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- 1: Document related to Unit 1
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NUCL/REP/EPR UKEPR							
HPC (doc: HK)				SZC (doc: SZ)			
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BUILDING **000**

SYSTEM **000**

SCALE	**DESCRIPTION**
SIZE **A4**	**SIZEWELL C COASTAL FLOODING ALARP PHASE 2 – FLOOD LEVELS ANALYSIS**
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DOCUMENT REFERENCE No

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Sizewell C Coastal Flooding ALARP Phase 2
Flood Levels Analysis

NNB GenCo

5134299/301/001
Issue 02
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Glossary

ALARP	As Low As Reasonably Practicable
AOD	Above Ordnance Datum
BLF	Beach Landing Facility
EA	Environment Agency
HAT	Highest Astronomical Tide
HPC	Hinkley Point C
IAEA	International Atomic Energy Agency
ISFS	Interim Spent Fuel Store
LiDAR	Light Detection and Ranging
MHWS	Mean High Water Springs
MOLF	Marine Off-Loading Facility
MSL	Mean Sea Level
ONR	Office for Nuclear Regulatio
OS	Ordnance Survey
ReFH	Revitalised Flood Estimation Hydrograph
SAPs	Safety Assessm nt Princi es
SMP2	Shoreline Manage en Plan 2
SWL	Still Wate Level
SZA	Sizew l A
SZB	Sizewell B
SZC	Sizewe
UKCP09	UK Climate Projections 2009

1. Introduction

Atkins is performing a study for NNB GenCo, which builds on a previous scoping study [1], to inform the selection of the Sizewell C (SZC) platform height and any associated external barriers. The study aligns with the development of the coastal flooding safety case to ensure that the nuclear safety risk from the hazard is acceptable and as low as reasonably practicable (ALARP). This main phase of the work is entitled 'SZC Coastal Flooding ALARP Phase 2'. An updated basis of assessment for the SZC Coastal Flooding ALARP Phase 2 study [2] was issued in September 2014 to capture the key areas that had been developed or confirmed since the scoping study and following the production of NNB GenCo's work specification [3] and Atkins' offer [4] for the main phase of work.

This report presents an analysis of extreme flood levels adjacent to the SZC site in order to inform the selection of appropriate platform level options and associated external barriers for the ALARP assessment recognising the protection required against the coastal flooding hazard and accounting for climate change up to the credible maximum. It should be noted that the present flood levels analysis is undertaken specifically to support the ALARP assessment and associated decision-making on the SZC platform height. NNB GenCo / EDF will present further analysis of coastal flooding and coastal change as necessary to support the development and substantiation of the formal coastal flooding safety case.

The main focus of this report is the prediction of the still flood water levels which could arise on the lower-lying land (known as Sizewell Belts) adjacent to the north and west of the SZC site as a result of tidal overflow and overtopping of the shoreline dune/bank frontage to the north and south of the Sizewell power station sites (A, B and C). The still flood water levels to the north and west of the SZC site are analysed using a TUFLOW model. Sea conditions (still water levels and waves) with a 10 000 year return period are taken as input to the flood levels analysis accounting for the joint probability of the phenomena. The performance of the modelled barriers is described in relation to potential platform levels.

A second objective of this report is to inform the selection of the SZC platform level which would satisfy the 'dry site concept' (i.e. without any external barriers). As described in the basis of assessment report [2], the SZC platform level would have to be above the level of significant wave run-up for the 'dry site concept' to be fulfilled. The definition of 'dry site concept' was carefully reviewed at the basis of assessment stage [2] to ensure consistency with International Atomic Energy Agency (IAEA) safety guide IAEA-SSG-18 [5] and the consultation version of the new Safety Assessment Principles (SAPs) [6A] being developed by the Office for Nuclear Regulation (ONR)[1]. The original scope of this study was to define a single dry site platform level above significant wave run-up for credible maximum climate change. Further to a workshop in November 2014, NNB GenCo requested that Atkins also define a dry-site platform level for reasonably foreseeable climate change [7]. The platform level for this additional option is evaluated in the present report but not reflected in the overall options tables to reduce editing.

The order of the report reflects the driving direction of the coastal flooding process and associated wave propagation from offshore to onshore culminating in the overland flood modelling. This allows the coastal inputs and boundary conditions for the overland flood modelling to be progressively defined from sea conditions through shoreline geomorphology to tidal overflow and overtopping into the onshore flood pathways. While the overland flooding is strongly driven by sea conditions, the modelling also takes reasonable account of coincident fluvial flows and high groundwater conditions.

