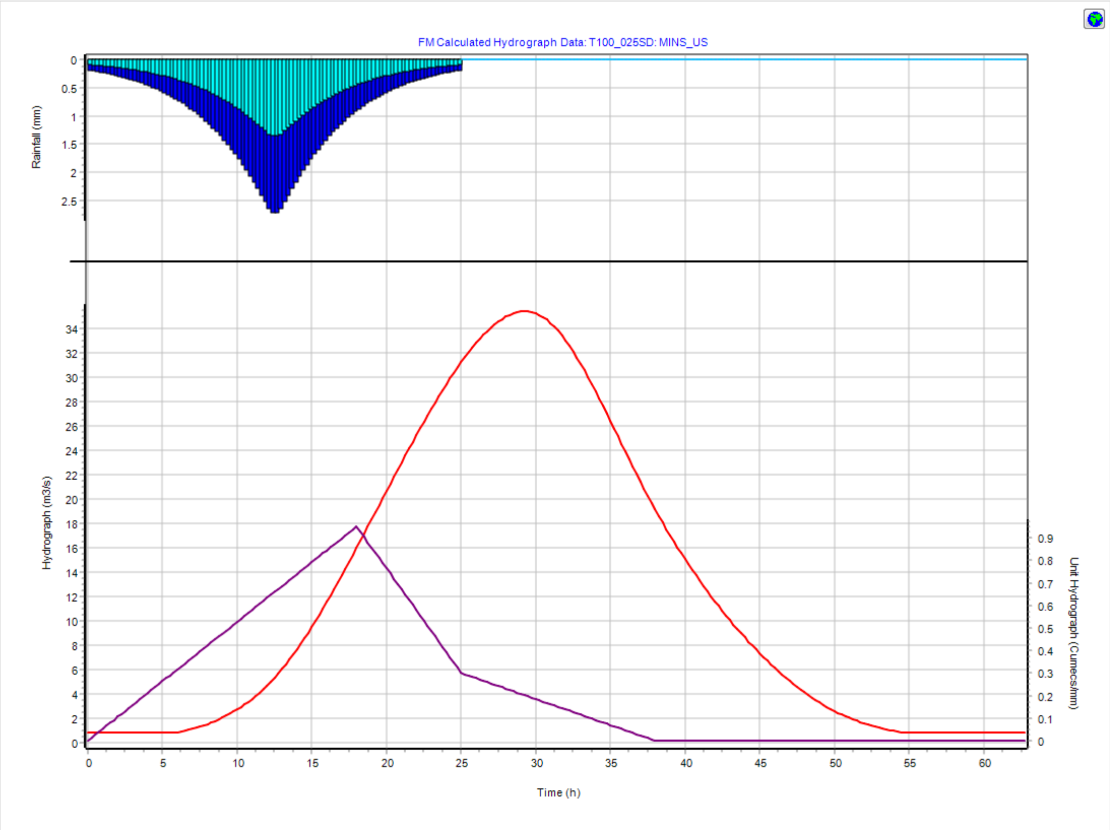


Figure A-4. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 25-hour storm duration

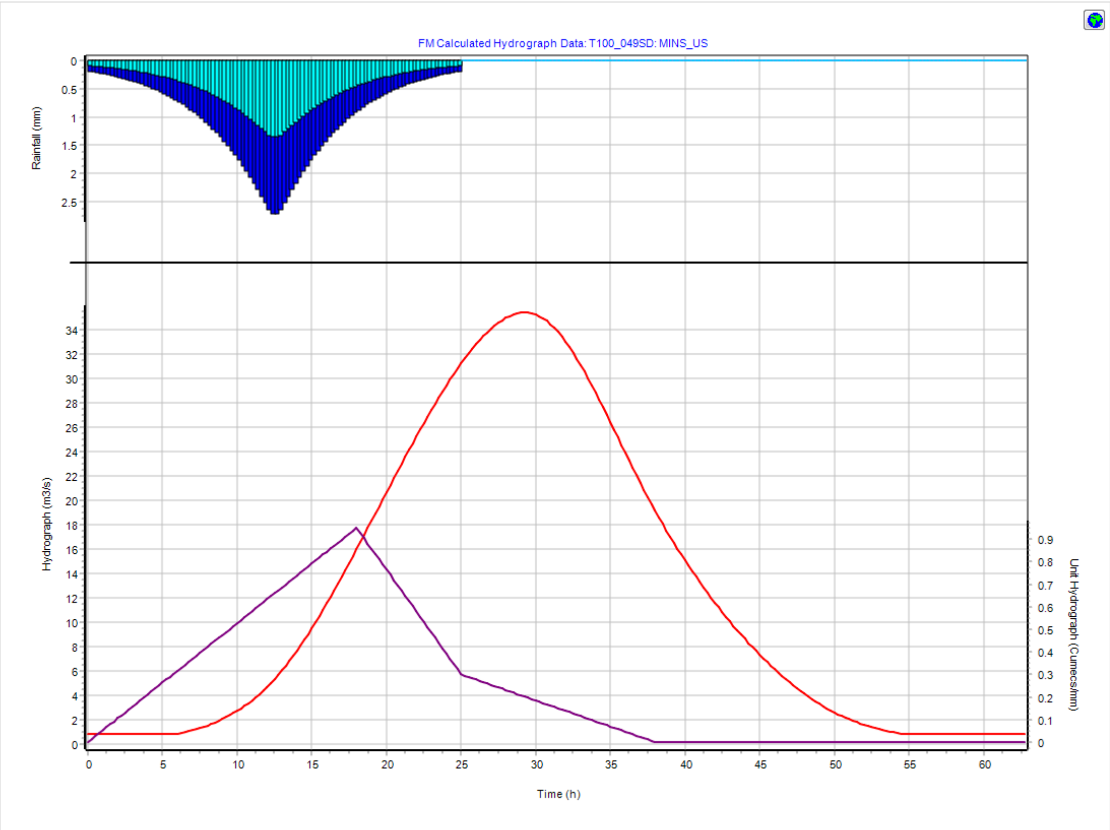


Figure A-6.3. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 49-hour storm duration

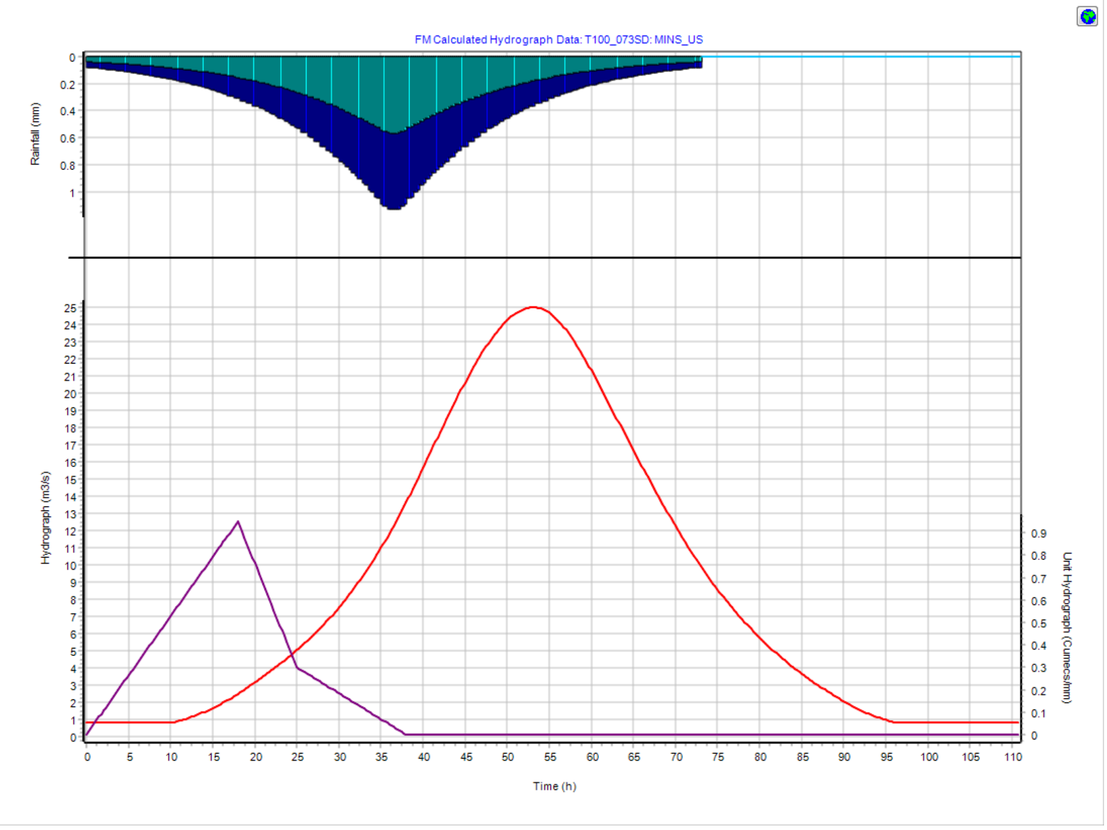


Figure A-6.4. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 73-hour storm duration

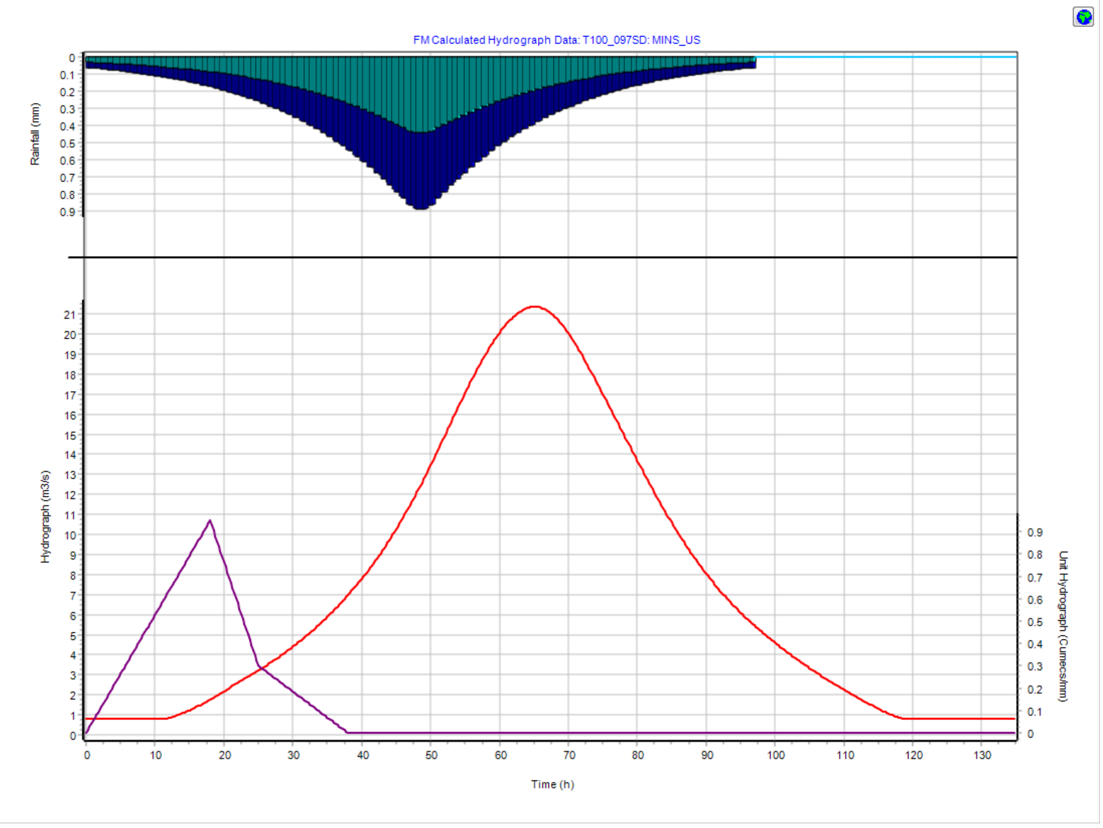


Figure A-6.5. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 97-hour storm duration

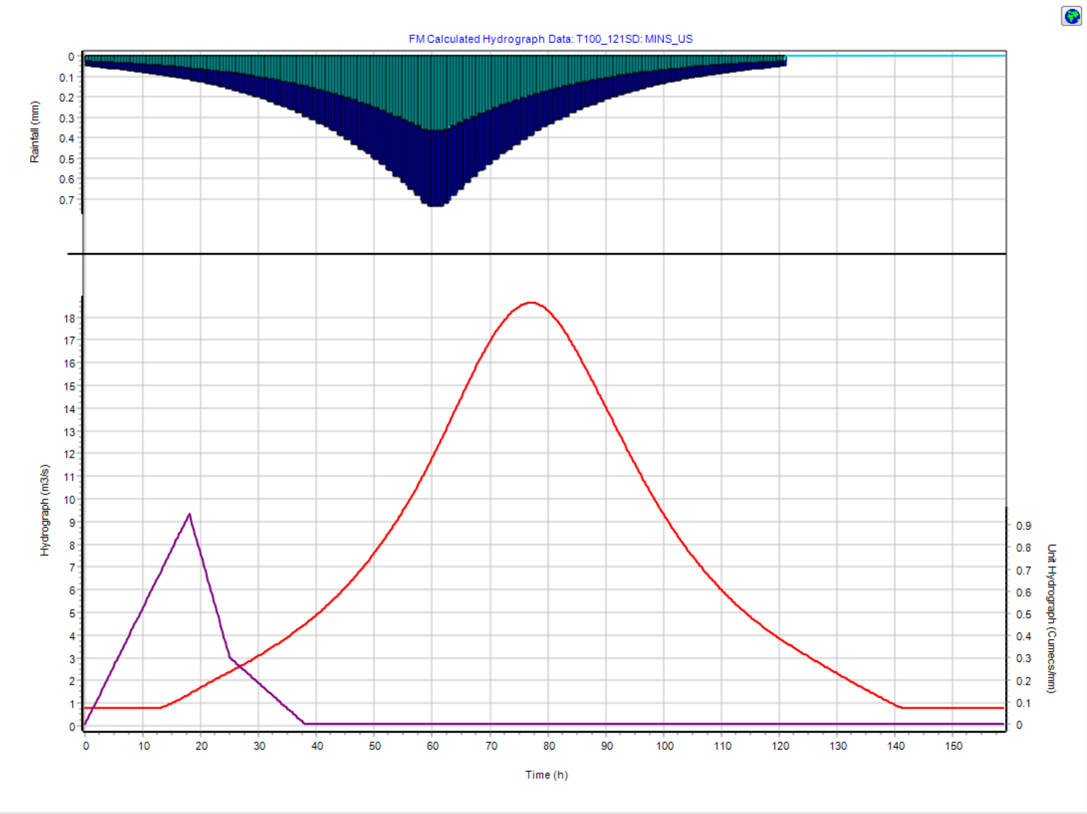


Figure A-6.6. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 121-hour storm duration

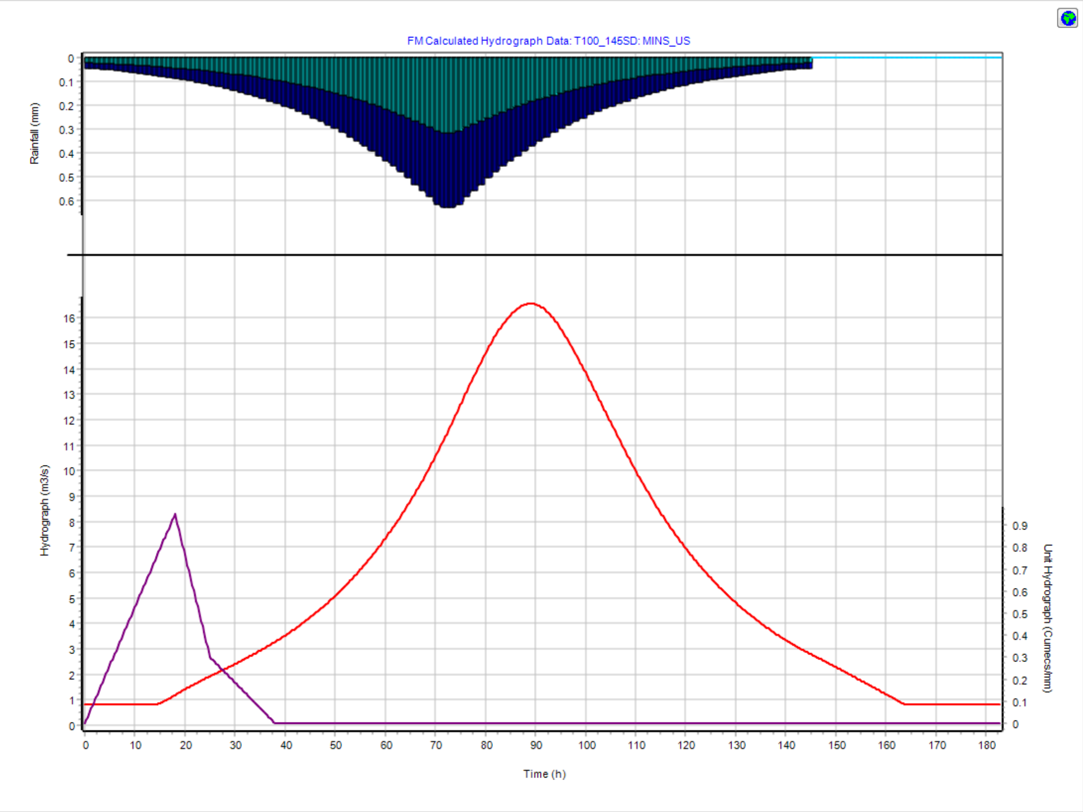


Figure A-6.7. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 145-hour storm duration

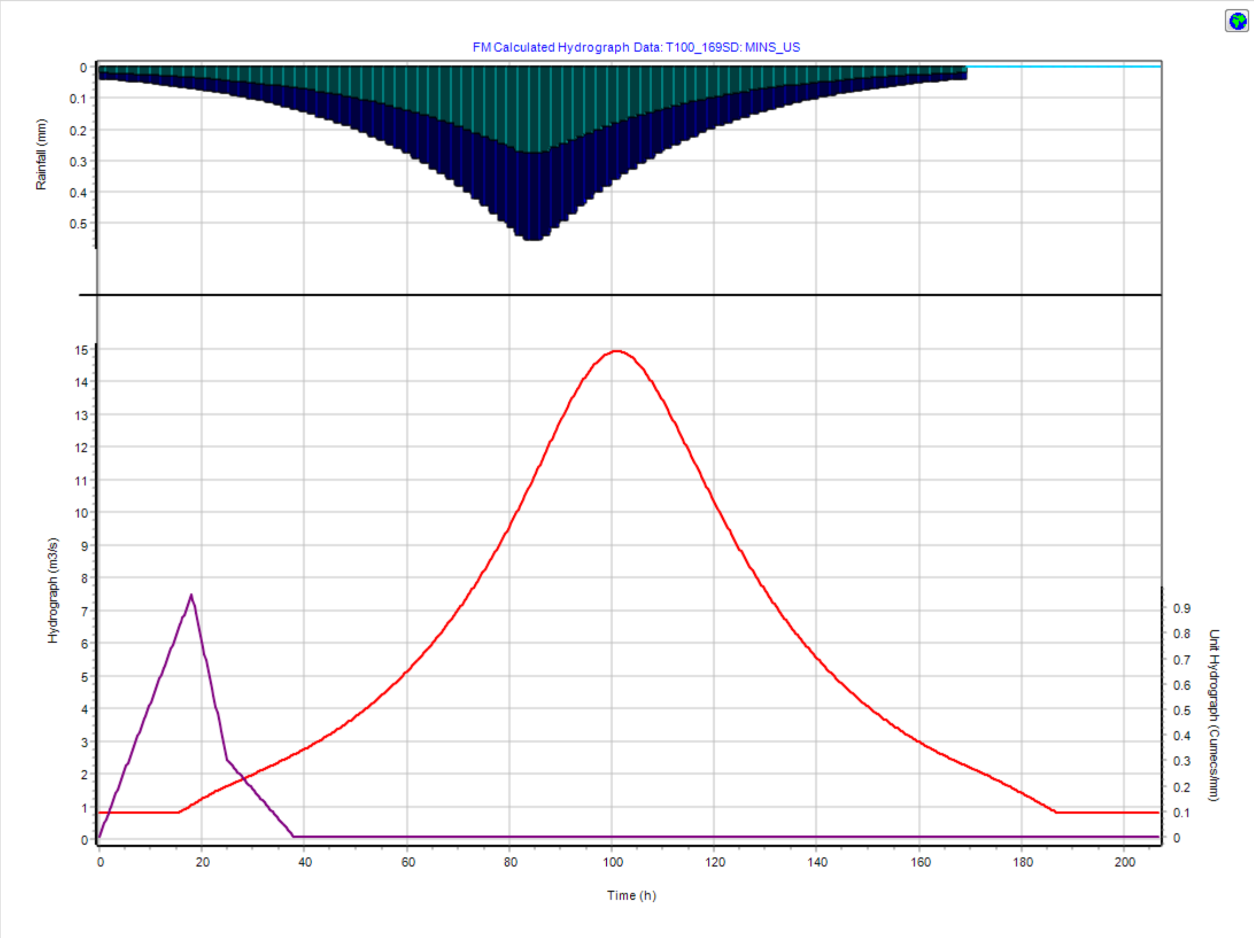


Figure A-6.8. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 169-hour storm duration

Appendix E

Derived Sub-Catchment Hydrographs – All Return Periods

Figure B-1 – Figure B-7 present hydrographs derived for the main inflow sub-catchment, i.e. MINS_US for all considered return period events with 121-hour storm durations.