[1] Issue 02 update: the formal version of the 2014 SAPs has now been published [6B]. The clauses on the dry site concept are the same as in the consultation version.

2. Philosophy and Basis of Assessment

This section sets out the basis of assessment for the flood levels analysis and the underpinning nuclear safety principles for coastal flooding protection at SZC.

2.1. Coastal Flooding Safety Case Principles

As set out in the scoping study report [1], the basis of assessment for the flood level analysis has followed a clear set of safety case principles for coastal flooding protection, consistent with those adopted for Hinkley Point C (HPC). The principles are in line with the following regulatory guidance and requirements published by the ONR, the Environment Agency (EA) and the IAEA:

[5] Meteorological and Hydrological Hazards in Site Evaluation for Nuclear Installations
International Atomic Energy Agency, Specific Safety Guide SSG 18, November 2011

[8] Safety Assessment Principles for Nuclear Facilities[2]
Health & Safety Executive, 2006 Version, Revision 1 (updated February 2008)

Including the latest consultation version:

[6A] CI's Fukushima Reports Rec. IR-5: SAPs Review: Response Form
SAPs Text Editor, Civil Engineering and External Hazards, March 2014
http://www.onr.org.uk/consultations/2014/saps/civil_engineering_and-external-hazards-text.pdf

[9] Technical Assessment Guide – External Hazards
Health & Safety Executive, T/AST/013 Issue 4, July 2011

[10] Joint Advice Note
Principles for Flood and Coastal Risk Management
Office for Nuclear Regulation and Environment Agency, Version 3, April 2013

The principles collated here are also aligned with NNB GenCo's interpretation of the regulators' Joint Advice Note [10] as documented in [11].

The key principles used to define the basis of assessment for the present flood levels analysis are as follows:

1. Design basis coastal flooding events should be conservatively predicted at a return period of 10,000 years allowing for uncertainty.
2. The design basis coastal flooding conditions should account for all phenomena contributing to high sea water levels and waves (including appropriate combinations of high astronomical tides, storm surge and waves) with rational treatment of joint probabilities. A similar approach should be taken to account for interactions with other forms of flooding (pluvial, fluvial and groundwater).
3. All potential mechanisms and pathways by which the coastal flooding hazard could affect the site should be identified and evaluated in order to demonstrate the sufficiency and adequacy of the coastal flooding protection.
4. The effects of "reasonably foreseeable" climate change (see definition in Section 2.2) over the lifetime of the station should be included in the design basis for the original build of the coastal flooding protection in a precautionary manner.
5. The effects of "credible maximum" climate change (see definition in Section 2.2) over the lifetime of the station should be evaluated for the design of the coastal flooding protection and should either (1) be included in the design basis for the original build in a precautionary manner, or (2) be shown to be feasibly accommodated using a managed adaptive approach.
6. The effects of coastal change (offshore, inshore and onshore) on the coastal flooding hazard should be identified and evaluated. Coastal change may be long-term or sudden (e.g. progressive or driven by the extreme coastal flooding event itself). The coastal change scenarios and erosion allowances should account for the effects of climate change at the appropriate level (reasonably foreseeable or credible maximum).

[2] Issue 02 update: the formal version of the 2014 SAPs has now been published [6B]. The clauses relevant to coastal flooding and the dry site concept are the same as in the consultation version.

2.2. Climate Change Scenarios and Allowances

The following three climate change scenarios for sea conditions are considered in the food levels analysis:

- Present-day
  This case is taken as a baseline against which the present-day margins in the original build of the coastal flooding protection can be gauged.
- Reasonably foreseeable climate change (to end of station lifetime)
  This is taken to define a standard level of climate change which is moderately likely to occur during the lifetime of the station. The intent is that this level of climate change should be included in the design basis for the original build of the coastal flooding protection.
- Credible maximum climate change (to end of station lifetime)
  This is taken to define a higher level of climate change beyond the likely range but within the limits of physical plausibility. The intent is that this level of climate change should be considered when planning and designing the coastal flooding protection and it should be shown that it would be feasible to maintain nuclear safety as this scenario develops by enhancing the coastal flooding protection in a timely manner through a managed adaptive approach.

The flood levels analysis applies the allowances for the end-of-life timescale in a deterministic manner without consideration of different climate change scenarios or intermediate timescales. The ALARP assessment will consider the probability of different climate change scenarios and the associated increases in extreme sea level with time.

The climate change allowances adopted for this study are applied over a nominal timescale from 2008 (baseline hydrological datum) to 2110. This is sufficient to cover the lifetime of the main SZC site including generation and decommissioning. The Interim Spent Fuel Storage (ISFS) is the only facility with a longer lifetime and it will be converted to operate autonomously following the end of generation. The safety case for the ISFS will contain specific arguments to justify its protection against coastal flooding accounting for the greater period over which climate change effects should be considered and the effects of decommissioning/clearing of the main SZC site. It is noted that the SZA and SZB safety cases will ensure adequate protection of the SZA and SZB sites from coastal flooding over their appropriate lifetimes which may be exceeded by the 2110 end-of-life climate change timescale adopted in this report for SZC.

The climate change allowances adopted in this study, as agreed by NNB GenCo at the basis of assessment stage [2], are listed in Table 1. The derivation of the allowances is summarised below and detailed in the scoping study report [1].

Climate Change Scenario	Relative Mean Sea Level Rise	Storm Surge Increase	Offshore Wave Height Increase
Reasonably foreseeable (2008 to 2110)	+0.75m	0m	+10%
Credible maximum (2008 to 2110)	+2.12m	+1.0m	+10%
Note: All allowances are relative to present-day conditions (nominally 2008).			

Table 1. Climate Change Allowances

According to ONR guidance [9], reasonably foreseeable climate change may be represented by projections for the medium emissions scenario at a reasonably high (84%) confidence level. The reasonably foreseeable climate change allowances adopted for this study are derived from the UK Climate Projections Science Report (2009) (UKCP09) [12] using data appropriate to the SZC site location. Allowances are considered for mean sea level rise, storm surge increase and wave height increase. The allowances adopted here are greater than or equal to the ‘change factor’ allowances given in the EA’s current guidance on climate change with respect to flood and coastal risk management [13].

In keeping with the approach applied for HPC, the credible maximum allowances adopted here are aligned with the upper H++ estimates for mean sea level rise and storm surge given in UKCP09 [12] and the associated allowances in the EA guidance [13]. A recent scoping paper focusing on the SZC location [14] found very similar values for the increase in extreme still water levels at the upper end of the credible maximum range (3.20m from 1990 to 2100 [14] compared to 3.12m from 2008 to 2110 [1]).

As observed in the ALARP scoping study [1] and basis of assessment report [2], the selection of platform level is significantly influenced by the top-end climate change and coastal change scenarios (i.e. credible maximum) because the platform level would not be adaptable through the SZC lifetime.

2.3. Dry Site Concept and External Barriers

As described in IAEA safety guide IAEA-SSG-18 [5], coastal flooding protection for a nuclear site may be achieved through the following two approaches (or a combination of these):

- dry site concept
- permanent external barriers

These two approaches are recognised in the consultation version of the new SAPs [6] being developed by the ONR. As described in the basis of assessment report [2], the ONR indicates a preference for the dry site concept [6, 10] in stating that protection should be provided through the dry site concept where reasonably practicable. The approach being taken in the present study satisfies this intent since a dry site platform level option is to be identified and its practicability assessed through the detailed ALARP assessment.

The definition of dry site concept was carefully reviewed at the basis of assessment stage [2] to ensure consistency with IAEA safety guide IAEA-SSG-18 [5] and the consultation version of the new SAPs [6] being developed by the ONR. It was concluded and agreed with NNB GenCo that the SZC platform level would have to be above the level of significant wave run-up for the dry site concept to be fulfilled.

In order to inform the dry site platform level, direct wave run-up levels on the seaward slope to the east of the SZC site are calculated in Section 5.3 using established empirically-based methods. The geometry and roughness of the seaward slope and toe are assumed to be similar to those of the preferred concept design option for the sea protection embankment (Option 2.1 in [15]).

For lower platform level options (below the dry site level), external barriers are required to supplement the protection provided by the platform level. The extent and size of the external barriers increases with climate change and with reducing platform level. The four options to be analysed in the ALARP assessment are outlined in Section 2.5.

2.4. Coastal Flood Pathways for SZC Site

A total of five potential coastal flood pathways were identified for the SZC site in the scoping report [1] and at the basis of assessment stage [2]. These are listed in Table 2.