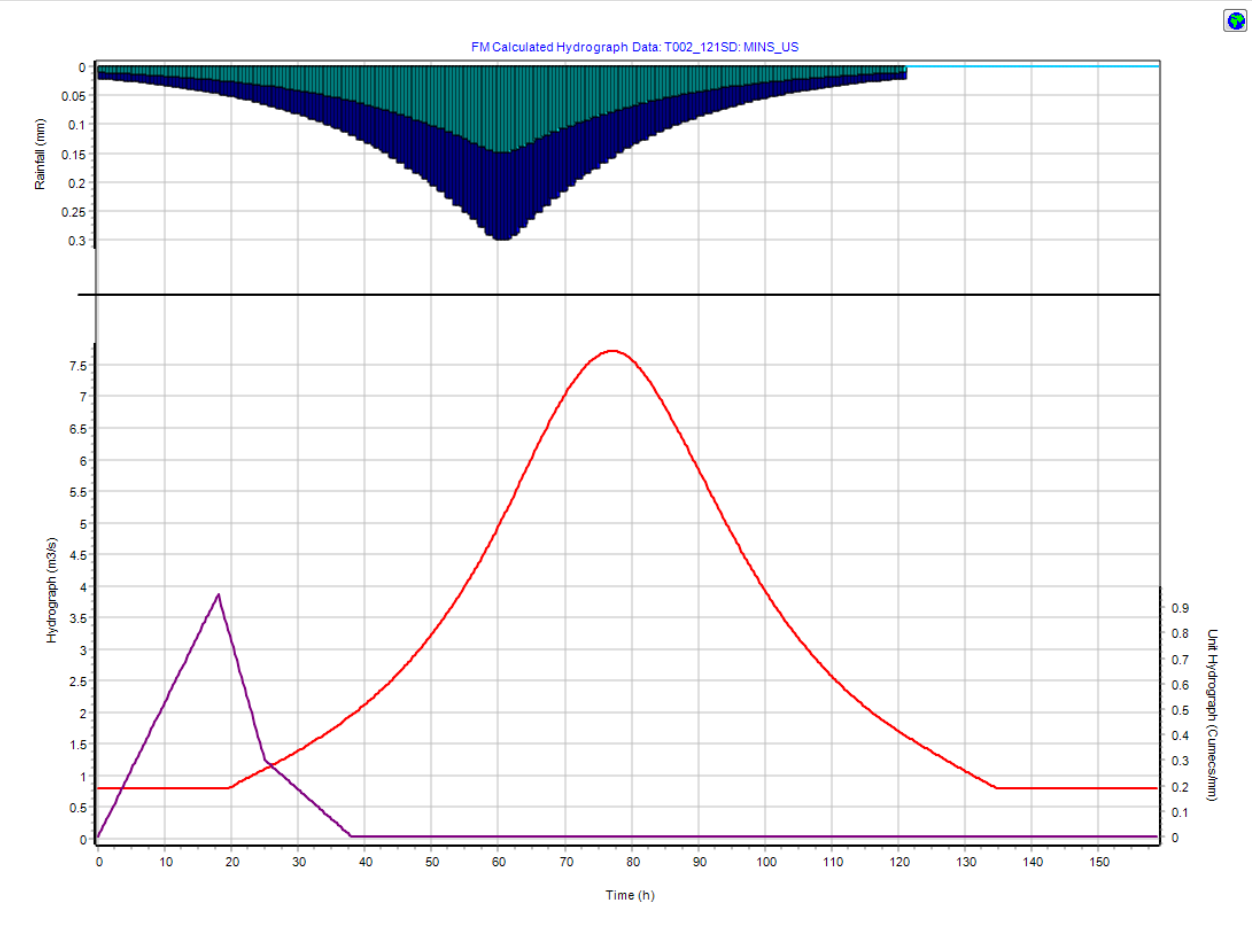


Figure B-1. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 2y return period event with 121-hour storm duration

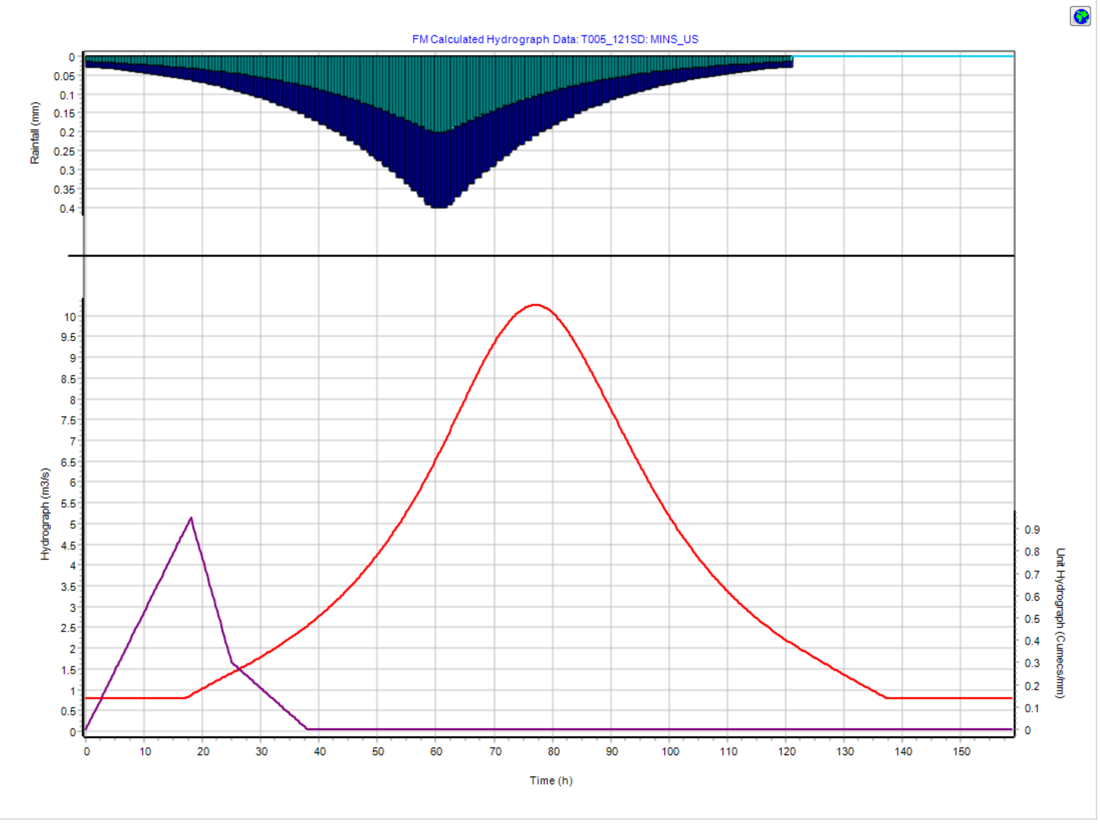


Figure B-2. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 5y return period event with 121-hour storm duration

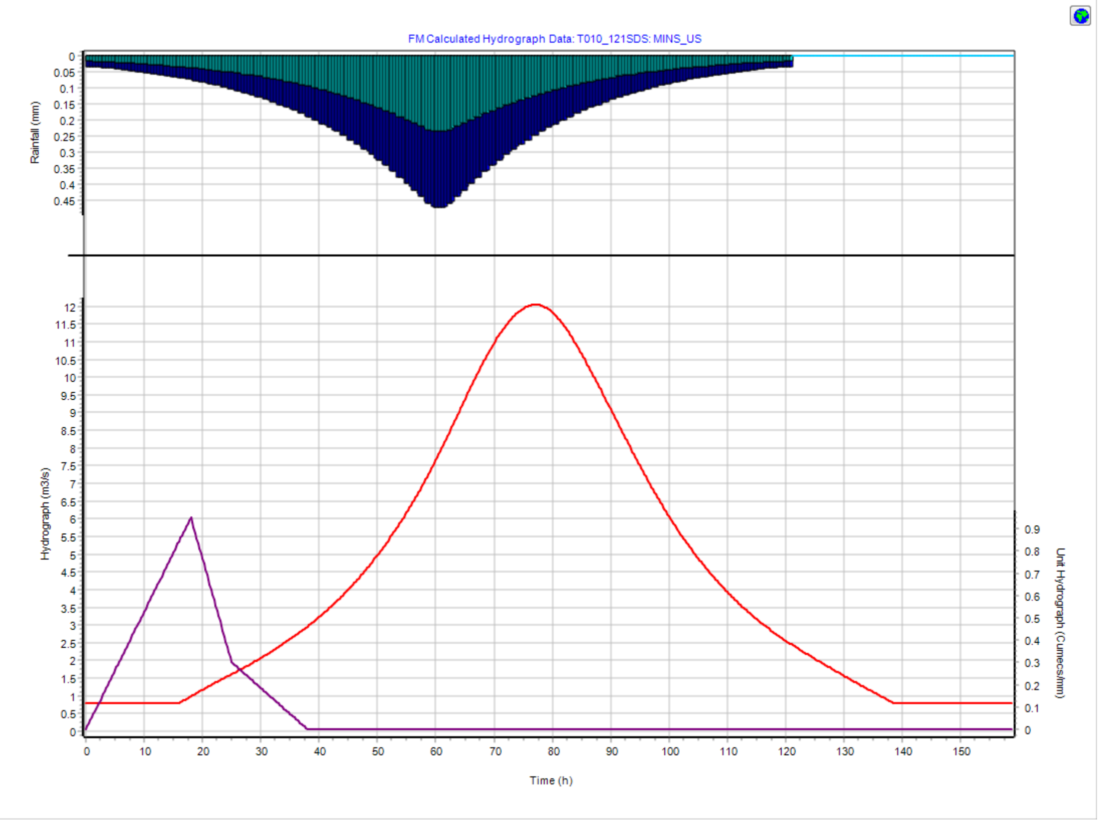


Figure B-3. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 10y return period event with 121-hour storm duration

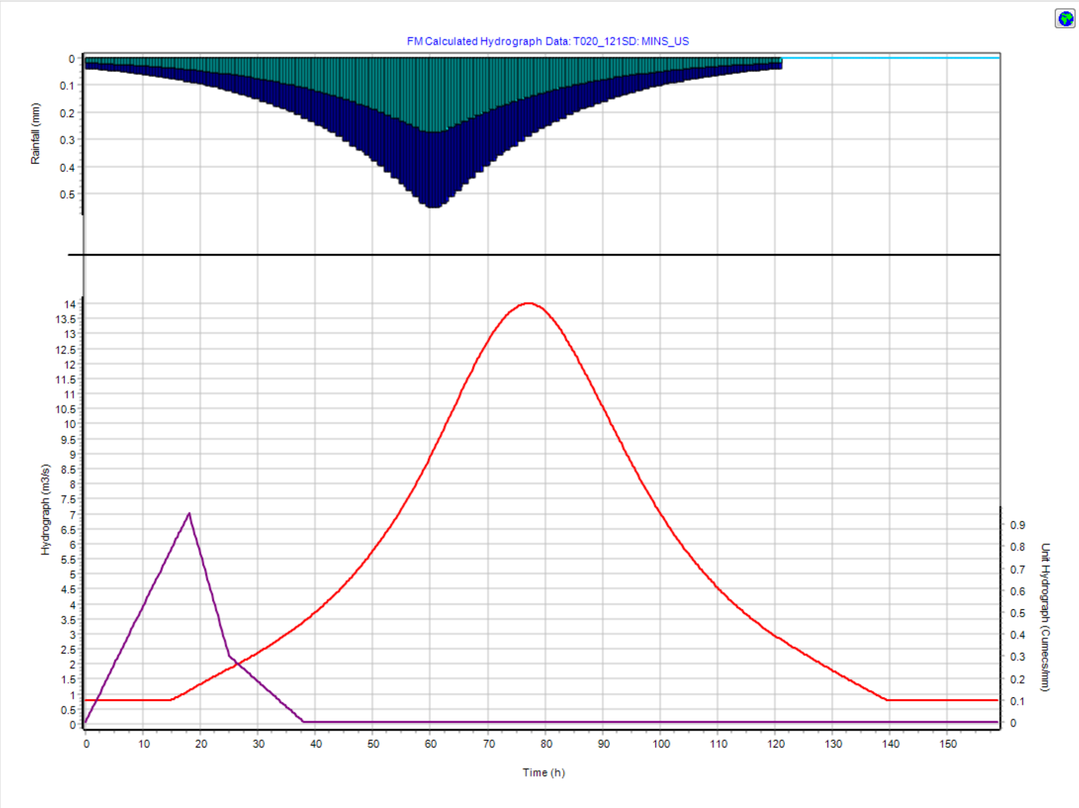


Figure B-4. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 20y return period event with 121-hour storm duration

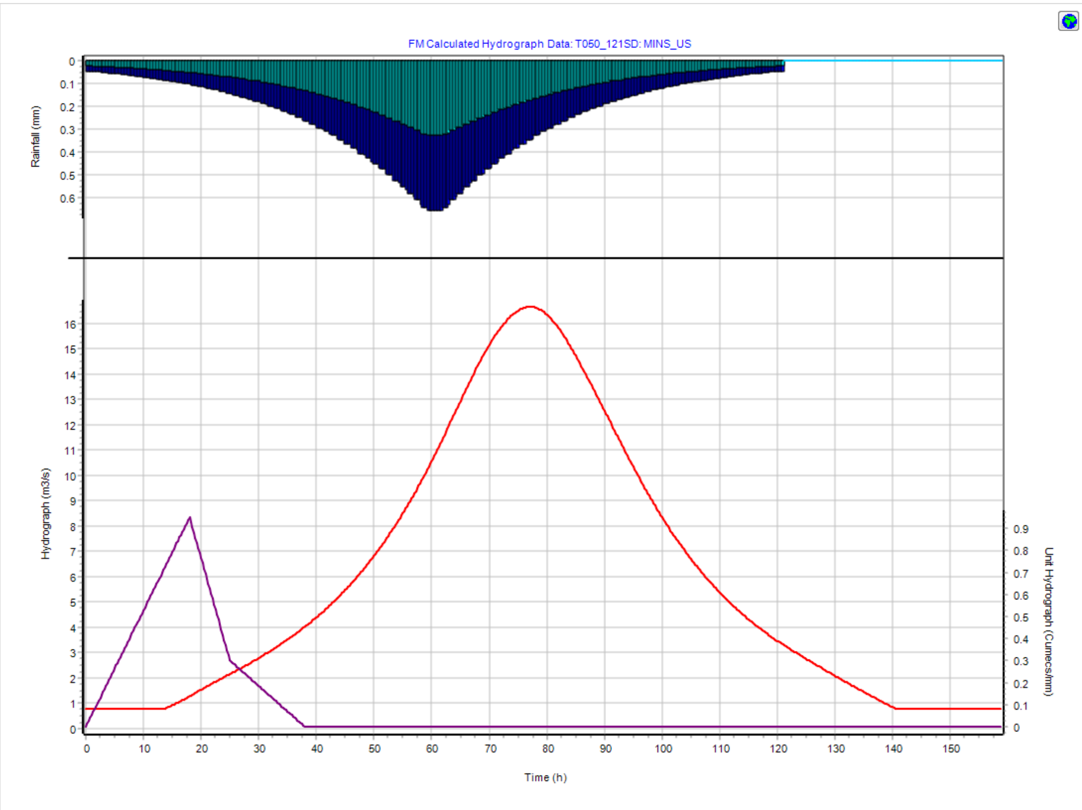


Figure B-5. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 50y return period event with 121-hour storm duration

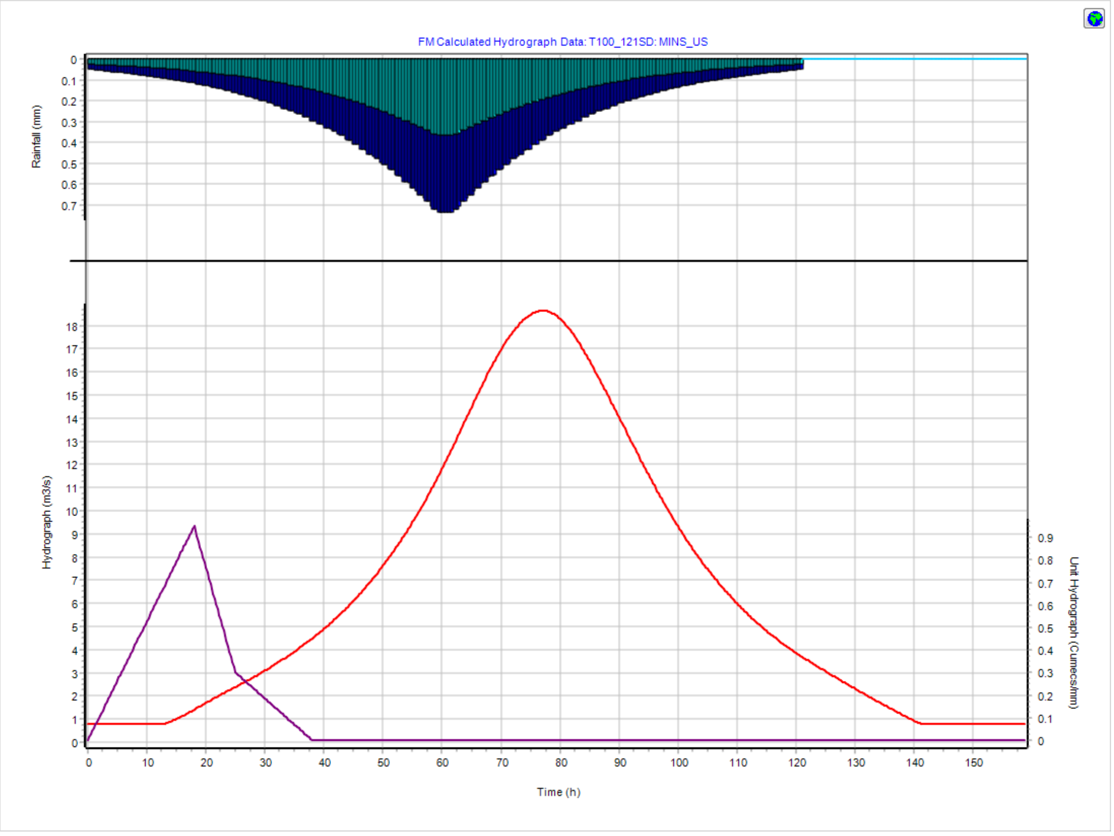


Figure B-6. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 100y return period event with 121-hour storm duration

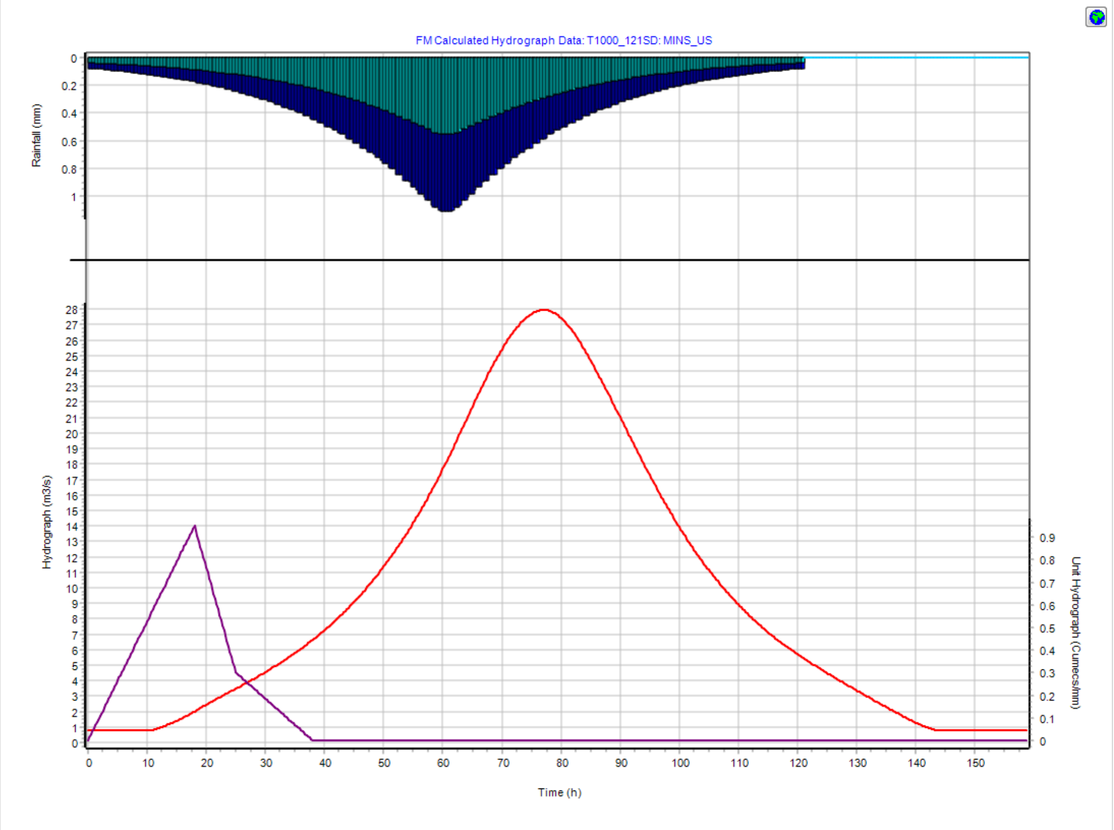


Figure B-7. Derived Hydrograph for 'Upland' area inflow at MINS_US for 1 in 1000y return period event with 121-hour storm duration

APPENDIX 4 - BREACH MODELLING REPORT

CONTENTS

TABLES

PLATES

1 INTRODUCTION

1.1 Overview

1.1.1. This report describes the tidal breach and coastal inundation models of the Sizewell and Minsmere frontages developed for the Sizewell C Project. It includes updates to the model and reporting based on comments received from the Environment Agency following their review of model submission in March 2019.

1.1.2. The report summarises results from model runs carried out to date to assess coastal flood risk to the proposed Sizewell C development and potential impacts of the development to any off-site receptors as a result of tidal breach and inundation of coastal defences.

1.1.3. Outcomes of the modelling study were used to inform the Sizewell C Flood Risk Assessment (FRA) that would be included in the Environmental Impact Assessment (EIA) to be submitted as part of the Development Consent Order (DCO) application for a new nuclear build development.

1.1.4. The report is intended to update EDF Energy and the Environment Agency on the latest model developments, results and assumptions adopted in the breach and coastal inundation modelling studies undertaken to inform the Sizewell C FRA.