Pathway ID	Pathway Description
1 'northern'	Overflow / overtopping of the coast to the north of SZC (Minsmere frontage) leading to overland flooding to the north-west and west of the SZC site (Sizewell Belts).
2 'southern'	Overflow / overtopping of the coast to the south of SZA (Sizewell Gap) leading to overland flooding to the west and north-west of SZC site (Sizewell Belts).
3	Overtopping onto the SZA or SZB sites leading to surface flooding onto the south of the SZC site via SZB.
4	Direct overtopping onto the east or north-east of the SZC site.
5	Flooding of the SZC site platform: A. Via the cooling water intake/discharge tunnels which link the offshore heads to the forebays, pumping stations and discharge ponds. B. Via the fish return routes and tunnel which connect the debris recovery buildings and fish lifts to the outfall below low tide level.

Table 2. Coastal Flood Pathways

The flood levels analysis described in this report addresses Flood Pathways 1, 2 and 4 which drive the options for platform level setting and future external barrier provision. Flood Pathways 3 and 5 can be addressed by smaller barriers or other measures at a local level within the SZC site or along the boundary with the SZB site.

The impacts assessment and ALARP analysis will take direct account of Flood Pathways 3 and 5 and any associated barriers.

For Flood Pathway 4, direct wave run-up levels on the seaward slope to the east of the SZC site are calculated in Section 5.3 using established empirical methods. No other flood or overtopping analysis is required for this pathway since the ongoing design of the sea protection embankment [15] is to provide protection against sea conditions of 10,000 year return period with appropriate climate change allowances.

The main focus of the flood levels analysis is the prediction of still flood water levels which could arise on the lower-lying land (Sizewell Belts) to the north and west of the SZC site as a result of the overland flood pathways (1 and 2). For simplicity (as shown in Figures A.1.1 and A.1.2), these are called the 'northern' flood pathway and the 'southern' flood pathway respectively in this report.

- The northern flood pathway involves tidal overflow/overtopping of the Minsmere frontage stretching north from SZC for about 3km to Minsmere Cliffs with resultant overland flooding of the Minsmere catchment through to Sizewell Belts via the valley between Goose Hill and the SZC site. NNB GenCo have confirmed [16] that a variant of this pathway (identified as Flood Pathway 1A in the scoping study report [1]) involving flow from the Minsmere catchment into Sizewell Belts via a saddle point further west along Goose Hill will not be viable due to the permanent smoothing of the developed topography in this area along the line of the permanent access road.
- The southern flood pathway involves tidal overflow/overtopping of the Sizewell Gap frontage (about 200m long) to the south of the SZA site with resultant overland flooding of Sizewell Belts.
- Although the width of the valley between Goose Hill and the SZC site is comparable to the width of Sizewell Gap, the potential floodwater conveyance via the northern pathway is significantly greater primarily due to the lower topography across the valley (0.5m AOD to 1m AOD) than along the crest of the dune/bank at Sizewell Gap (approximately 6m AOD present-day), even allowing for possible erosion. An additional factor is the gentle rise in topography above the platform level (and bridge deck level) at the eastern end of Goose Hill which increases the wetted valley width at the highest flood levels.

2.5. Initial Option Definition – Platform Levels and Barriers

As indicated in the basis of assessment report [illegible], the flood levels analysis is to inform the selection of options for ALARP assessment each consisting of a defined platform level and a consistent set of external barriers. The platform levels covered by the options range from 6.4m AOD through to 15m AOD. An outline of the options as they were defined at the basis of assessment stage [2] leading into the flood levels analysis is given in Table 3. The position and characteristics of the overland flood barriers are summarised in Table 4 and Figures A.1.3 & A.1.4 accounting for NNB GenCo's preferences obtained at the basis of assessment stage [2, 16]. It should be noted that, as requested by NNB GenCo, it has been necessary to account for certain openings in the northern barrier.

The flood levels analysis is to assess the performance of different flood barrier configurations across the northern and southern flood pathways in limiting the still water levels to the north and west of the SZC site.

Option No.	**Option 1**	**Option 2**	**Option 3**	**Option 4**
Platform Level	6.4m AOD	~7.5m AOD	~8.5m AOD	~15m AOD
Title	No change (as SZB)	Intermediate lower	Intermediate higher	Dry site concept
External Barriers (for credible maximum climate change to 2110)	Sea protection embankment required to east of SZC. Northern barrier expected to be required with limited openings - to be investigated through flood levels analysis. Southern barrier expected to be required - to be investigated through flood levels analysis.	Sea protection embankment required to east of SZC. Northern barrier expected to be required but greater openings may be permitted. Requirement for southern barrier to be investigated through flood levels analysis.	Sea protection embankment required to east of SZC with extended wave barrier to north to limit wave transmission towards northern edge of SZC site. Northern barrier not required. Southern barrier not required.	None required.