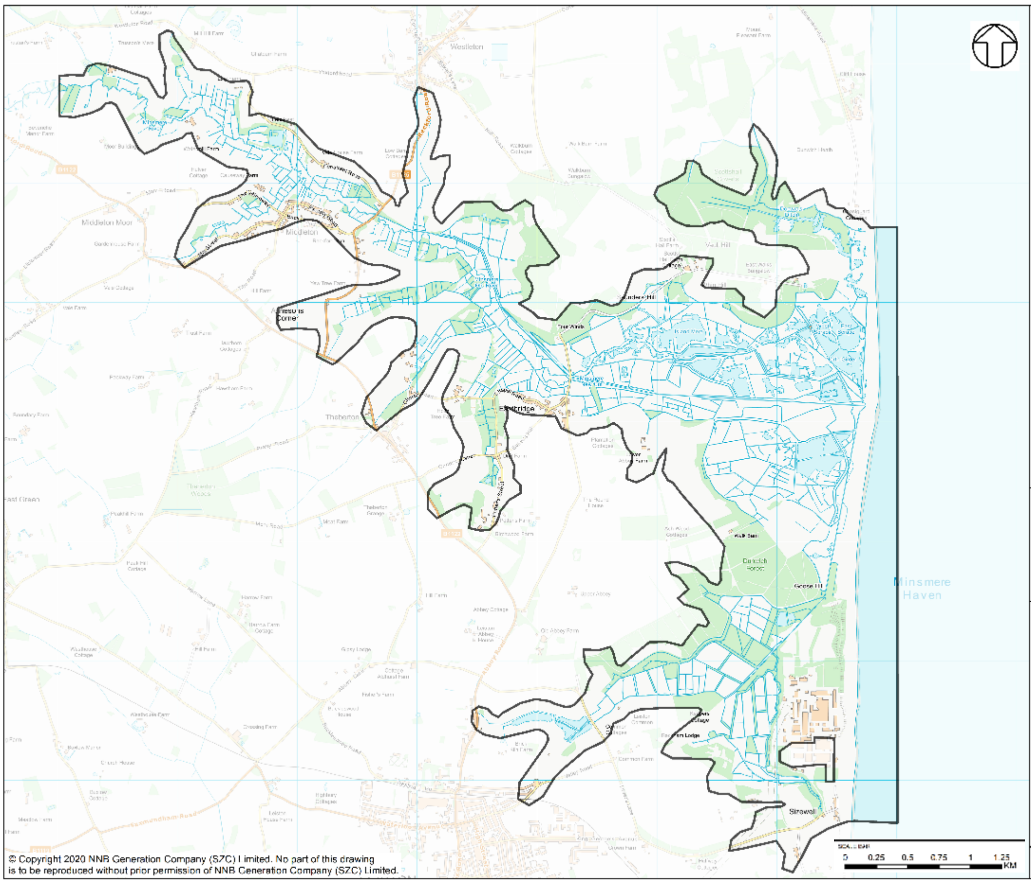
1.2 Model development

1.2.1. To inform the FRA for the proposed development at Sizewell C, a hydraulic model has been developed using industry standard hydraulic modelling software TUFLOW.

1.2.2. The preliminary breach assessment was carried out in 2015, where a 2D TUFLOW model was built (Ref 1), however due to limited information at the time the model was relatively coarse. It was therefore decided that for the purpose of this study more detailed 2D (TUFLOW) model domain developed for the 1D/2D fluvial model (Ref 2) would be used.

1.2.3. The 2D domain was optimised to ensure correct representation of key catchment and development features. Improvements to the model were also made following Environment Agency model review that is discussed in **section 3.1c**. **Plate 1.1** presents the extent of the adopted 2D model domain.

Plate 1.1. 2D TUFLOW model domain



1.3 Purpose of the model

1.3.1. To inform the Sizewell C FRA assessment of coastal flood risk was required. For that purpose, two analyses were carried out to determine (i) flood risk from the tidal breach of sea defences; and (ii) risk due to overtopping of the sea defences and inundation of the Minsmere Levels and Sizewell Belts areas behind the defences.

1.3.2. The assessment was carried out to provide an understanding of flood risk to the development site itself, as well as potential changes in flood risk to the off-site receptors.

1.3.3. As required for the FRA, the 1 in 200-year and 1,000-year return period events were considered, each with an allowance for climate change, for both the baseline and ''with scheme'' model schematisations. Lower return period events were not assessed as the risk of breach occurring would be very low and the extreme still water levels would be significantly below the crest of existing coastal defences along the frontage, posing no risk of inundation. The impact of breach on the development site and the off-site receptors would therefore be negligible.

1.3.4. Flood risk would be assessed at different phases of the development's lifespan, as further presented in this note. The key points in time for the Sizewell C development considered in this assessment are:

- **2030** – End of substantial construction of key infrastructure, used for assessment of construction phase flood risk (it is acknowledged that construction activities may continue to 2034);
- **2140** – Interim spent fuel store decommissioned, used for assessment to risks on site from breach of main hard defences (HCDF); and
- **2190** – Theoretical maximum site, used for assessment of impacts and/or changes in flood risk-off site up to the end the end of development's lifetime.

1.3.5. To assess the breach and coastal inundation modelling, wave overtopping assessment was required. This was carried out following the approach adopted for the modelling of the main Sizewell C defence (HCDF) overtopping modelling completed as a part of the FRA study. Details on the methodology and inputs to the overtopping modelling are discussed in a separate Coastal modelling report (Ref 3).

2 MODEL SCHEMATISATION

2.1 Baseline model build

2.1.1. Baseline model schematisation was developed based on the 2D domain from the 1D/2D fluvial model with key river channels and drains from the linked model represented using elevation break-lines to lower or raise ground levels in the 2D domain, as appropriate.

2.1.2. River and drain networks (i.e. Minsmere Old River and Minsmere New Cut, Scotts Hall Drain, Leiston Drain, Sizewell Drain and Drain #7) were represented as 'THICK' break-lines to ensure a continuous flow path through the grid cells. 'THIN' break-lines were used to raise elevations for banks or road networks as illustrated in **Plate 2.1**. This approach ensures water does not flow prematurely out of bank (over the break-lines) and that storage volume within the cells along the banks is not overestimated.

2.1.3. Elevations of the banks were extracted from latest available 1m resolution LiDAR Digital Terrain Model (DTM) data obtained on 18 March 2019 from DEFRA data services platform (https://environment.data.gov.uk). The data tile covering the whole model area (LIDAR-DTM-1M-TM46) was used that has a date stamp of 12 July 2018. The bank levels along the break lines were read directly from the composite lidar layer.

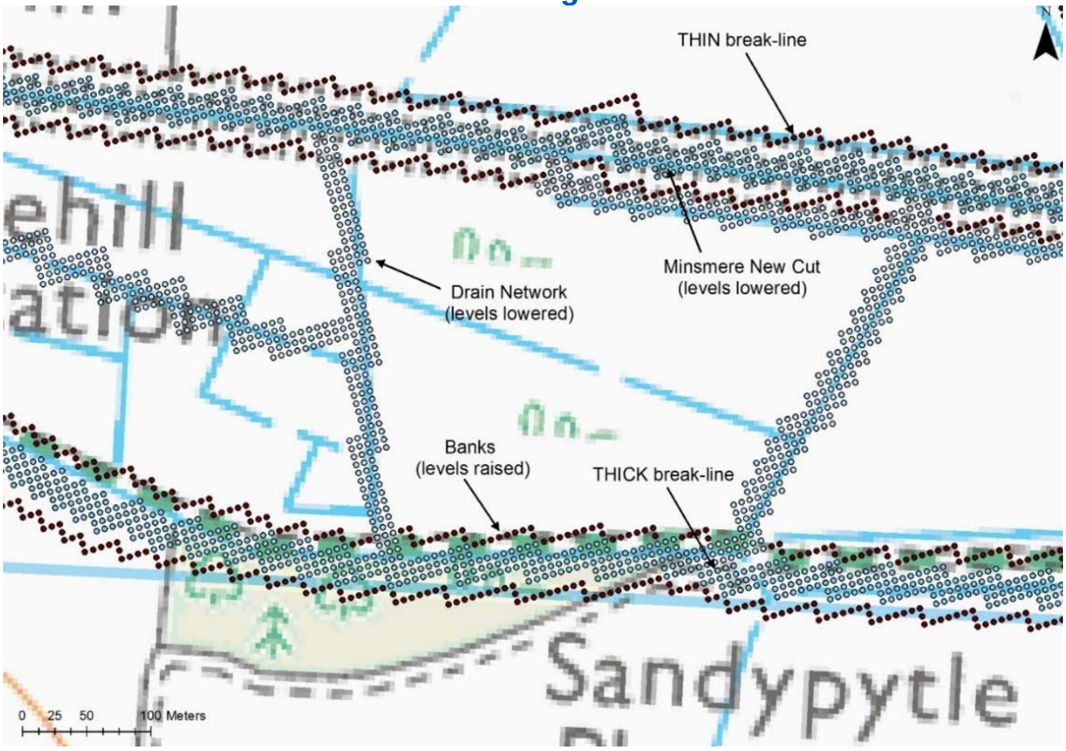
2.1.4. Key flow constriction features such as culverts and bridges, including the Minsmere Sluice (discussed further in **section 2.1b**) were represented in the TUFLOW model using 1D ESTRY units to account for flow restrictions and structure losses.

2.1.5. The model boundaries were defined by the North Sea to the east and Minsmere and Leiston catchment extents to the west. Further details on boundary conditions are provided in **section 4**.

2.1.6. 2D initial water levels were set to a level of 0.5m AOD that was applied to the entire model domain. This resulted in the lowest points in the domain as well as the river channels and drains being wet at the start of the simulation, giving a more conservative approach where some of the storage available within the floodplains was already occupied.

2.1.7. Following the approach from the 1D/2D fluvial model, the Aldhurst Farm developed as a part of the habitat compensation scheme was included in the baseline model, as the scheme has already been implemented and currently contributes to flood storage in the upstream part of the Leiston Drain (Leiston Ditch) catchment. Further details on implementation of the Aldhurst Farm are provided in the following **section 2.1a**.

Plate 2.1. Model schematisation using break-lines



a) Aldhurst Farm

2.1.8. The Aldhurst Farm Habitat Creation Scheme has been implemented to create lowland ditches and a mosaic open water habitat (‘wetland habitat’) to compensate for future loss of SSSI required for construction of the main development platform and causeway.

2.1.9. The total Aldhurst Farm site comprises area of 76ha, with wetland habitat occupying approximately 6.3ha of low-lying land alongside two existing watercourses, namely the Aldhurst Valley Stream and the Leiston Ditch receiving treated effluent from Leiston Waste Water Treatment Works.

2.1.10. The scheme has been incorporated into the hydraulic model in both the ‘baseline’ and ‘‘with scheme’’ scenarios. **Plate 2.2** illustrates the extent of Aldhurst Farm in the model schematisation.

2.1.11. The basins where included in the 2D TUFLOW model domain by lowering the ground and to the initial water levels as per a topographic survey of the scheme site (drawing ref. ‘5m grid topo survey.dwg’ provided by EDF on 25th May 2018). This approach resulted in lower storage capacity of the basins and therefore a more conservative approach.

2.1.12. In addition, the survey data was used to define ground enforcing elevations in the topography layer of the model. The survey is presented in **Plate 2.3**. No change in roughness to the grounds at the Aldhurst Farm was assumed.

2.1.13. To ensure that each basin was connected to the adjacent drains, ground levels of the banks and the roads were lowered at appropriate locations, see **Plate 2.2** ‘drain connections’ lines. This represents an intermediate level of detail given that Aldhurst Farm is a considerable distance from the coastal boundary, but still allows the model to represent its storage effect.

2.1.14. Further details on the Scheme and environmental screening are provided in the Aldhurst Farm Scheme EIA Screening Report (Ref 4).

Plate 2.2. Representation of the Aldhurst Farm Habitat Creation Scheme within the 2D model

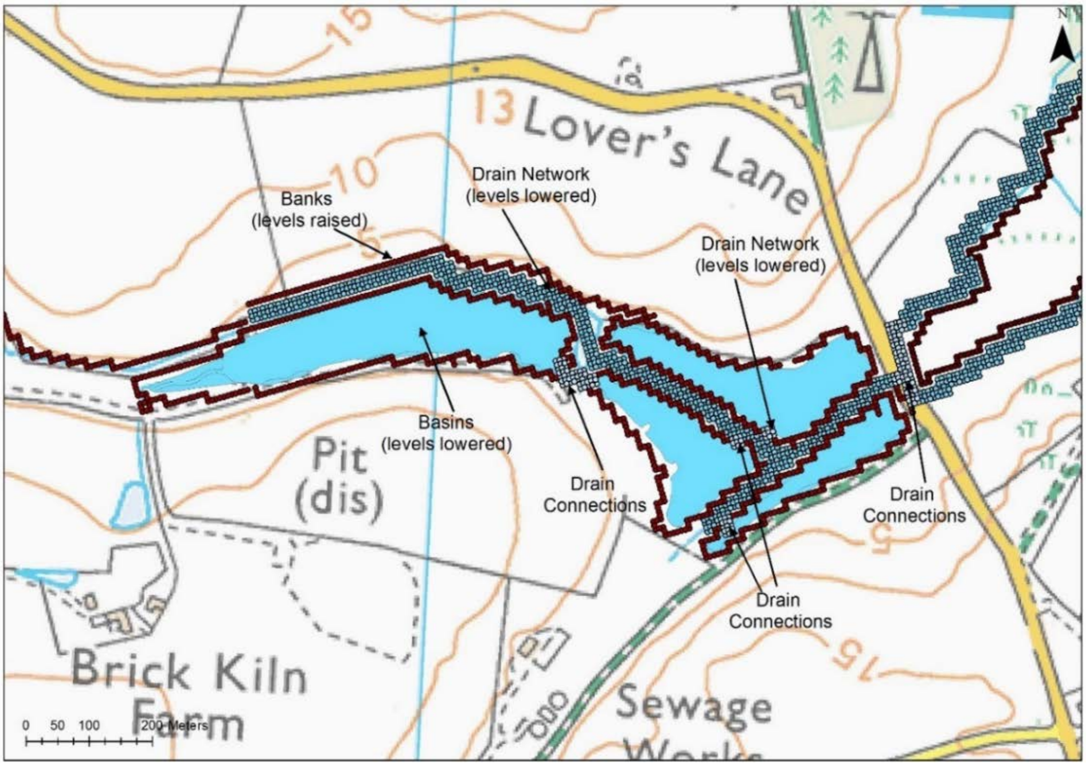


Plate 2.3. Extract from Aldhurst Farm – 5m grid topographic survey drawing

b) Minsmere sluice

2.1.15. The Minsmere sluice in the fluvial model was represented as a 1D structure within the Flood Modeller, comprising the four inlet structures, flap gates, chamber and two outfall pipes giving a high level of detail.

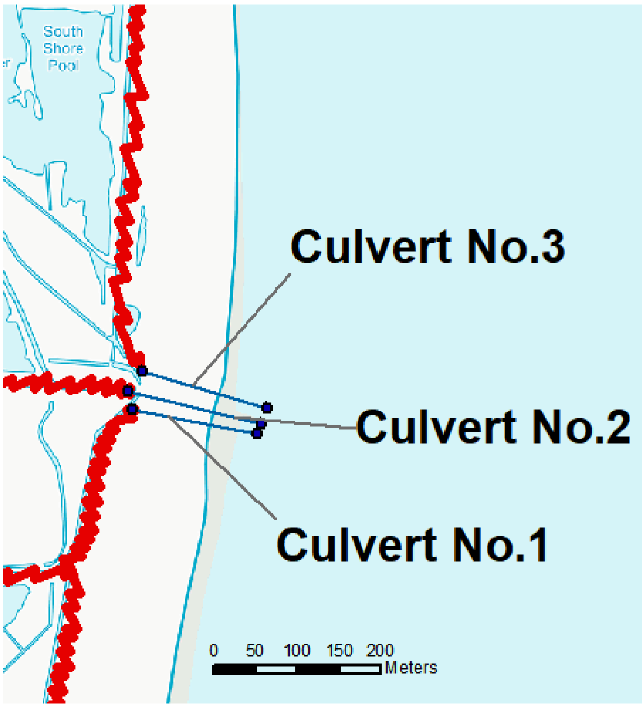
2.1.16. Analysis of the preliminary fluvial model results confirmed that the inlet losses are dominant where considering impact of the structure on water levels in the upstream sections of all four watercourses. Based on this, a simplified, more stable representation of the combined structure was adopted in the breach model, that could simulate rapid changes in water levels in the system associated with the start time of breach.

2.1.17. The structure was therefore represented as three ESTRY culverts, each with a flap valve, as shown in **Plate 2.4**.

2.1.18. Culvert No.1 (Leiston Drain) has a length of 151m, a width of 0.70m and a height of 1.00m. Culvert No.2 (Minsmere New Cut) has a length of 164m, a width of 1.28m and a height of 1.29m. Culvert No.3 (Scotts Hall Drain) has a length of 155m, a width of 0.49m and a height of 1.20m.

2.1.19. For all three culverts, inlet and exit loss coefficients of 0.5 and 1.0 respectively were applied. As the Minsmere South penstock in the fluvial model was assumed to be closed for model scenarios, it was not included in the breach model and therefore only three inlet culverts were considered.

Plate 2.4. Minsmere Sluice representation



2.2 'With scheme' model build

a) Main development site

2.2.1. The proposed Sizewell C development requires raising of the ground to create a level platform for construction and operation of the nuclear power station and other associated infrastructure. Part of the raised areas are located within current floodplain. It was therefore required within the FRA to assess flood risk to the site itself as well as potential impact of the development on flood risk to the off-site receptors.

2.2.2. Further to the main platform, the SZC development comprises other developments required for infrastructure and facilities. **Figure 3A.1** in the Sizewell C Description of the Development illustrates an indicative construction site layout plan.

2.2.3. Some of the development areas would only be present during the construction phase (considered up to 2030 when all key infrastructure would substantially be in place) and others would form the permanent development with theoretical maximum site lifetime of up to 2190.

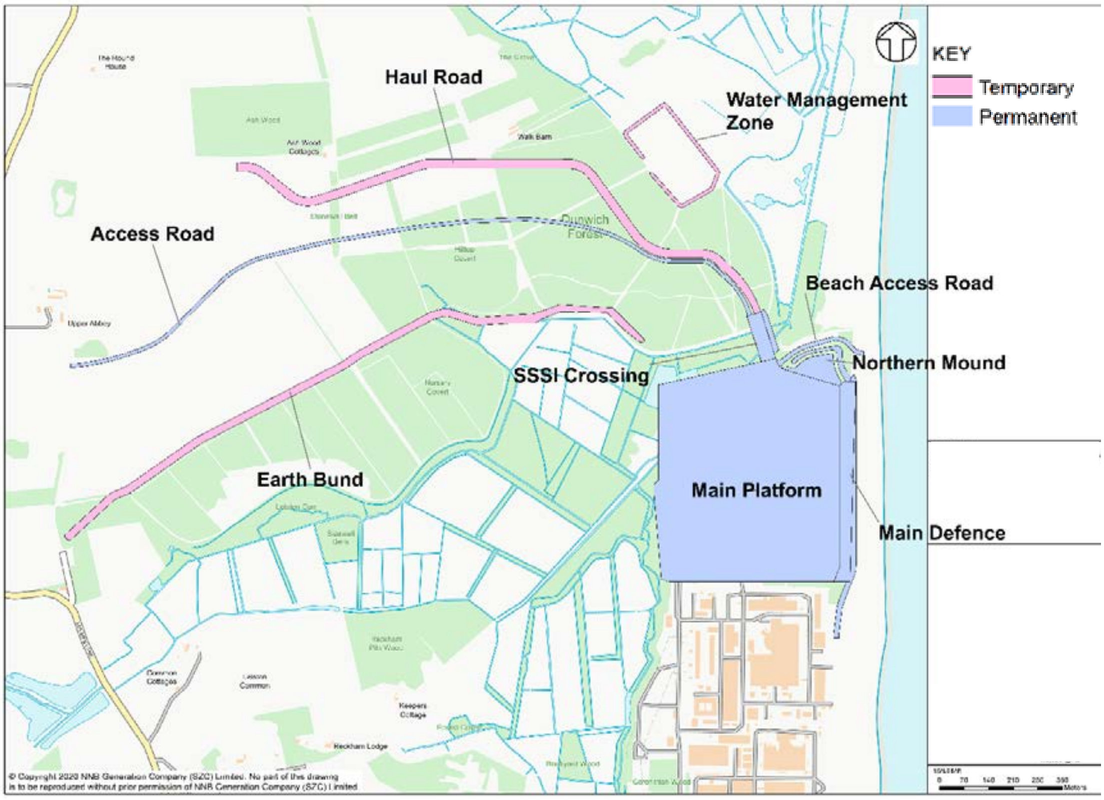
2.2.4. For assessment of flood risk during the construction phase (2030 epoch) the Sizewell C development components included in the model are the main platform, the SSSI crossing and access road with temporary haul road, northern mound (with access road to beach landing facilities), main sea defence (HCDF), earth bund (acoustic bund - part of the temporary earthworks in the construction area), part of the drainage infrastructure, namely water management zone 1 (WMZ1) and the realigned Sizewell Drain.

2.2.5. Only permanent features were incorporated in the model to assess the risks on-site and impacts or changes in flood risk off-site through operation (up to 2090) and decommissioning phases (up to 2140 and up to 2190) of the Sizewell C project. The model schematisation for operation and subsequent phases includes the permanent features of the main platform, the SSSI crossing, northern mound, main sea defence (HCDF), access road and haul road embankment.

2.2.6. Although some of the permanent development components would be removed in the early stages of the decommissioning phase. A conservative approach was adopted for the modelling that included all permanent components up to the theoretical maximum site lifetime (2190). **Plate 2.5** illustrates features of the development included within the model schematisation, differentiating between the permanent features and those temporary features which won't remain beyond construction phase. Realignment of the Sizewell Drain is shown in **Plate 2.6**.

2.2.7. Since the modelling was carried out for the 2030 epoch when the key infrastructure discussed in **paragraph 2.2.4** would be constructed, the road/platform/defence levels were assumed as completed with crest of 7.3m AOD for the platform and the SSSI crossing and 10.2m AOD for the coastal defence.

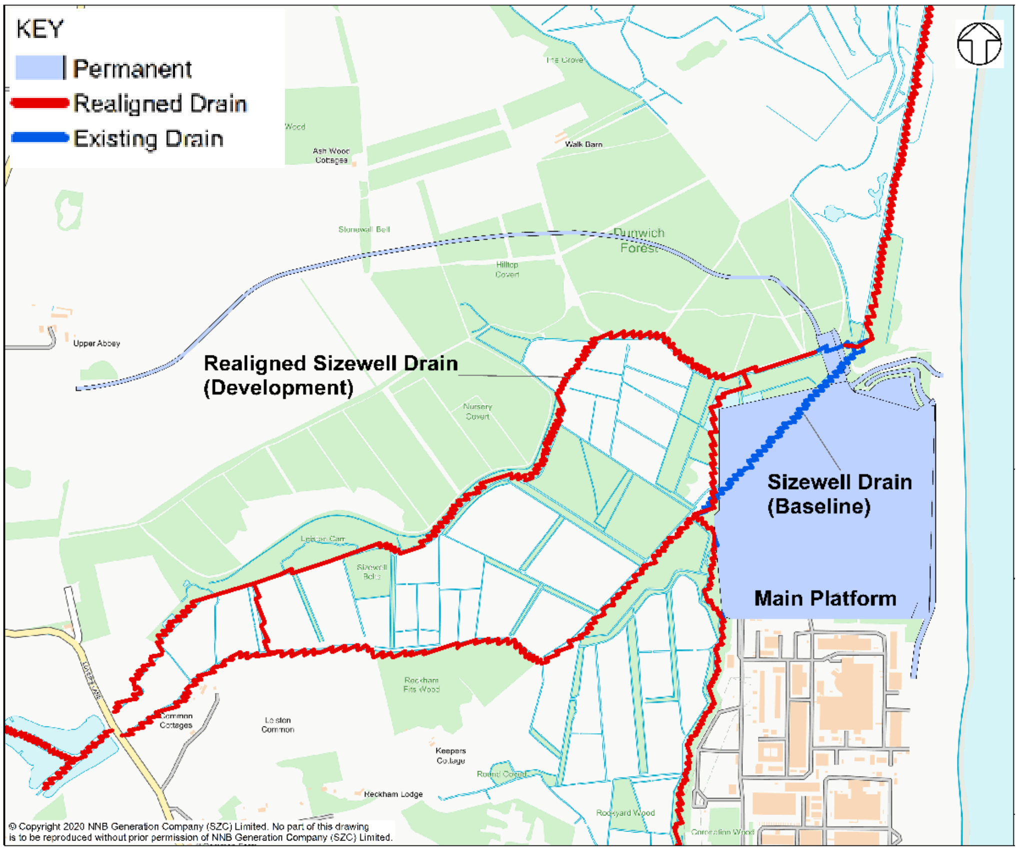
Plate 2.5. Development features included in the ''with scheme'' model configuration



2.2.8. Other features of the development were not included at this stage of the modelling work as they were not anticipated to have significant impact on flood risk or are located outside of flood zone 2 and 3. These include various car parks, proposed minor floorplan changes, and other unconfirmed or transitory features such as site offices.

2.2.9. The alignment and height of the main platform and the earth bund were taken from drawing no. SZC-SZ0100-XX-000-DRW-100000 (provided by EDF in July 2018).

Plate 2.6. Realignment of Sizewell Drain



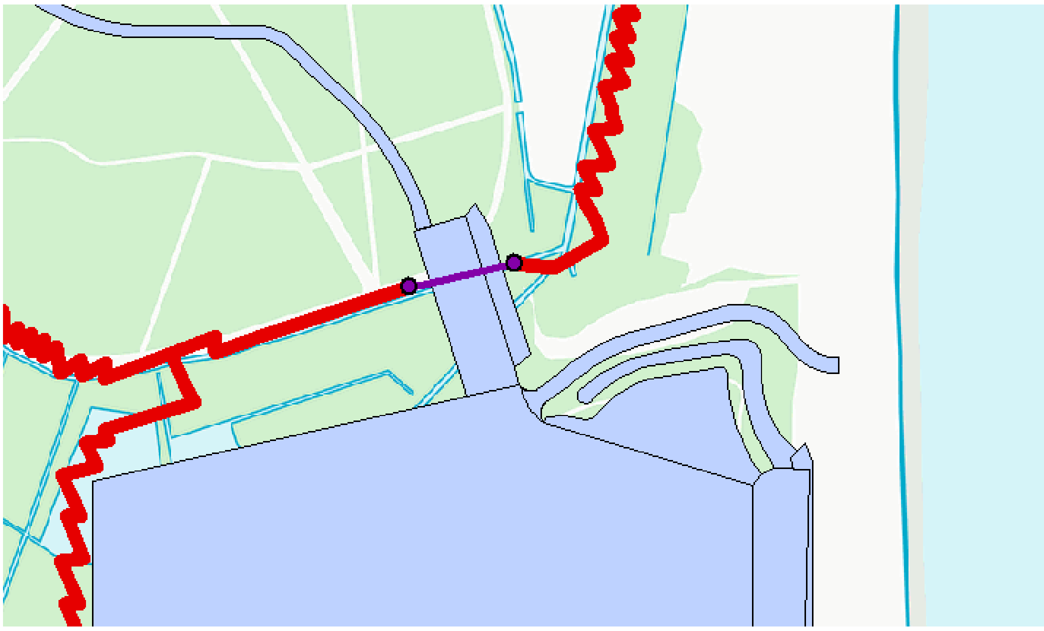
b) SSSI crossing

2.2.10. The SSSI crossing was represented using 1D (ESTRY) elements within the TUFLOW model (**Plate 2.7**). A rectangular ESTRY culvert, 8m wide and 4.5m high, was adopted with an upstream and downstream invert level of 0.973m AOD.

2.2.11. The dimensions of the culvert under the crossing as well as the soffit and invert levels, road width and elevation were taken from set of drawings provided by EDF in June 2018 (no. SZC-SZC008-XX-000-DRW-100000, 100001 and 100002).

2.2.12. For model stability reasons the culvert under the SSSI crossing has been extended by 8m from the 78m length depicted in the relevant drawings (assumed length is from base of embankment to base of embankment). This is to ensure appropriate connection of the culvert on each side of the Leiston drain channel.

Plate 2.7. SSSI crossing model representation



2.3 Model improvements

2.3.1. Following the comments received from the Environment Agency following model review (June 2019), the model schematisation was further refined. Additional minor model updates were carried out to improve model stability and to address outstanding concerns, including:

- Re-orientation of the grid to north-south (aligned with the coastline);
- Reduction of grid size from 8m to 4m (and subsequent timestep reduction);
- Realignment of the Leiston fluvial inflow point location to better position within the channel;
- Changing start of the simulation to include more of astronomical tide cycle before the peak surge event occurs;
- Improvement on stamping and trimming the bathymetry grid to ensure more accurate elevations were read;
- Moving the realigned Sizewell Drain and the earth bund to ensure that flow paths were continued;
- Improvement of file naming system for clarity;
- Adding extra PO lines to enable better inspection of results; and
- Improving alignment of temporary and permanent features as well as including other features (beach access road/WMZ/haul road, etc.).

2.3.2. Model schematisation discussed in section 2.1 and section 2.2 were used both in the tidal breach and coastal inundation assessment. The only difference was addition of the breach in the defences and associated changes to overtopping volumes (i.e. no overtopping assumed through the breach itself).

3 BREACH CONFIGURATION

3.1 Breach location

3.1.1. Preliminary assessments indicated that the greatest relative impacts of the scheme on flood levels in the Minsmere Levels and Sizewell Belts are predicted if a breach occurs at the Tank Traps, just north of the proposed Sizewell C power station platform.

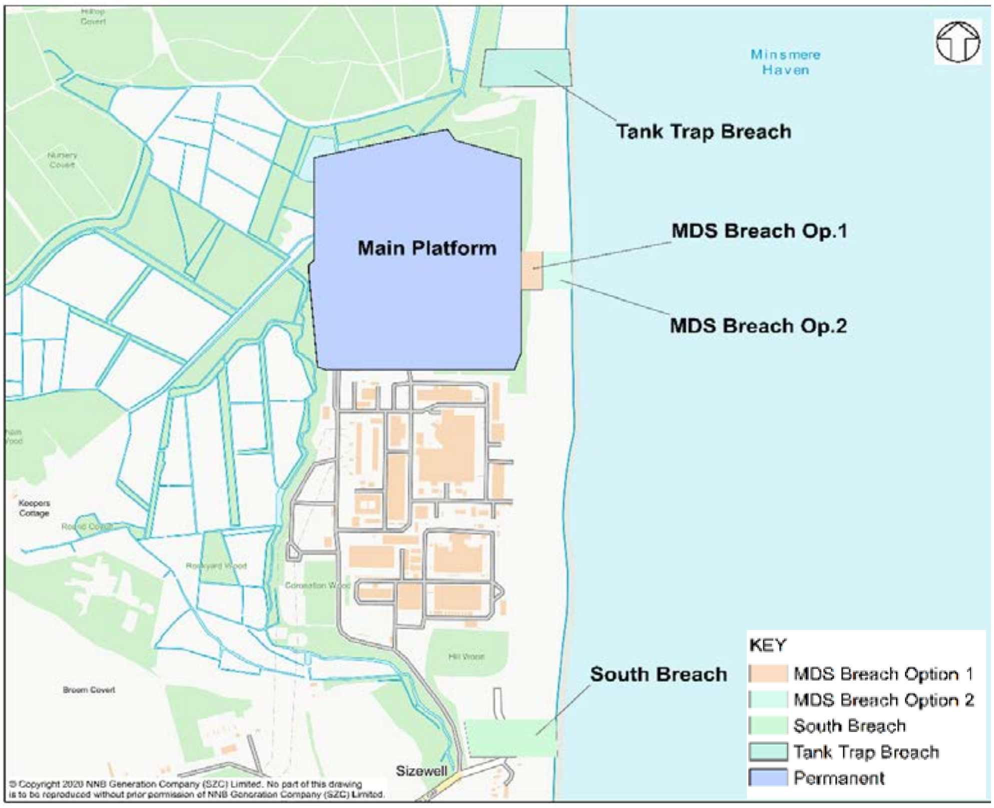
3.1.2. This is due to the SSSI crossing and the main platform restricting flow, meaning that if a breach occurs on the southern side of the development, water is held back within Sizewell Belts, whereas if breach occurs north of the development, water is more constricted within Minsmere Levels. Most receptors are located close to Eastbridge. This location (Tank Traps) is therefore the focus of the breach assessment, however a comparison with breach south of the development (at Sizewell Gap) has been conducted and results are presented in **section 6.4b)**.

3.1.3. Further information on the analysis and selection of the breach location can be found in the preliminary breach modelling report issued to EDF in November 2015 (Ref 1).

3.1.4. In addition to breach of the existing sand dunes / shingle defences, further model runs were carried out with the breach of the main Sizewell C defence (HCDF). This was assessed with a focus to determine the impact of such scenario on the flood risk to the main platform and other infrastructure of the development.

3.1.5. **Plate 3.1** illustrates the adopted locations and extents of assessed breach scenarios.

Plate 3.1. Adopted breach locations



a) Tank traps

3.1.6. A breach in the existing sand dunes with shingle beach just north of the proposed Sizewell C development (main platform) was modelled at the location known as the 'tank traps'. The crest levels along the sand dunes at this location are low and there is a break in sand dunes on the landward side of the shingle beach. Tank traps was therefore determined to be the area highly vulnerable to a breach during an extreme storm event with potential greatest impacts on water propagation through the breach within Minsmere Levels relative to the Sizewell C development.

3.1.7. The tank traps' breach was set to align with ground levels on the seaward side of the sand dunes whilst levels on the landward side were lowered to 0.8m AOD (broadly in line with typical ground levels behind the defences).

b) South (Sizewell Gap)

3.1.8. Another potential breach was assessed at location of low-lying area within the current sand dunes with shingle beach to the south of the Sizewell A power plant.

3.1.9. This scenario was assessed as a part of sensitivity testing to determine if such breach location on the side of the Sizewell C development could results in greater impact on flood risk to off-site receptors. Therefore, the model was only run for one event, the 1 in 200-year return period at 2190 epoch. This scenario was selected as resulting in the greatest difference in flood levels within the Minsmere Levels and Sizewell Belts as reported in the previously submitted model.

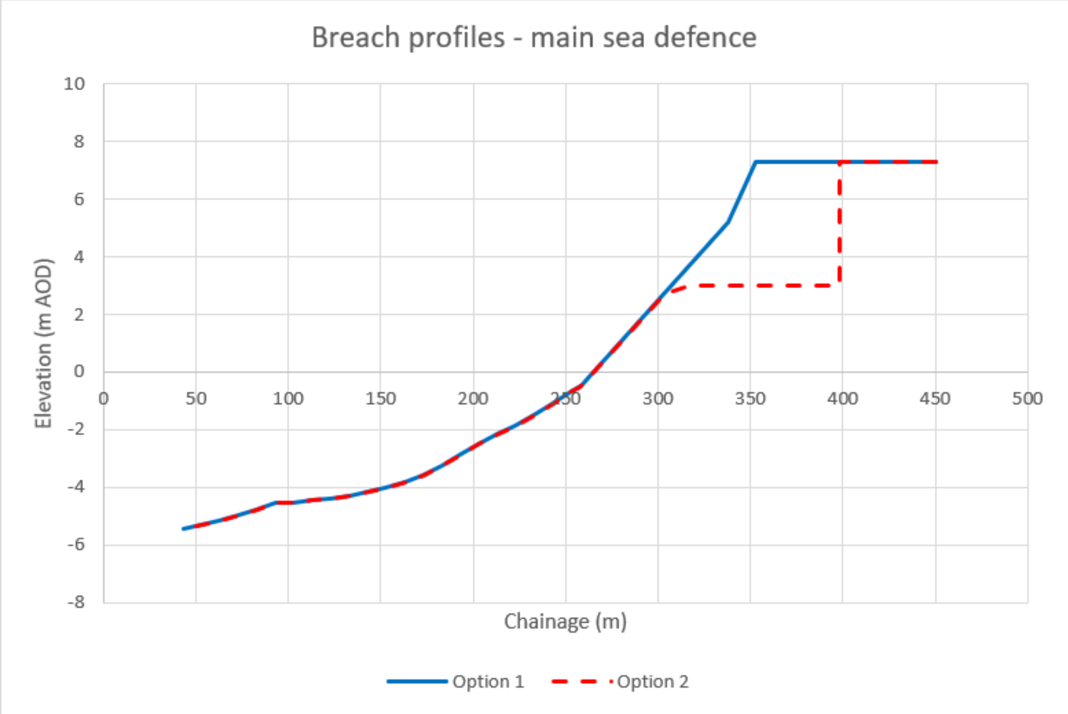
3.1.10. Following the same approach as for the tank traps' breach scenario, the breach levels were set to match the ground levels on both the seaward and landward sides of the coastal defence.

c) Main sea defence (HCDF)

3.1.11. Consideration was also given to the potential breach of the completed main hard coastal defences (HCDF) in front of Sizewell C. Due to the characteristics of the constructed defences and the main platform (including cut-off wall), two possible configurations of the breach were tested, 'option 1' to match the platform level behind the defences (at 7.3m AOD), and 'option 2' with breach level set to the approximate level during construction (3.0m AOD).

3.1.12. It should be noted that in 'option 1' with the defence breached to the level of the platform, the rest of the breach profile was sloping down to align with beach levels on the seaward side of the sand dunes at the shingle beach, whereas in 'option 2' the ground was flattened down to 3.0m AOD but with the platform and the cut-off wall not damaged, therefore creating a vertical wall on the landward side of the breach, as illustrated in **Plate 3.2**.

Plate 3.2. Main sea defence (HCDF) breach profiles



3.1.13. A breach with of 100m was used as in the other breach scenarios. It is acknowledged that this is a conservative approach since typically for hard defences a breach width of 50m is applied, however it is assumed acceptable considering a risk to nuclear site was being assessed.

3.1.14. During early construction stages, ground improvement works would take place on the seaward side of the existing circa 6.0-9.0m AOD secondary defence that extends from the constructed Sizewell B defences to the northern mound. The temporary haul road is then built up in sections to 7.0m AOD, including rock foundations for the new main sea defence (HCDF). No breach scenario has been considered during this short interim construction phase as the risk of breach would be minimal. Instead, overtopping assessment was carried out to determine risk to the site itself, that is discussed in the Coastal Modelling report (Ref 3).

3.2 Breach settings

3.2.1. In each scenario, the breach was set to occur 15 minutes before peak surge level, in line the Environment Agency guidance, and then was left open until the end of the simulation.

3.2.2. Breach width of 100m was used. This was previously discussed and agreed with EDF and is in line with the Environment Agency advice for breach width in open coast for dune type defence.

3.2.3. Each model run was simulated with the following settings:

- 7.2 days simulation;
- Start of the simulation 5.5 hours before the peak water level (to allow propagation of fluvial inflows);
- Breach occurring 15 minutes before peak water level;
- Breach open for 7 consecutive days.

4 BOUNDARY CONDITIONS

4.1 Tidal boundaries

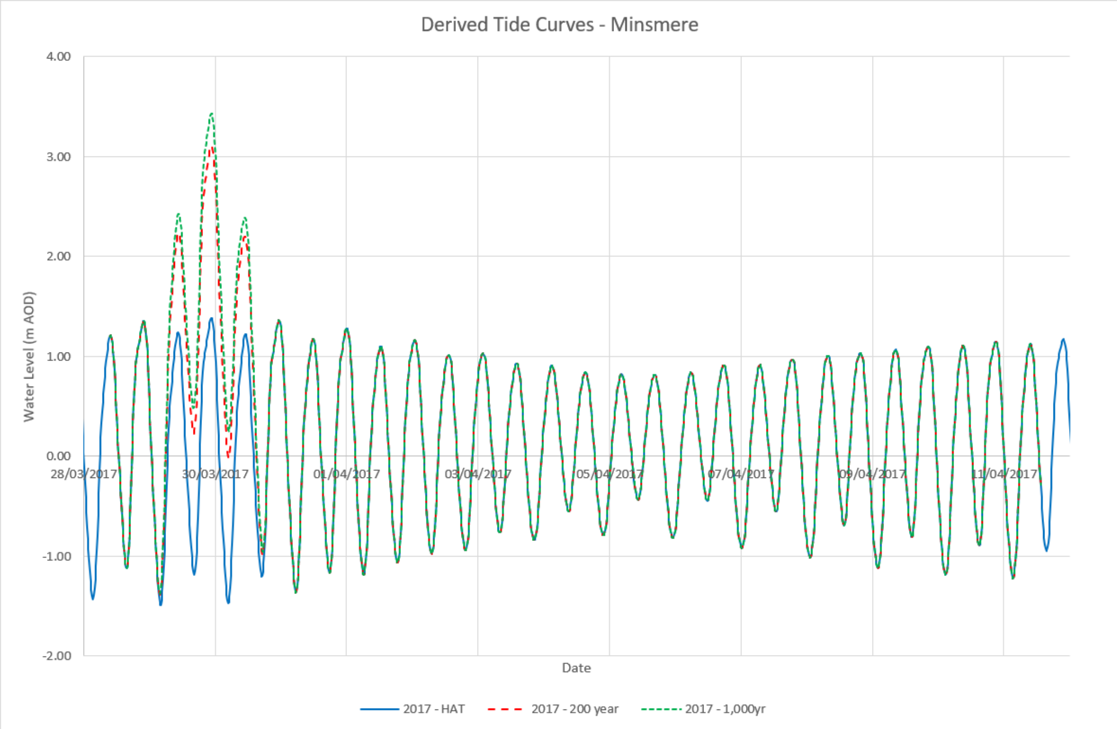
4.1.1. A tide curve was derived for each considered surge event for 1 in 200-year and 1 in 1,000-year return periods to represent the rise and fall of the water levels, as illustrated in **Plate 4.1**. This required several steps as follows:

- Deriving a time series of astronomical tidal elevations based on harmonic constituents for Lowestoft, adopted from Admiralty Tide Tables for 2017 (**Ref 7**) and then transforming to Minsmere using the same approach as described in section 4.3 of the fluvial modelling update report (Ref 2);
- Selecting a surge shape profile. The surge donor profile selected (Lowestoft) was obtained from the Environment Agency Coastal Flood Boundary Conditions Database (CFBD), (Ref 8);
- Selecting extreme water levels for required return period events based on recently updated UK Coastal Flood Boundary Dataset (2018) for point ID 4761 with base year of 2017 (Ref 9);
- Deriving the design tidal curves by scaling the surge shape so that when combined with the tide data, the peak of the water level equalled the required extreme water level for each return period. Peak of the surge was timed so that it coincides with the highest predicted astronomical tide in 2017 (1.31mOD at Lowestoft, 29/03/2017).

4.1.2. In the coastal modelling of overtopping, more conservative extreme water levels provided by Cefas were used to assess very extreme events, whereas for the breach assessment it was assumed that best practice for derivation of extreme water levels (Ref 8) is more appropriate. This was supported by preliminary model results, where impact on flood levels from the ‘worst’ considered scenario (1 in 1,000-year event for 2190 epoch) was less than for the less extreme scenario with lower water levels (1 in 1,000-year event for 2030 epoch.

4.1.3. Allowance to account for relative sea level rise was added to the derived tide curves. This was based on UK Climate Projections published in 2018 and is discussed further in **section 4.5**.

Plate 4.1. Derived time series of tide levels



4.2 Wave overtopping

4.2.1. Wave overtopping was calculated using AMAZON, a one-dimensional (in house) software that is specifically designed for simulating wave overtopping of coastal structures. AMAZON has been tested for wave overtopping calculations on single slope walls, slope walls with berms, and vertical seawalls. Further details on AMAZON, its use for the Sizewell C project and comparison with industry standard EurOtop method are provided in the coastal modelling update (Ref 3).

4.2.2. For the purpose of the breach and coastal inundation modelling, wave overtopping was required. This was calculated as overtopping rates (l/s/m) for three profiles representative of the sand dunes and shingle beach system along the Sizewell frontage between the Sizewell Gap and the northern end of the defence along Scott's Hall Drain.

4.2.3. The profiles were derived from topographic surveys supplied by EDF, that were carried out as a part of EDF Energy Beach Monitoring Programme for

Sizewell B Power Station Coast Protection. The extents of the representative cross section profiles are presented in **Plate 4.2**.

4.2.4. For the Scott's Hall Drain frontage profile B16 and for the Sizewell Gap frontage profile B11 were adopted as most representative. Cross-sections of the representative profiles are presented in **Plate 4.3**.

Plate 4.2. Extents of the representative profiles along Sizewell and Minsmere frontages

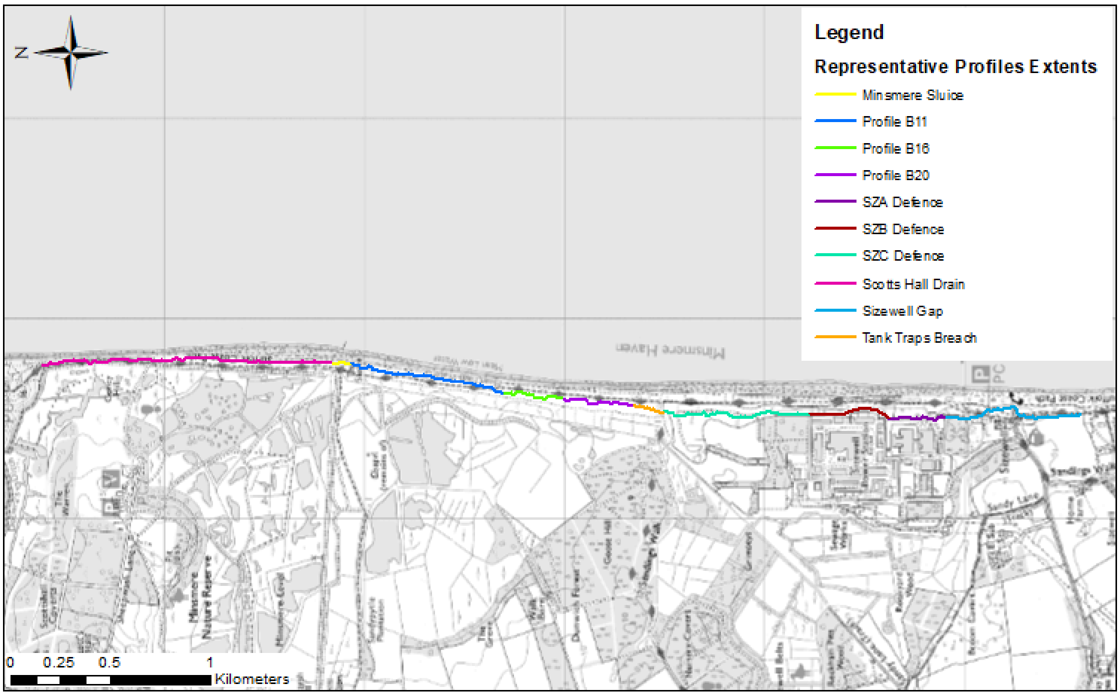
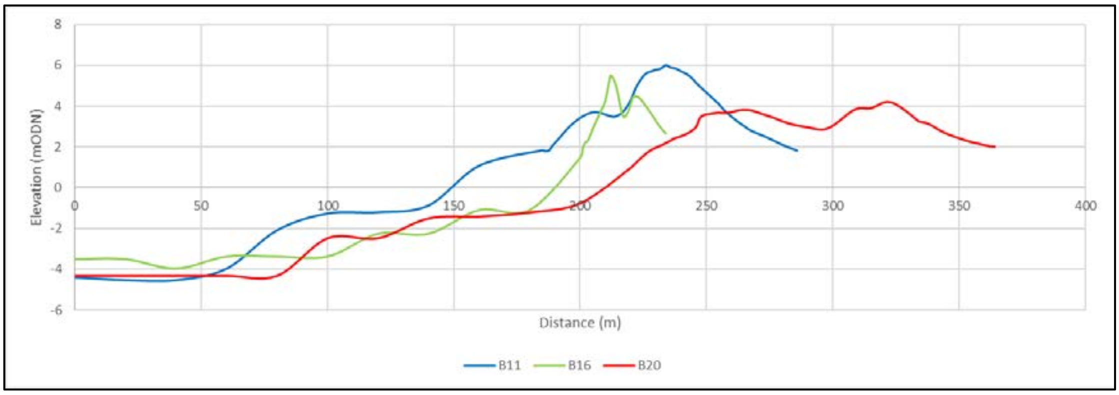


Plate 4.3. Representative profiles cross sections

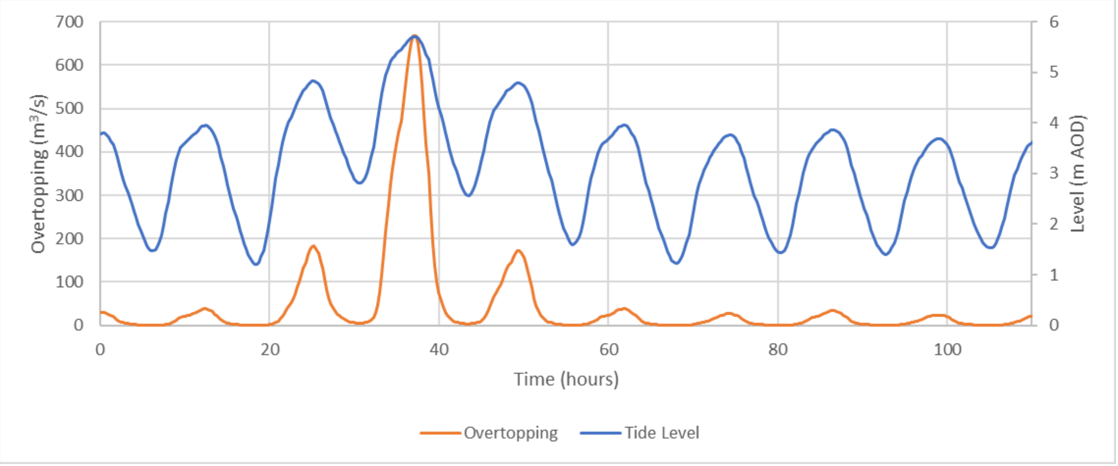


4.2.5. To calculate the overtopping rates, the same approach was adopted as for the modelling carried out for the main development site, using nearshore wave conditions from the wave transformation TOMOWAC model derived for the Sizewell C study (Ref 3) and the still extreme water levels as discussed in **section 4.1**.

4.2.6. In order to derive wave overtopping variable throughout the adopted tide cycle, the overtopping calculations were made at discrete points / water levels of the tide cycle, to provide a relationship between still water level and overtopping rates and derive time-series of overtopping rates depending on tide level.

4.2.7. The overtopping rates calculated for the representative profiles were then converted to volumes for respective frontage sections, depending on the length of the frontage represented by each profile as illustrated in **Plate 4.4** (section represented by cross-section profile B11). These were applied in the model using a Flow vs Time (ST) boundary in TUFLOW.

Plate 4.4. Wave Overtopping rate for the 1 in 200yr event 2190 epoch (representative profile B11)



4.2.8. An f factor for each ST line was applied to divide the total overtopping inflow over the line by the number of grid cells the line crosses in the model.

4.2.9. No overtopping was applied through the breach throat as the still water level was above the invert in the breach throat and therefore rapid flow drowns out wave effects.

4.3 Fluvial inflows

4.3.1. A full set of fluvial boundary conditions for all sub-catchments within the model domain were derived for the works carried out as a part of the fluvial flood risk assessment for the Sizewell C project (Ref 2).

4.3.2. For the purpose of the breach and coastal inundation modelling, two fluvial inflow points were included in the 2D TUFLOW model to represent the two main rivers flows, namely the Leiston Drain and the River Minsmere.

4.3.3. Considering the focus of this exercise was on assessing impacts of tidal breach and coastal inundation, only a nominal baseflow of $0.1m^3/s$ and $0.8m^3/s$ was applied for the Leiston Drain and River Minsmere respectively.

4.4 Initial water levels

4.4.1. An initial water level of 0.5m AOD was applied across the whole 2D model domain so that the watercourses and low-lying areas in the Minsmere Levels and Sizewell Belts were 'wet' at the start of the simulation. Although it is a slightly conservative approach, it was considered appropriate for this study as it ensures storage within the system is not overestimated and also helps with stability of the model at initial timesteps. This initial water level was derived from the fluvial model (Ref 2).

4.5 Climate change scenarios

4.5.1. For the climate change allowances, two main datasets were used. The UK Climate Projections (Ref 11), published in November 2018 (UKCP18), were adopted for the reasonably foreseeable scenario, whereas the more conservative, BECC Upper allowances derived for the Sizewell C project (Ref 12) were applied for the credible maximum scenario.

4.5.2. For UKCP18 allowances for the Representative Concentration Pathway (RCP) scenario 8.5 (similar to high emissions scenario from UKCP09) at the 95^{th} percentile were adopted, based on the plume of sea level anomalies for marine projections around UK coastline. In line with the Environment Agency advice (Ref 13) on 'How to extrapolate the UKCP18 dataset for sea level rise allowances beyond 2100', the 21st century projections were extrapolated up to 2125 and the extended projections to 2300 were used for epochs beyond 2125.

4.5.3. Following initial consideration, it was found that the UKCP18 climate change allowances would not produce high enough sea levels to assess flood risk from breach of the main hard defences (HCDF). This was because the still water levels for the 1 in 1,000-year event at 2140 would be 5.2m AOD and 6.0m AOD at 2190 whilst the platform is designed to a level of 7.3mAOD. Therefore, a breach using this allowance would be unlikely and would not pose risk of inundating the site.

4.5.4. Therefore, the more conservative BECC Upper climate scenario was adopted for those runs. The epoch chosen was the 2140 (end of interim spent fuel store decommissioning phase) as the primary interest for these runs was the impact upon the site itself. The applied extreme sea levels for this scenario are higher than the reasonably foreseeable scenario for 2190 epoch and therefore results from this assessment could be used to estimate flood risk on site up to the theoretical maximum site lifetime.

4.5.5. **Table 4.1** provides a summary of derived extreme sea levels with adopted sea level rise allowances for the two considered return period events, namely 1 in 200-year and 1 in 1,000-year and corresponding climate change epochs.

Table 4.1. Adopted relative sea level rise allowance and derived extreme sea levels for considered scenarios

Year / Epoch	Climate Change Scenario	Relative Sea Level Rise Allowance (m)	Extreme Water Level (m AOD)	
			200yr	1,000yr
2017	-	-	3.11	3.43
2030	RCP8.5 (UKCP18)	0.09	3.20	3.52
2190		2.59	5.70	6.02
2140	BECC Upper	3.92	7.03	7.35

4.5.6. In this study, 2030 epoch was modelled as indicative time within construction phase when key infrastructure would be in place, i.e. the main platform, the SSSI crossing, access and haul road, main sea defence (HCDF) and the northern mound. The projected timeframe for the construction phase is 10-12 years starting from 2022 (end of construction at 2034). However, the sea level rise allowance between 2030 and 2034 is 3cm and therefore considered not significant to overall conclusions of this assessment.

4.5.7. Further details on derivation of climate change allowances are provided in the report on UKCP18 review and proposed response (Ref 14).

5 MODEL RUN PARAMETERS

5.1 Run parameters

5.1.1. All simulations were run using TUFLOW version 2017-09-AA-iDP-w64. It is acknowledged that currently latest version of the TUFLOW software was released in 2018, however due to time constraints that version was not used at this stage of the Sizewell C works. A sensitivity test on potential changes to the results within 2D model domain was carried out as a part of the fluvial modelling works for Sizewell C that is discussed in the fluvial modelling report (Ref 2).

5.1.2. Each model simulation was run to simulate 99 hours which allowed suitable time for the flood water to propagate within the Minsmere Levels and Sizewell Belts systems ensuring that maximum flood levels were reached within the model domain.

5.1.3. A fixed time-step of 2 seconds was applied to the model. This time-step was chosen as appropriate given the 4m cell size of the 2D grid.

5.2 Model scenarios

5.2.1. As discussed in **section 1.3**, to inform the FRA, two return periods were assessed in the breach and coastal inundation models, namely 1 in 200-year and 1 in 1,000-year, each with an allowance for climate change for both, baseline and ''with scheme'' model schematisations.

5.2.2. Lower return periods (than the 1 in 200-year) were not assessed as risk of breach at such events is significantly lower and corresponding extreme sea levels would be below the crest of the coastal defences resulting in very limited impact of the breach and inundation of areas behind the defences.

5.2.3. As discussed in **section 4.5**, for the scenario with breach of main hard sea defence (HCDF) only one epoch was considered, namely 2140 with credible maximum climate change allowance. This was to ensure extreme sea levels are high enough to represent a plausible breach scenario of the engineered hard defence that would result in flooding of the main platform (at 7.3m AOD).

5.2.4. For the assessment of flood risk due to breach, three breach locations were simulated, as discussed in **section 3.1**. A list of scenarios adopted for the breach modelling is provided in **Table 5.1**.

5.2.5. Additionally, sensitivity tests on the model roughness was carried out. A standard +/- 20% change in the Manning's roughness coefficient was applied for the whole model domain. This was run for the baseline model schematisation for the 1 in 1,000-year return period event at 2190 epoch.

Table 5.1: List of assessed breach model scenarios

<table>
<tr><th>Return Period</th><th>Epoch</th><th>Climate Change Scenario</th><th>Breach Location</th><th>Model Schematisation</th></tr>
<tr><td rowspan="2">200</td><td rowspan="4">2030</td><td rowspan="4">RCP8.5 (UKCP18)</td><td rowspan="2">Tank traps</td><td>Baseline</td></tr>
<tr><td>'with scheme'</td></tr>
<tr><td rowspan="2">1,000</td><td rowspan="2">Tank traps</td><td>Baseline</td></tr>
<tr><td>'with scheme'</td></tr>
<tr><td rowspan="2">200</td><td rowspan="4">2190</td><td rowspan="4">RCP8.5 (UKCP18)</td><td rowspan="2">Tank traps</td><td>Baseline</td></tr>
<tr><td>'with scheme'</td></tr>
<tr><td rowspan="2">1,000</td><td rowspan="2">Tank traps</td><td>Baseline</td></tr>
<tr><td>'with scheme'</td></tr>
</table>

Return Period	Epoch	Climate Change Scenario	Breach Location	Model Schematisation
200	2190	RCP8.5 (UKCP18)	Sizewell Gap	Baseline
				'with scheme'
200	2140	BECC Upper	Main sea defence (HCDF)	Baseline
				'with scheme' – Option 1
				'with scheme' – Option 2
1,000	2140		Main sea defence (HCDF)	Baseline
				'with scheme' – Option 1
				'with scheme' – Option 2
1,000	2190	RCP8.5 (UKCP18)	Tank traps	-20% roughness (Baseline)
				+20% roughness (Baseline)

5.2.6. Similar to the breach modelling, a set of model runs was carried out to assess flood risk to the site itself as well as potential change in flood risk to off-site receptors as a result of coastal inundation (with a breach of defences) and propagation of flood water within Minsmere Levels and Sizewell Belts systems. A list of scenarios adopted for the coastal inundation modelling is provided in **Table 5.2**.

Table 5.2. List of assessed coastal inundation model scenarios

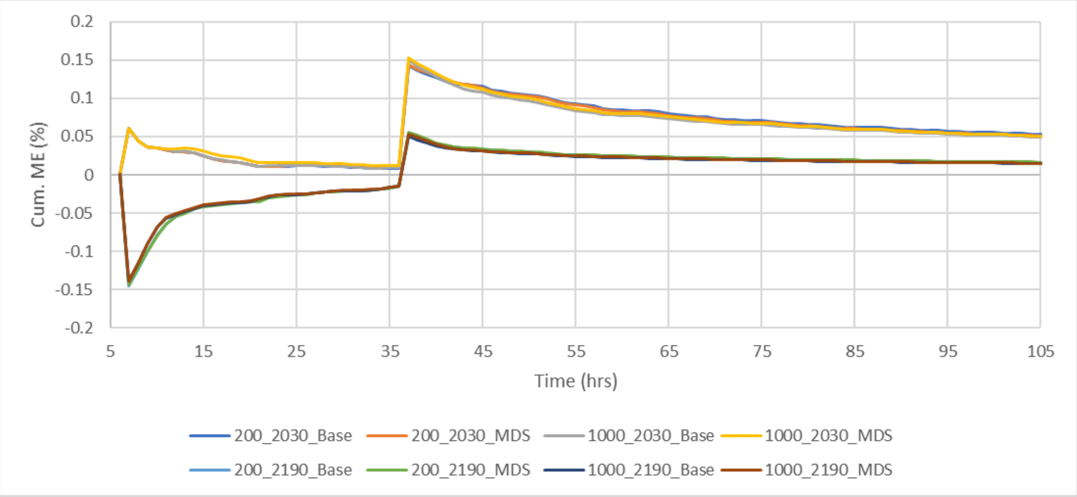
Return Period	Epoch	Climate Change	Model Schematisatio
200	2030	RCP8.5 (UKCP18)	Baseline
			'with scheme'
1,000			Baseline
			'with scheme'
200	2190	RCP8.5 (UKCP18)	Baseline
			'with scheme'
1,000			Baseline
			'with scheme'

5.3 Model stability

5.3.1. Numerical convergence has been checked through examination of the mass balance time series within the MB2D.csv results. In general terms, a model is considered to have good stability if cumulative mass balance errors are less than 1%.

5.3.2. The mass balance outputs for the baseline and ''with scheme'' breach model runs are shown in **Plate 5.1**. The results of the mass error show that the model is within the expected tolerance.

Plate 5.1. Mass balance plots for the baseline and ''with scheme'' model runs for the considered scenarios



5.3.3. When investigating model results, negative depths were identified in several of the modelled scenarios, however these are significantly reduced from the previous model iteration, ranging from 0 negative depths in both of the baseline 2030 epoch runs, up to 620 negative depths for the ''with scheme'' model run for the 1 in 200-year event at 2190 epoch.

5.3.4. Given the relatively large temporal and spatial scale of the model and combined with the good mass balance plots, this is deemed acceptable at this stage of the works.

6 MODEL RESULTS

6.1 Overview

6.1.1. Model results were investigated for all considered scenarios and corresponding flood depth, velocity and hazard maps were produced and are presented in **Appendix A** for the baseline scenarios and **Appendix B** for the 'with scheme' scenarios.

6.1.2. Plots of difference in maximum flood depth, velocity and hazard for key considered scenarios are provided in **Appendix C**.

6.1.3. For assessment of change in maximum flood depth, six sample locations (one within each broad geographical area of the floodplain, i.e. Sizewell, Leiston, Minsmere South, Minsmere Central, Minsmere North and Eastbridge) were selected (**Plate 6.1**).

6.1.4. Additionally, based on NRD dataset (2014), comparison of results was also undertaken for residential, commercial and other public properties to determine whether the proposed Sizewell C development impacts flood risk to potential off-site receptors.

6.1.5. Overall the results show that within the Sizewell Belts area the maximum depth in the 'with scheme' scenario is slightly lower than for the baseline scenario. That is due to the SSSI crossing and its embankment constricting the flow of water between the Sizewell Belts and the Minsmere Levels. Consequently, results show slight increase in flood depth within the Minsmere Levels, primarily concentrated in the low-lying area of the marshes and near the SSSI crossing.

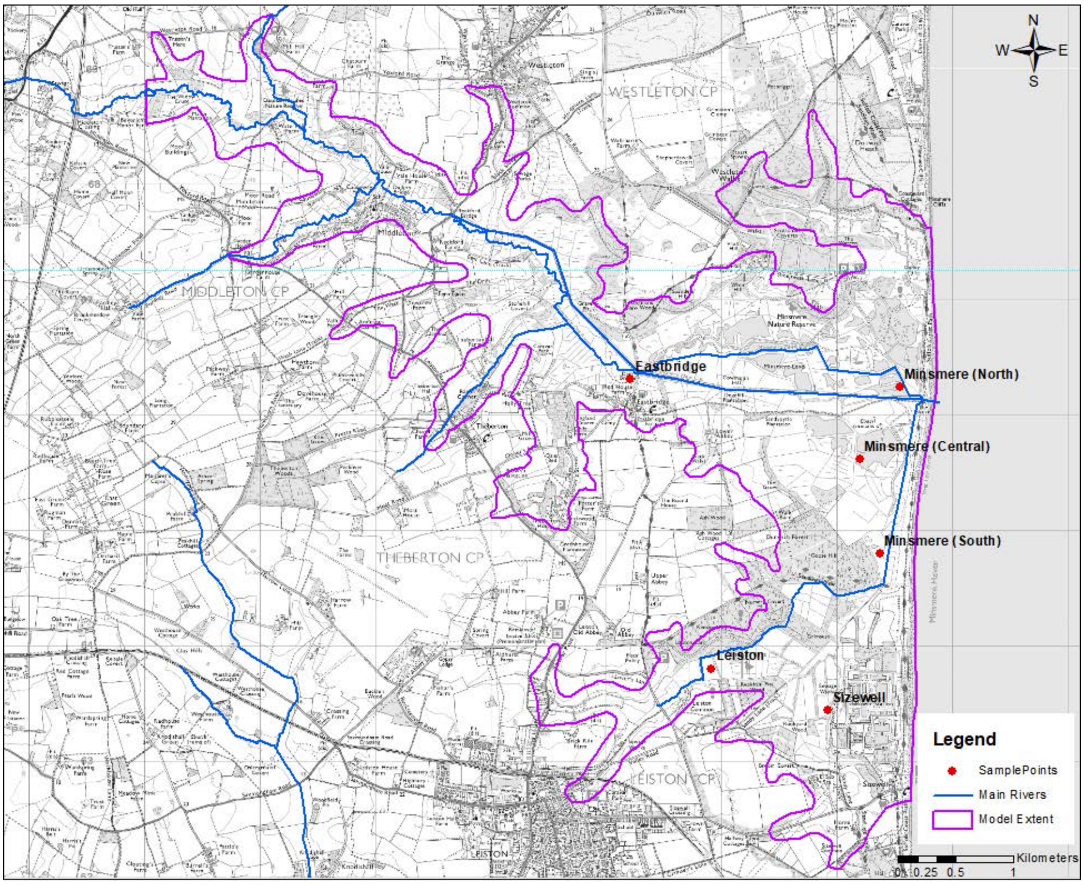
6.1.6. A sensitivity test on alternative breach location was also carried out to ensure that the most conservative breach location was chosen when assessing flood risk to the site and off-site impacts with results discussed further in **section 6.4c**.

6.2 Baseline breach model results

6.2.1. **Plate 6.2** – **Plate 6.7** illustrate the results of maximum flood depth, velocity and hazard for the baseline scenario 1 in 200-year and 1 in 1,000-year return period events for the 2030 with breach at tank traps respectively.

6.2.2. **Table 6.1** and **Table 6.2** present maximum water depths at the selected sample locations (**Plate 6.1**) for the considered return period events at 2030 and 2190 epochs respectively.

Plate 6.1. Location of the selected sample points



6.2.3. Figures presenting results for all considered baseline scenarios for breach at tank traps are provided in **Appendix A**. These include flood depth, velocity and hazard rating maps.

6.2.4. A full list of properties potentially affected (NRD dataset within model domain) with corresponding maximum flood depth, velocity and hazard rating for all considered scenarios are collated in a spreadsheet and provided also in **Appendix A**.

Plate 6.2. Maximum flood depth for the 1 in 200-year return period event at 2030 epoch – baseline: breach at tank traps

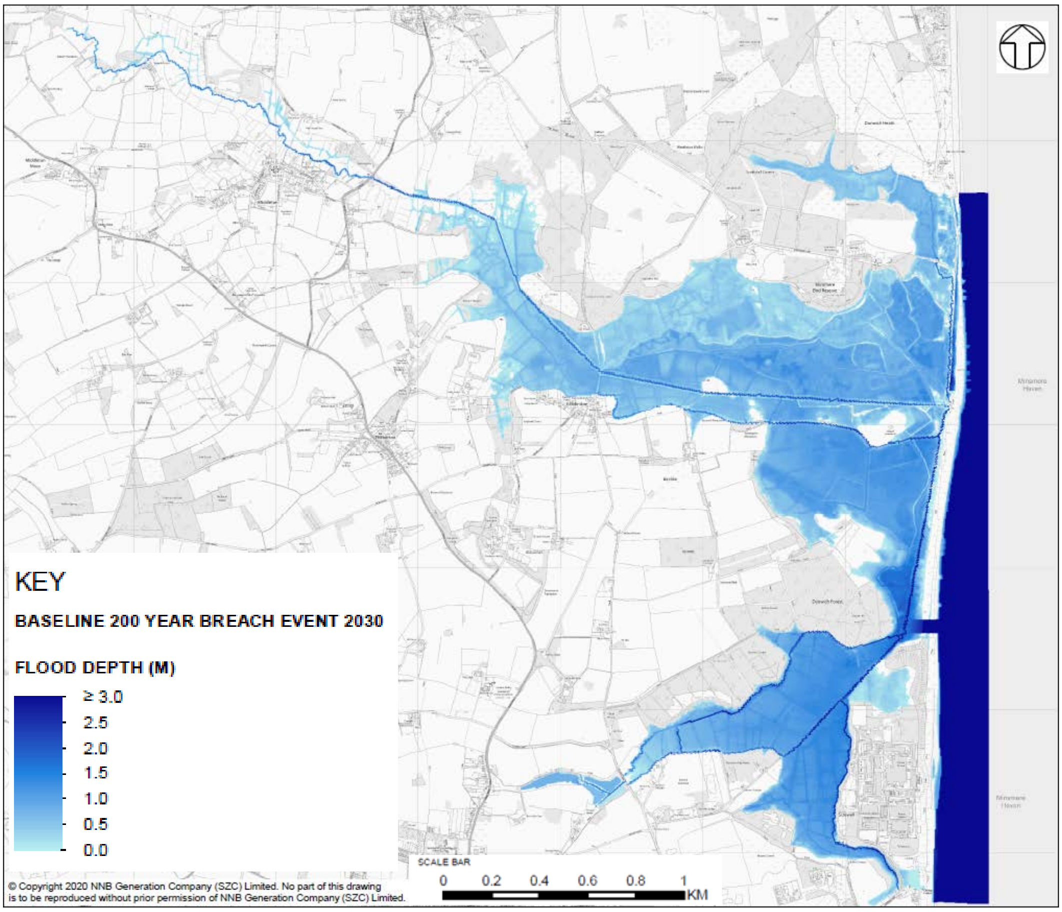


Plate 6.3. Maximum flood depth for the 1 in 1,000-year return period event at 2030 epoch – baseline: breach at tank traps

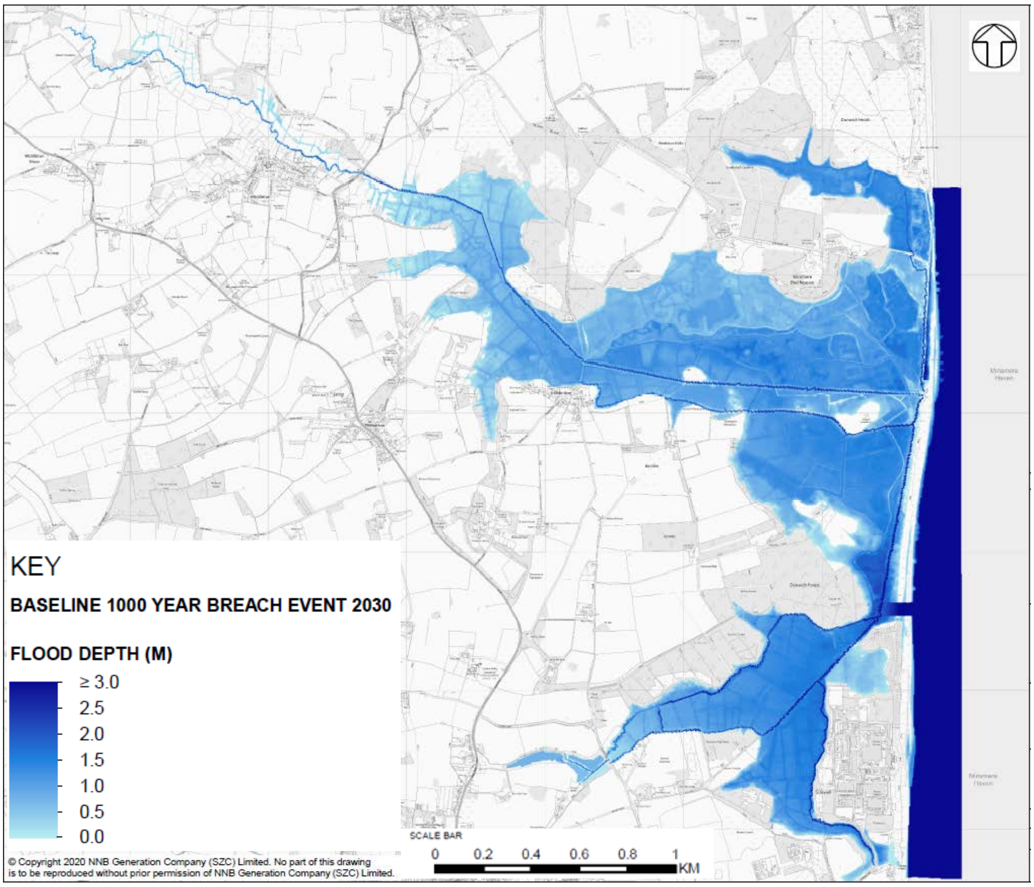


Plate 6.4. Maximum flood velocity for the 1 in 200-year return period event at 2030 epoch – baseline: breach at tank traps

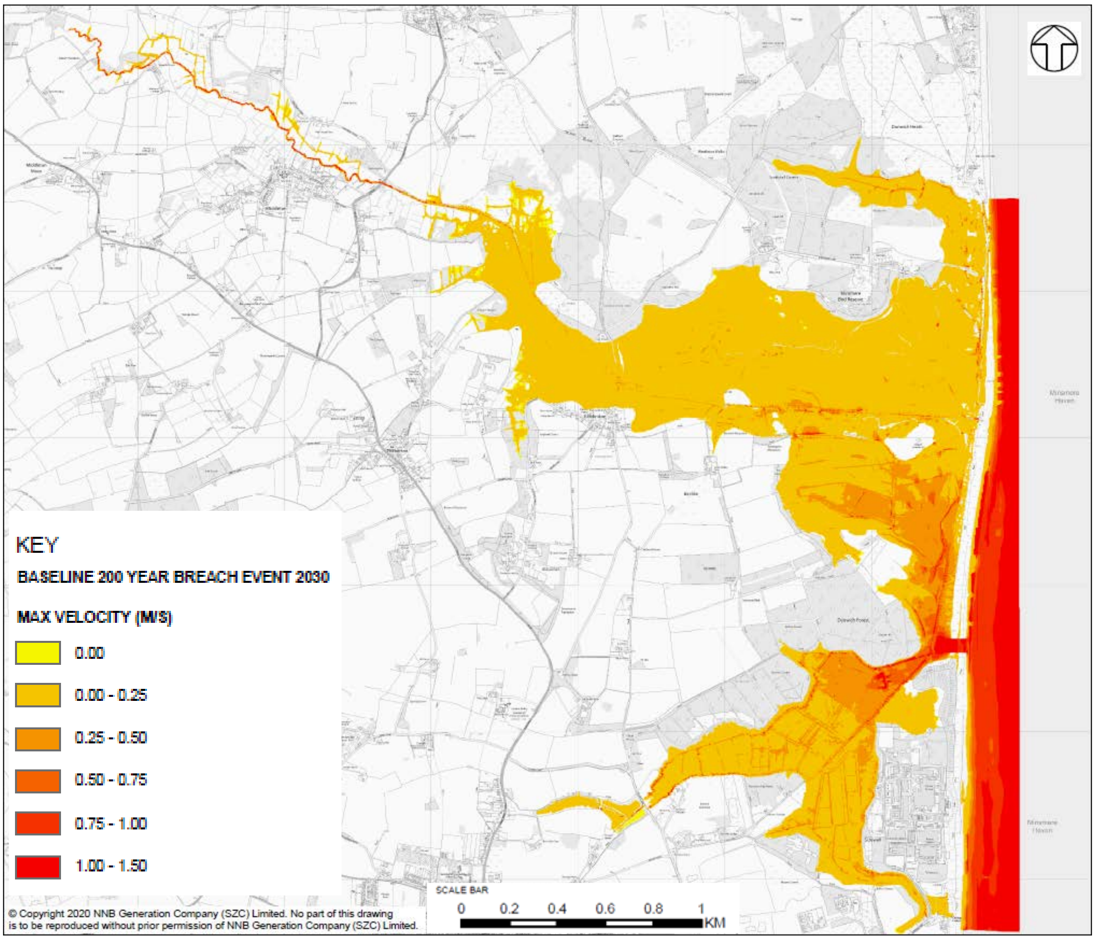


Plate 6.5. Maximum flood velocity for the 1 in 1,000-year return period event at 2030 epoch – baseline: breach at tank traps

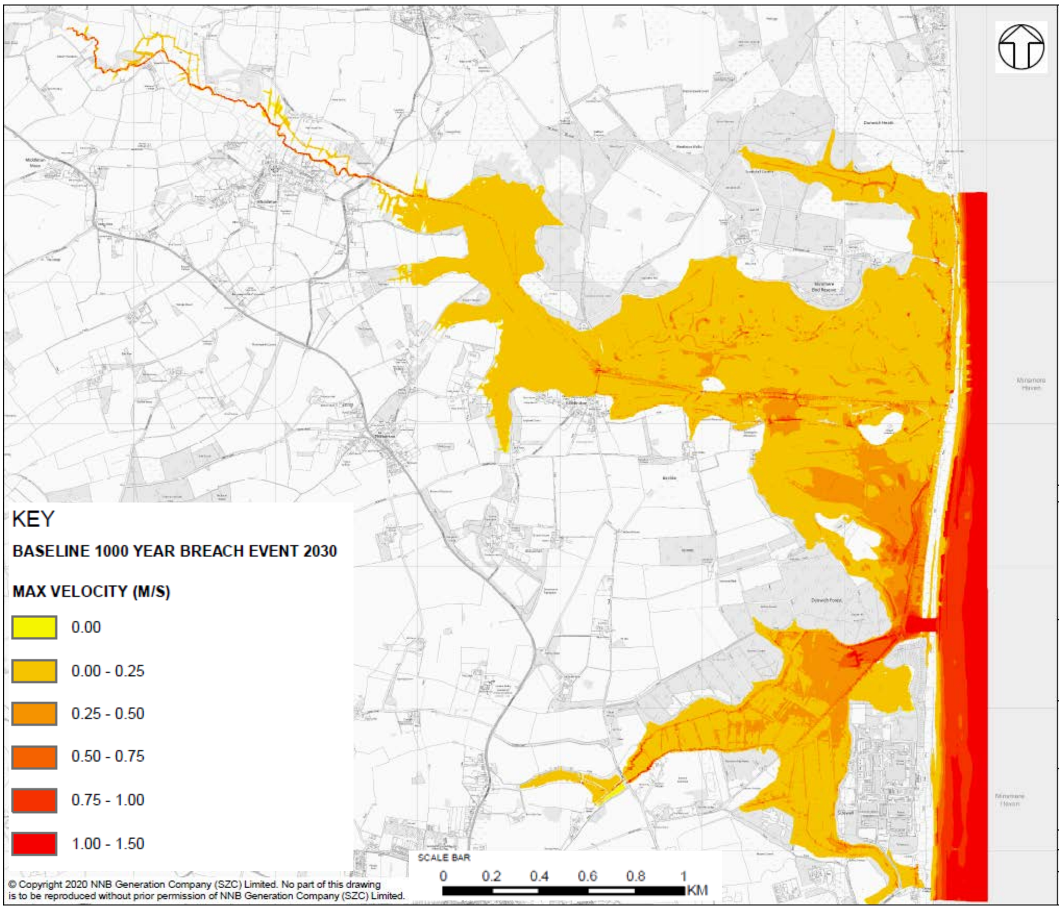


Plate 6.6. Flood hazard rating for the 1 in 200-year return period event at 2030 epoch – baseline: breach at tank traps

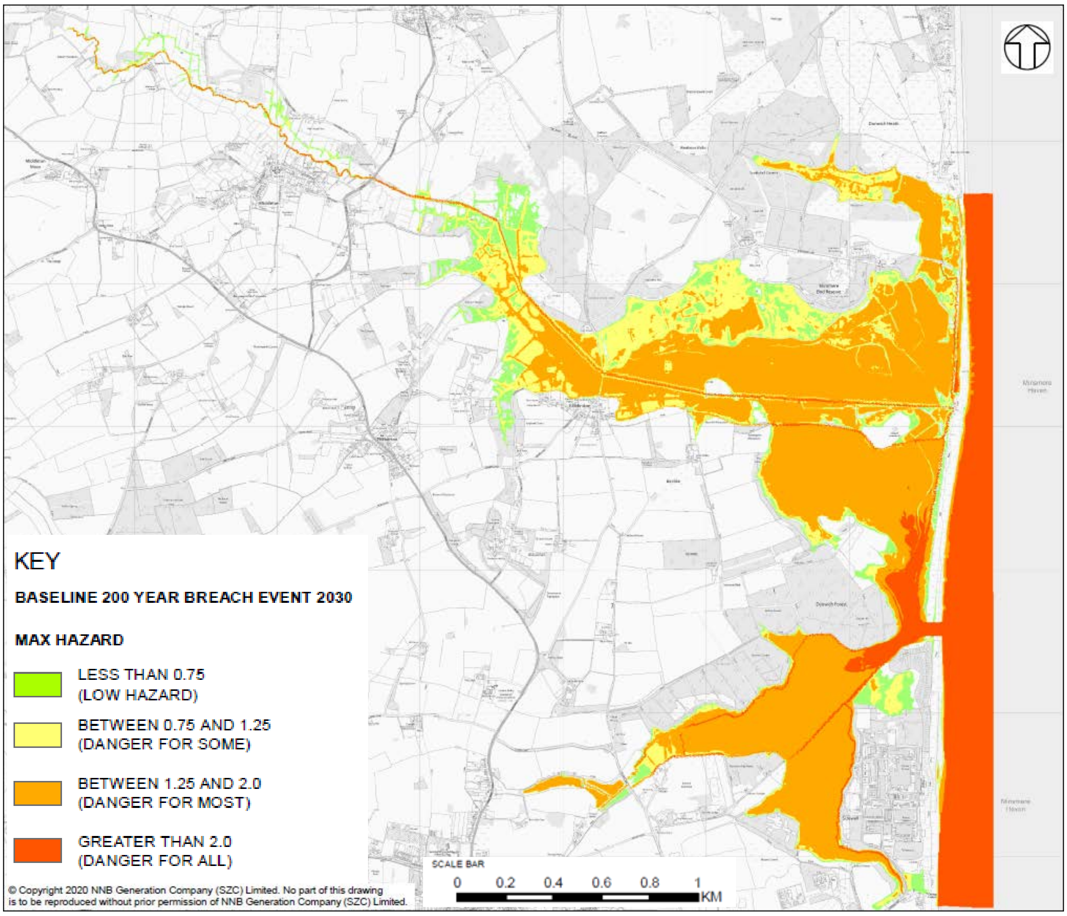


Plate 6.7. Flood hazard rating for the 1 in 1,000-year return period event at 2030 epoch – baseline: breach at tank traps

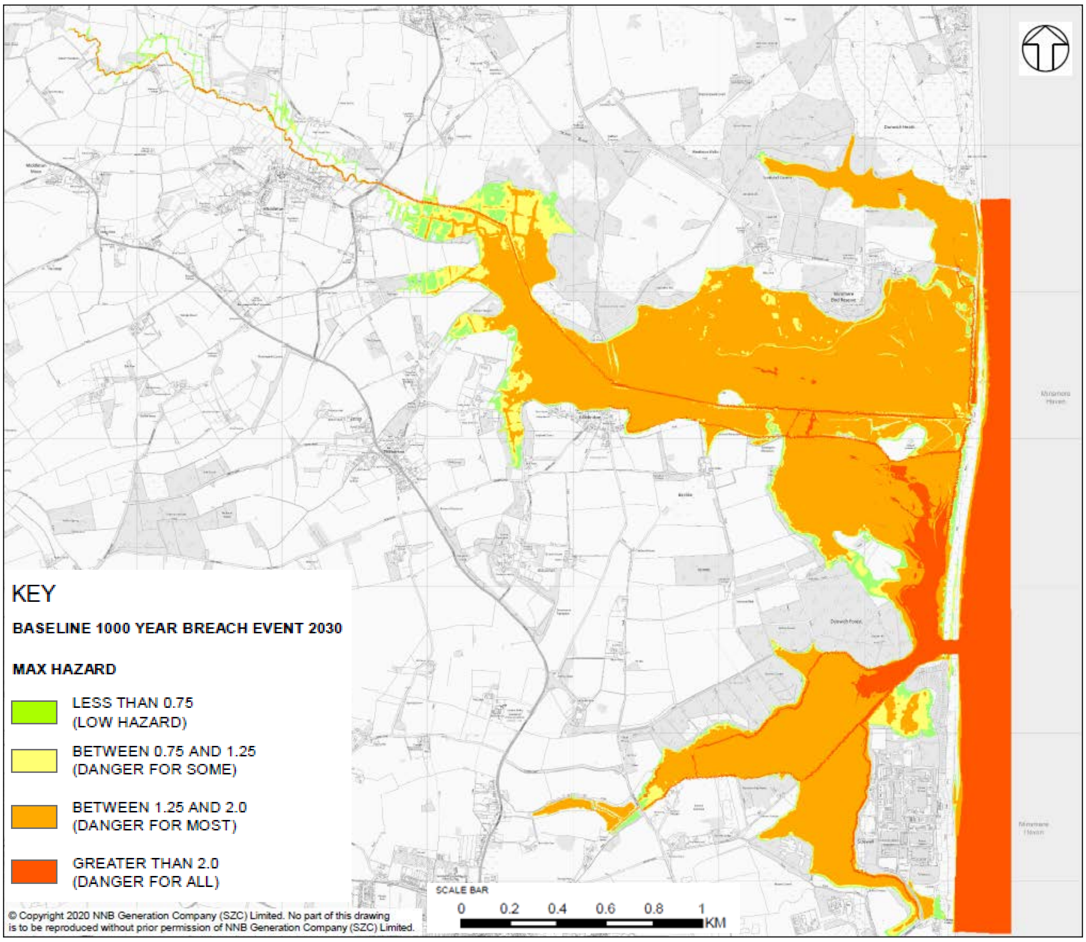


Table 6.1: Maximum flood depths for the 1:200 and 1:1,000-year events (2030 epoch) – baseline: breach at tank traps

Sample Location	Max Depth (m) 1 in 200-year	Max Depth (m) 1 in 1,000-year
Eastbridge	0.78	1.14
Minsmere (North)	1.34	1.70
Minsmere (Central)	1.13	1.38
Minsmere (South)	1.70	1.94
Leiston	0.65	0.88
Sizewell	1.24	1.47

Table 6.2: Maximum flood depths for the 1:200 and 1:1,000-year events (2190 epoch) – baseline: breach at tank traps

Sample Location	Max Depth (m) 1 in 200-year	Max Depth (m) 1 in 1,000-year
Eastbridge	5.25	5.48
Minsmere (North)	5.80	6.03
Minsmere (Central)	5.40	5.63
Minsmere (South)	5.10	5.33
Leiston	4.56	4.79
Sizewell	5.15	5.38

6.2.5. The results show that majority of the Minsmere Levels and Sizewell Belts areas get flooded as a result of breach at tank traps. The greatest flood depths are within the Minsmere Levels and the lowest at the upstream end of the Leiston catchment.

6.2.6. Flood velocity is below 0.25 m/s within most of the flood extent, with some localised areas of velocity up to 0.5 m/s. As a result of relatively great flood depths, the hazard rating for the area is ‘danger for most’ with some areas of ‘danger for all’.

6.3 ‘With scheme’ breach model results

6.3.1. **Plate 6.8** – **Plate 6.13** illustrate the results of maximum flood depth, velocity and hazard for the 1 in 200-year and 1 in 1,000-year return period event for the 2030 epoch with breach at tank traps respectively. **Table 6.3** and **Table 6.4** present the maximum water depths at the selected sample locations (**Plate 6.1**) for all considered ‘with scheme’ scenarios.

6.3.2. Figures presenting results for all considered ‘with scheme’ scenarios for breach at tank traps are provided in **Appendix B.** These include flood depth, velocity and hazard rating maps.

6.3.3. A full list of properties potentially affected (NRD dataset within model domain) with corresponding maximum flood depth, velocity and hazard rating for all considered scenarios are collated in a spreadsheet and provided also in **Appendix B**.

Plate 6.8. Maximum flood depth for the 1 in 200-year return period event at 2030 epoch – 'with scheme': breach at tank traps

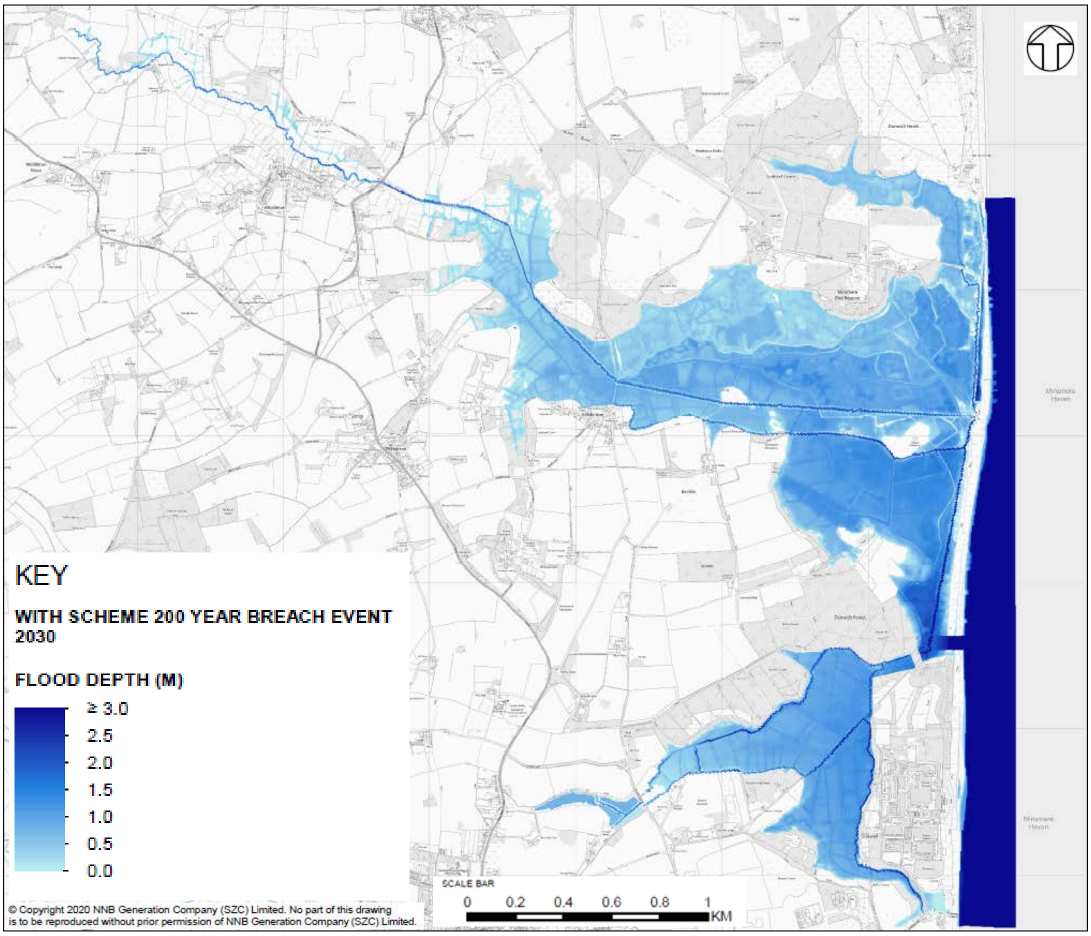


Plate 6.9. Maximum flood depth for the 1 in 1,000-year return period event at 2030 epoch – 'with scheme': breach at tank traps

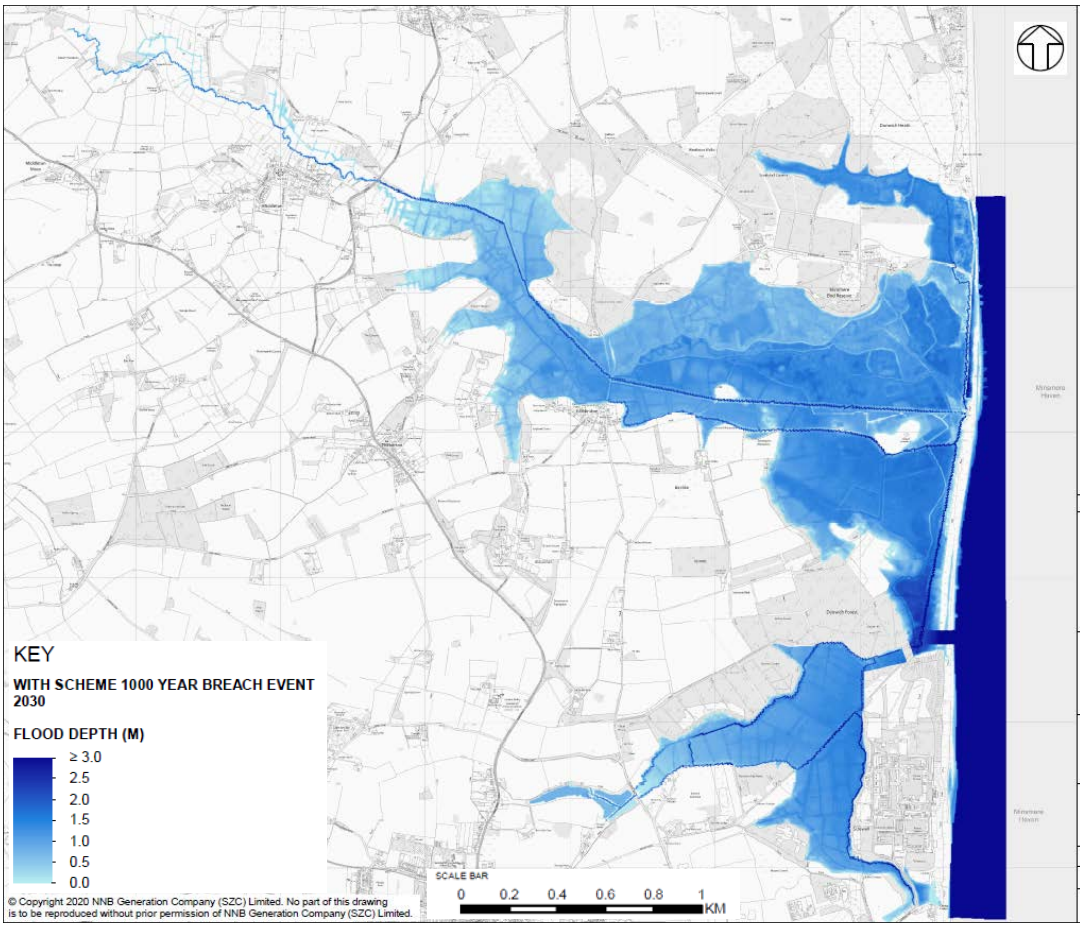


Plate 6.10. Maximum flood velocity for the 1 in 200-year return period event at 2030 epoch – 'with scheme': breach at tank traps

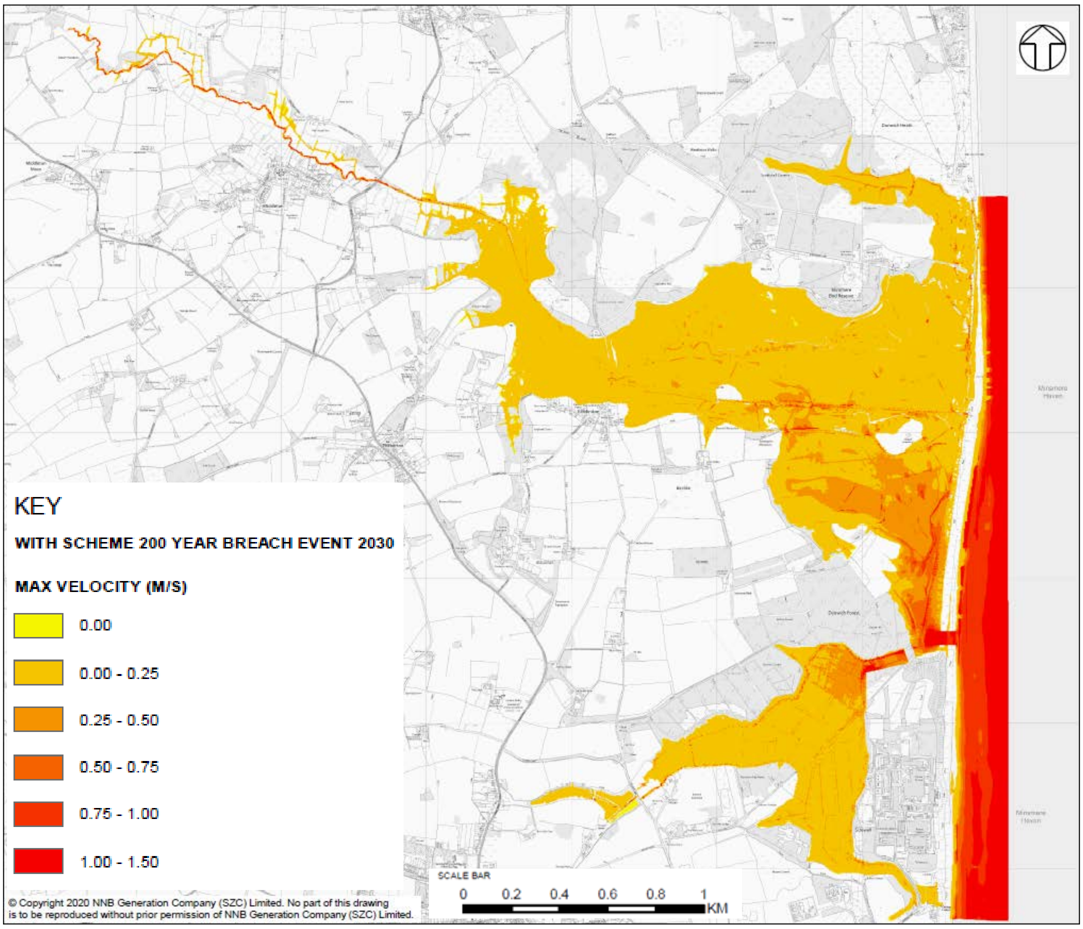


Plate 6.11. Maximum flood velocity for the 1 in 1,000-year return period event at 2030 epoch – 'with scheme': breach at tank traps

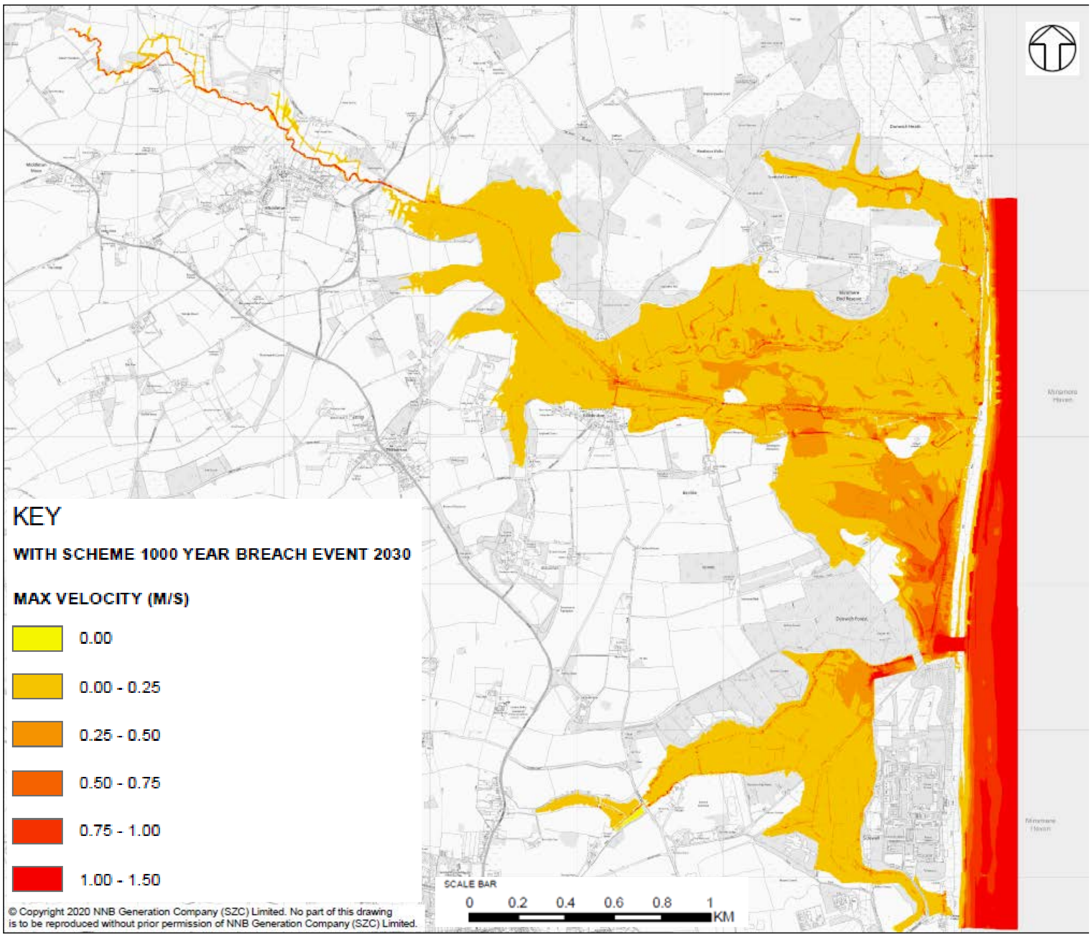


Plate 6.12. Flood hazard rating for the 1 in 200-year return period event at 2030 epoch – 'with scheme': breach at tank traps

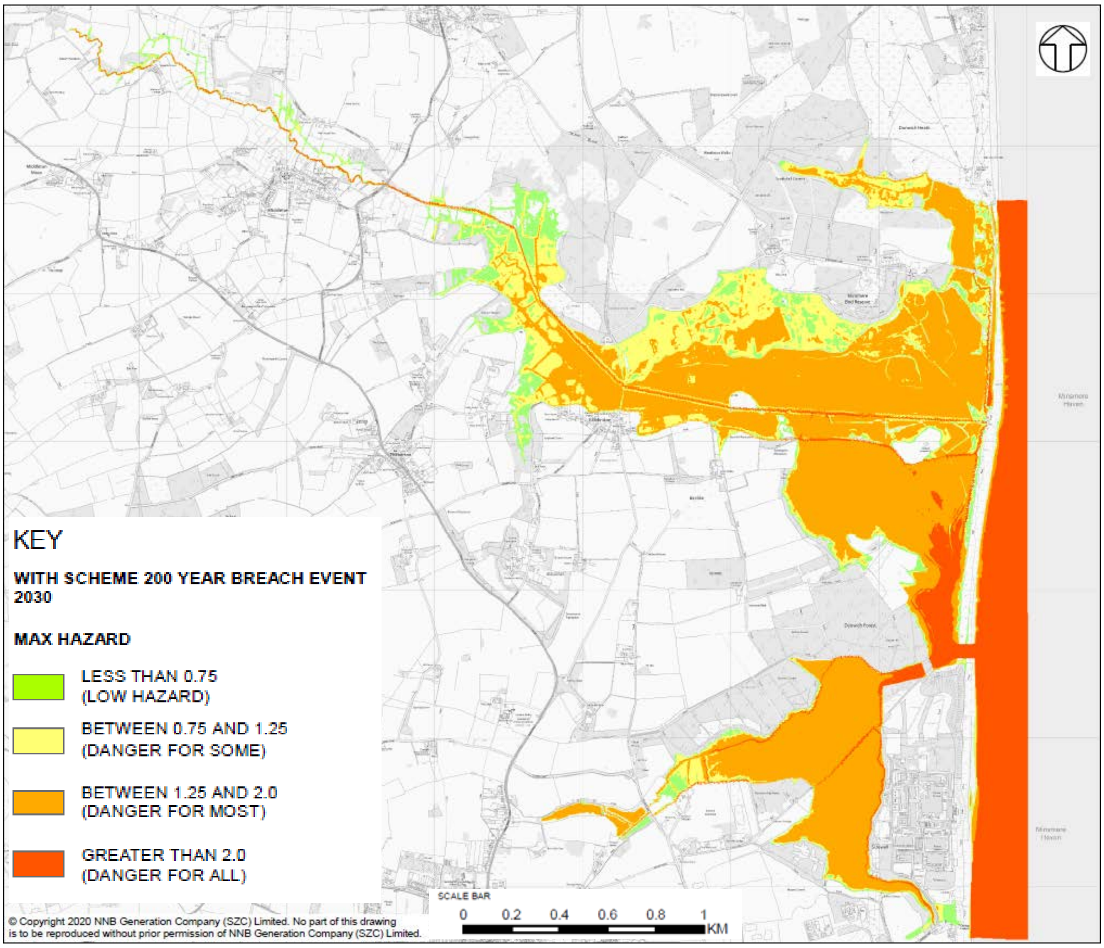


Plate 6.13. Flood hazard rating for the 1 in 1,000-year return period event at 2030 epoch – 'with scheme': breach at tank traps

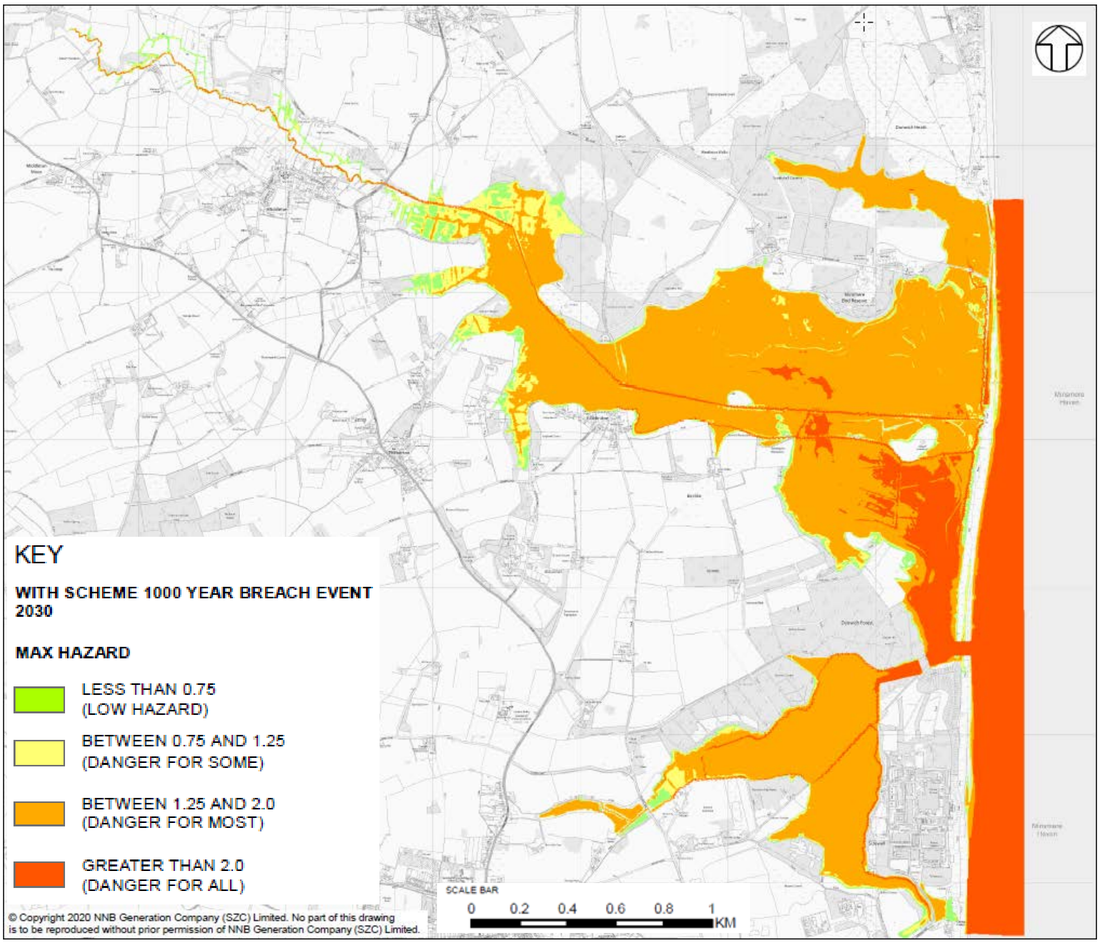


Table 6.3. Maximum flood depths for the 1:200 and 1:1,000-year events (2030 epoch) – 'with scheme': breach at tank traps

Sample Location	Max Depth (m) 1 in 200-year	Max Depth (m) 1 in 1,000-year
Eastbridge	0.80	1.18
Minsmere (North)	1.36	1.74
Minsmere (Central)	1.26	1.51
Minsmere (South)	1.88	2.15
Leiston	0.49	0.69
Sizewell	1.08	1.28

Table 6.4. Maximum flood depths for the 1:200 and 1:1,000-year events (2190 epoch) – 'with scheme': breach at tank traps

Sample Location	Max Depth (m) 1 in 200-year	Max Depth (m) 1 in 1,000-year
Eastbridge	5.25	5.48
Minsmere (North)	5.80	6.03
Minsmere (Central)	5.39	5.62
Minsmere (South)	5.09	5.32
Leiston	4.56	4.80
Sizewell	5.15	5.38

6.3.4. Similar to the baseline results, the 'with scheme' breach model results show that majority of the Minsmere Levels and Sizewell Belts areas get flooded with flood levels reaching over 5.0m AOD.

6.3.5. Flood velocity is very similar to the baseline results, mostly below 0.25 m/s, with some areas up to 0.5 m/s. As a result of relatively high flood depths, the hazard rating for the area is 'danger for most' with some areas of 'danger for all'.

6.3.6. All results presented above show that the proposed development (main platform, SSSI crossing and the access road) would not be at risk of flooding from breach of the coastal defences / sand dunes (tank traps) under the two considered return period events and climate change epochs for the reasonably foreseeable scenario. Although only two climate change epochs were assessed, based on the results for the 2190 epoch (theoretical site lifetime) and the fact that any intermediate epochs would be based on lower extreme water levels, it can be concluded that the site would not be at risk of flooding from breach scenario (with reasonably foreseeable climate change) throughout all development phases.

6.3.7. **Plate 6.14** and **Plate 6.15** show the maximum flood depth for the scenario with breach at the main hard coastal defence feature (HCDF), Option 1, for the 1 in 200-year and 1 in 1,000-year events at 2140 epoch respectively. The symbology for the water depth in those plots was selected to show different water depth at the platform, whereas the water depth elsewhere was represented in one colour not to skew the colour scheme by much greater water depths (above 6m) within the Minsmere Levels and Sizewell Belts.

6.3.8. The results for the breach at main sea defence (HCDF) show that majority of the platform would be flooded up to approximately 0.3m depth, with the area closest to the breach location flooded up to approximately 0.7m depth.

6.3.9. This scenario was simulated with credible maximum climate change allowances and therefore is very conservative. Also, the width of the breach was set 100m, which is also conservative for a breach of hard defences. In addition, no mitigation measures such as drainage design were included in the model, meaning that these results present the worst-case scenario.

Plate 6.14. Maximum flood depth for the 1 in 200-year return period event at 2140 epoch – 'with scheme': breach at main sea defence (HCDF)

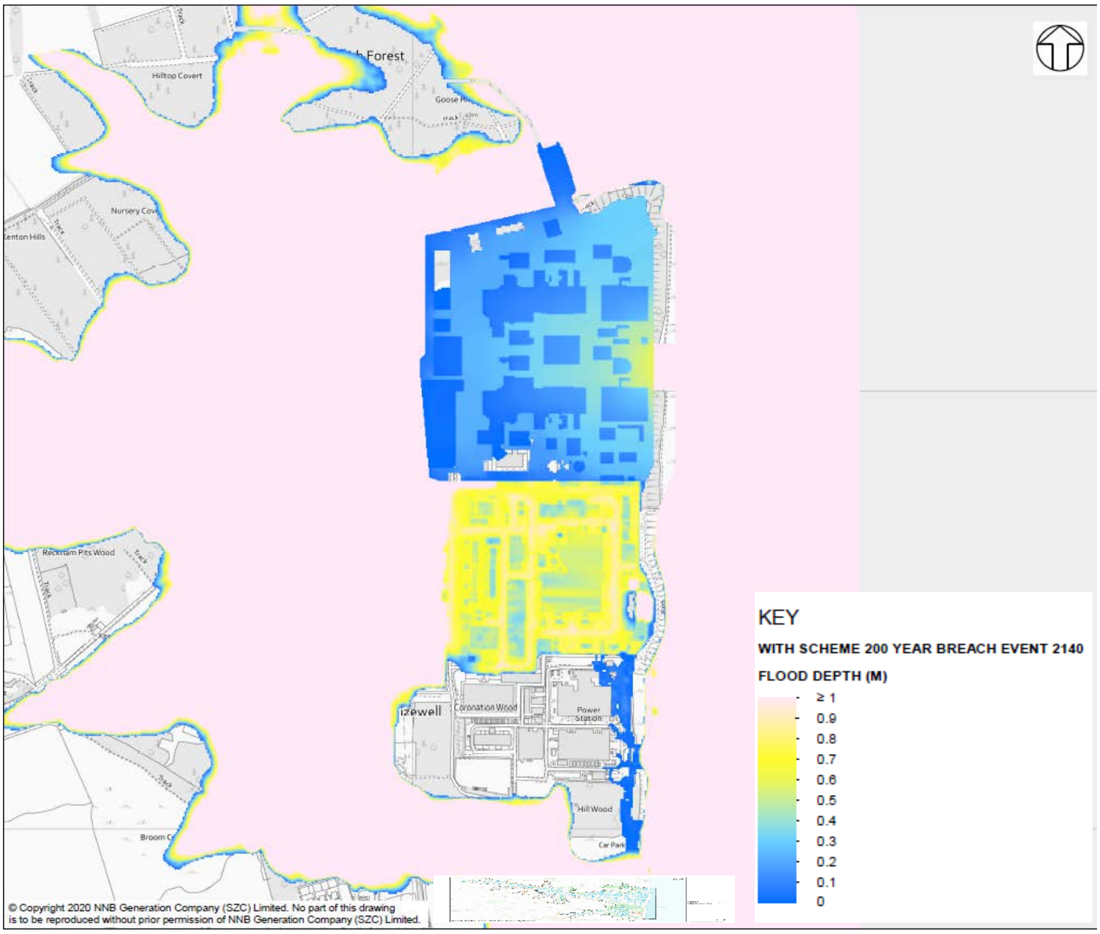
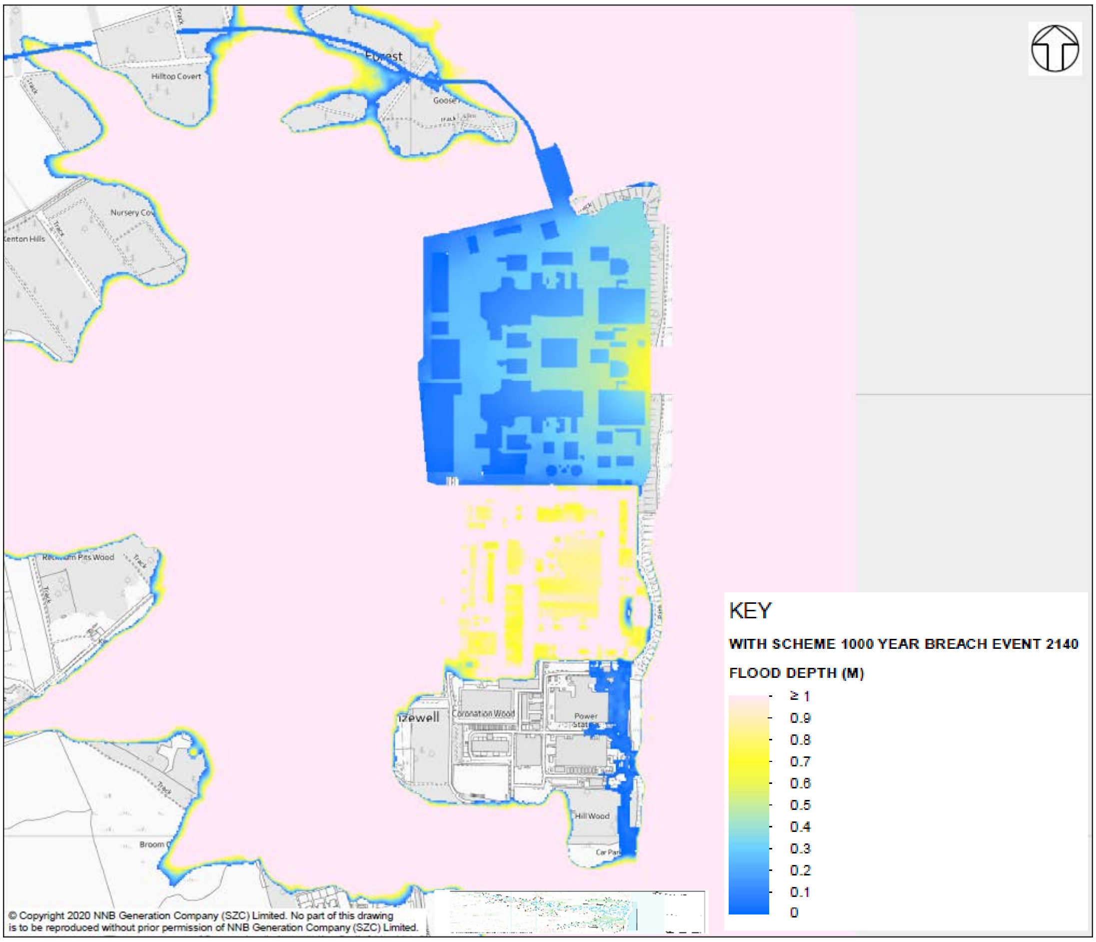


Plate 6.15. Maximum flood depth for the 1 in 1,000-year return period event at 2140 epoch – 'with scheme': breach at main sea defence (HCDF)



6.3.10. Although the results show that the platform would be at risk of flooding from breach of main sea defence (HCDF) and the resulting flood depth would be above the standard threshold for building, the proposed development would comprise other measures, such as water-resistant buildings, higher floor levels, drainage design on the platform, etc., that would mitigate flood risk to the operation of the site. Also, assessed breach scenario was at 2140 epoch, when the operation of the power station would be completed and therefore limited activities would be taking place.

6.3.11. Further assessment of the risk and mitigation measures would be discussed in the Safety Case assessment, as required by the ONR regulations.

6.4 Comparison of breach model results

a) Tank traps

6.4.1. **Plate 6.16** - **Plate 6.18** illustrate the differences in maximum flood depth ('with scheme' minus baseline) as a result of breach at tank traps for the 1 in 200-year at 2030 and the 1 in 1,000-year return period event at 2030 and at 2190 epochs respectively. **Table 6.5** and **Table 6.6** present the differences in the maximum water depths at the selected sample locations (**Plate 6.1**) for the 2030 and 2190 epochs respectively.

Plate 6.16. Difference in maximum flood depth for the 1 in 200-year return period event at 2030 – breach at tank traps

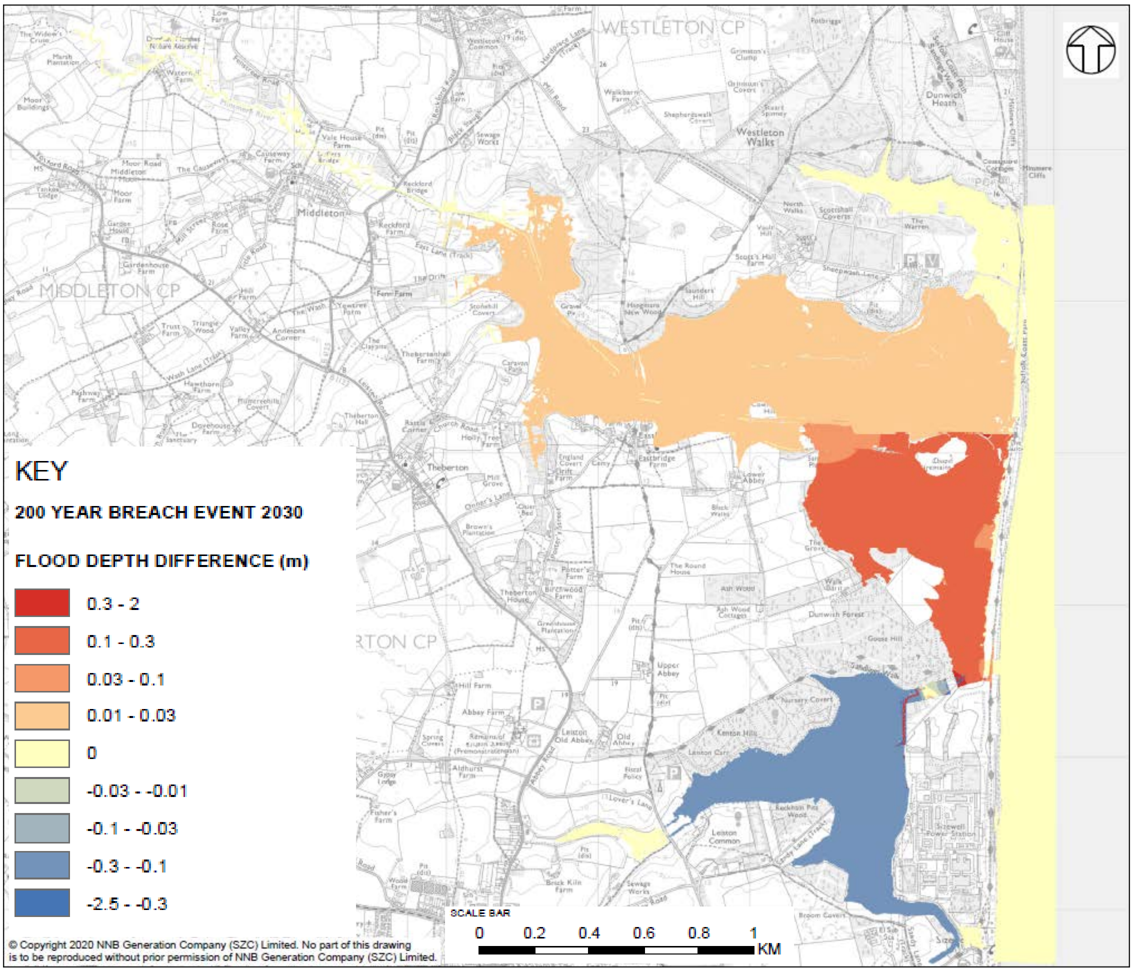


Plate 6.17. Difference in maximum flood depth for the 1 in 1,000-year return period event at 2030 – breach at tank traps

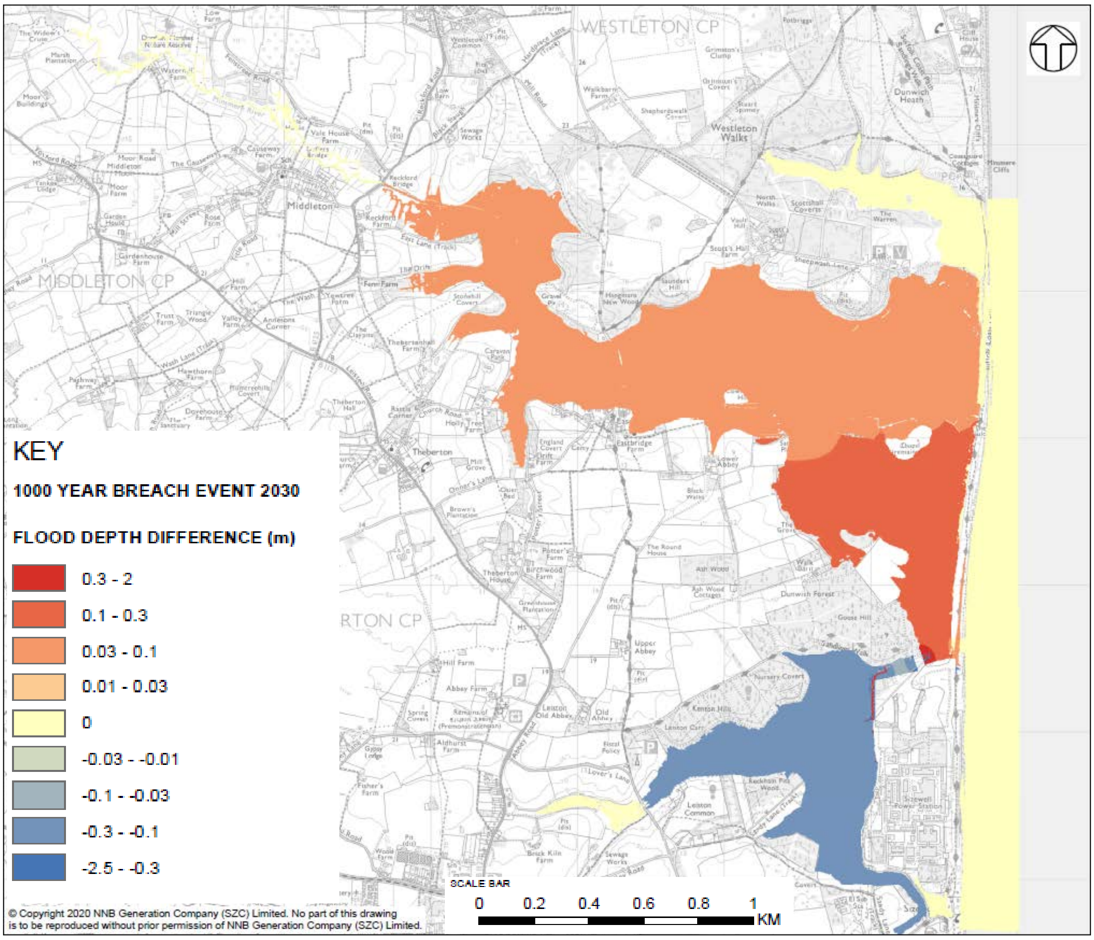


Plate 6.18. Difference in maximum flood depth for the 1 in 1,000-year return period event at 2190 – breach at tank traps

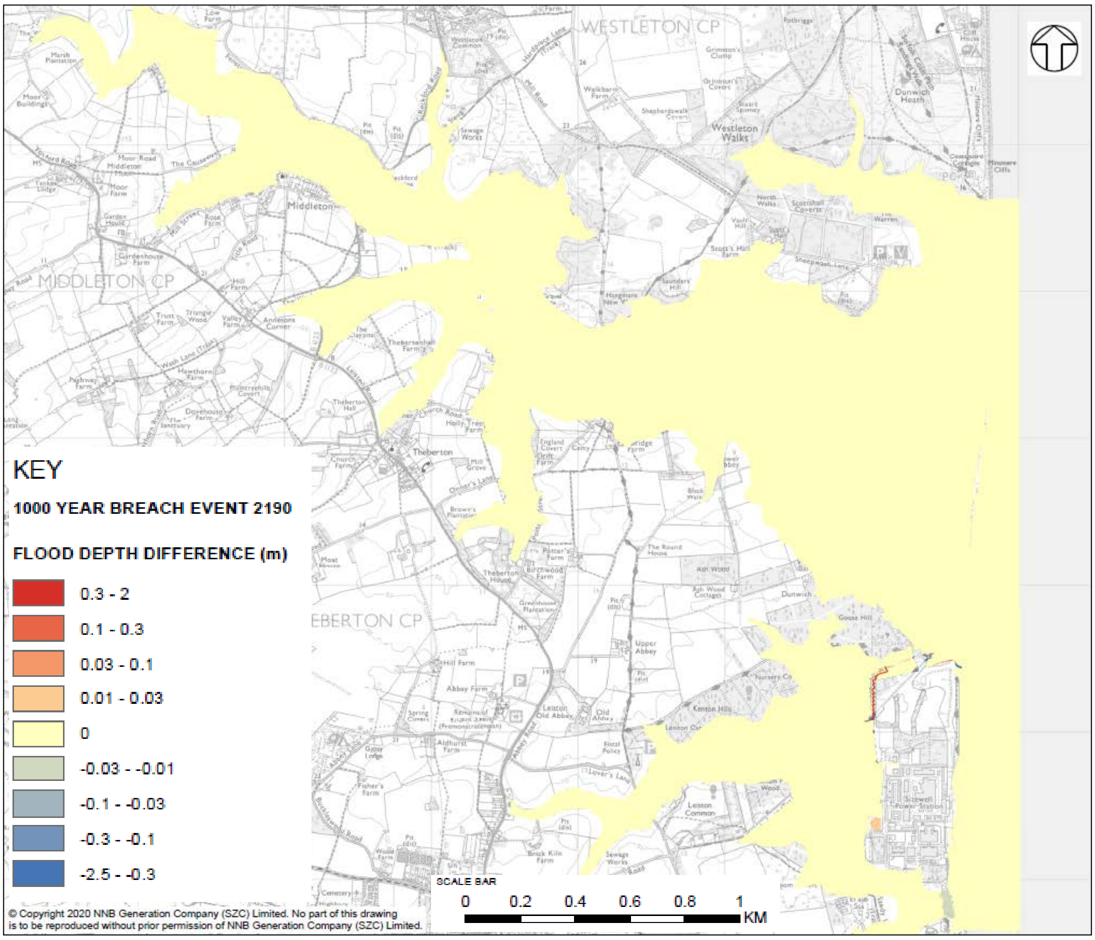


Table 6.5: Difference in maximum flood depth between the 'with scheme' and baseline scenarios – breach at tank traps: 2030 epoch

Sample Location	Difference in max Depth (m) – 2030 epoch	
	1 in 200-year	1 in 1,000-year
Eastbridge	0.02	0.04
Minsmere (North)	0.02	0.04
Minsmere (Central)	0.13	0.13
Minsmere (South)	0.18	0.21
Leiston	-0.16	-0.19
Sizewell	-0.16	-0.19

Table 6.6. Difference in maximum flood depth between the 'with scheme' and baseline scenarios – breach at tank traps: 2190 epoch

Sample Location	Difference in max Depth (m) – 2190 epoch	
	1 in 200-year	1 in 1,000-year
Eastbridge	0.00	0.00
Minsmere (North)	0.00	0.00
Minsmere (Central)	-0.01	-0.01
Minsmere (South)	-0.01	-0.01
Leiston	0.00	0.01
Sizewell	0.00	0.00

6.4.2. As shown in **Plate 6.16** – **Plate 6.18**, the adverse changes in flood levels (positive difference in flood depth) are mostly within the low-lying area of Minsmere marshes with maximum increase in most of the area up to 0.16m and 0.20m for the 1in 200-year and 1 in 1,000-year return period events at 2030 epoch respectively, with localised (near the SSSI crossing) higher difference up to 0.25m and 0.30m. Although the relative differences might seem significant, the actuall flood depths are in a range of 3.0-3.5m for the 2030 epoch and 5.5-6.0m for the 2190 epoch, and therefore the change in flood depth would not have significant imapct on already severly flooded areas.

6.4.3. Overall difference for the more extreme scenarios (2190 epoch) is lower as the system is severly inundated and therefore the development has less pronounced impact of flood levels. For that reason results for flood velocity and hazard below are presented for 2030 epoch only, whereas additional figures for 2190 epoch are provided in **Appendix C**.

6.4.4. **Plate 6.19** and **Plate 6.20** illustrate differences in flood velocities between the 'with scheme' and baseline scenarios with breach at tank traps for the 1 in 200-year and 1 in 1,000-year events at 2030 epoch respectively.

Plate 6.19. Difference in maximum flood velocity for the 1 in 200-year return period event at 2030 – breach at tank traps