Table 3. Options Identified from Basis of Assessment [2]

Flood Barrier	Location / Alignment	Openings	Crest Level
Northern	Across valley to east of access road / bridges, from south-east end of Goose Hill to Northern Mound (north-east of SZC site).	1. V-notch opening for fluvial channel. Invert of opening to suit fluvial channel. The assumed size is 3m wide at invert with 60º side slopes (see Appendix A, Figure A.1.4). 2. Rectangular notch opening for Beach Landing Facility (BLF) road near abutment to Northern Mound. Assumed road width of 10m (based on [17]). 3. The shoulder of Goose Hill would remain open above the level of the platform (and access road / bridges) unless the crest of the barrier is specifically extended. The alignment of openings would have to be carefully designed to avoid high velocity downstream flows being directed towards the northern edge of the SZC site with limited freeboard.	The crest level of the barrier would have to be sufficiently high to prevent direct overflow and to limit wave transmission towards the northern edge of the SZC site. A minimum crest level of about 10m AOD is estimated for credible maximum climate change.
Southern	Across Sizewell Gap, e.g. along line of highest (landward) dune/bank.	None.	The crest level of the barrier would have to be sufficiently high to prevent direct overflow and to avoid excessive overtopping volumes. A minimum crest level of about 9m AOD is estimated for credible maximum climate change. A moderate degree of wave overtopping is likely to be acceptable providing sufficient scour protection is provided on the landward slope.

Table 4. Overland Flood Barriers

2.6. Extreme Still Water Levels

The extreme still water levels applied in the flood levels analysis are based on a present-day level of 5.20m AOD at a 10,000 year return period and 95% confidence level. This value was provided by NNB GenCo at the basis of assessment stage [2]. It is slightly higher than the corresponding value derived previously in Atkins' scoping study [1] from the extreme water levels analysis in [18]. The increased value was obtained from an updated statistical analysis [19] accounting for the high still water levels observed during the storm on 5th December 2013.

The extreme still water levels for each climate change scenario are listed in Table 5.

Present Day (2008)	Reasonably Foreseeable Climate change (2110)	Credible Maximum Climate change (2110)
5.20m AOD	5.95m AOD	8.32m AOD
Note: All water levels are at 10,000 year return period and 95% confidence level.		

Table 5. Extreme Still Water Levels

It is evident that the lowest platform level option (6.4m AOD) exceeds the extreme ill water level in present-day and reasonably foreseeable (2110) conditions. As a result, the analysis of overla d flood levels with external barriers is limited to credible maximum (2110) conditions (Section 8 .2).

3. Sea Conditions

This section of the report sets out and develops the sea conditions (s ll water leve and waves) adopted for the flood levels analysis.

3.1. Tide Curves

For each climate change scenario, a tide curve has been gener d as an input to the flood levels analysis to define the time-varying still water level with the passage of the associated storm surge event and astronomical tide cycles. The construction of the tide curves follo s the p cedure and guidance given in the EA's Coastal Flood Boundary Conditions project [20]. The tide c rves are pre ented in Figure 1. The peak of each tide curve is anchored to the corresponding extreme still wa er level listed Table 5.

In order to define the relative contributions of astr omical tid and storm surge within the tide curves, it is assumed that the present-day extreme still r level 5 0mOD is made-up of an astronomical high tide of 1.50mOD in conjunction with a peak skew surge f 3.70m. This magnitude of astronomical high tide is between Mean High Water Springs (MHWS) an Highes Astronomical Tide (HAT). As summarised in Table 3 of [19], local data on astronomical tides for SZC indica s that Mean Sea Level (MSL) is 0.16mOD, MHWS is 1.22mOD and HAT is 1.68mOD. Taken in isolation a kew surge of 3.70m at SZC is estimated to have a return period of about 5,000 years (interpolatin tween alues in Table 11c of [19]). Hence, an astronomical high tide of 1.50mOD and a skew surge of 3.7 m are c nsidered (individually and collectively) to be reasonable contributors to the extreme st water l el event having a joint return period of 10,000 years.

The astronomical tide base c v has been generated using Atkins' TIDSIM computer programme[3]. Astronomical tide curve f approp iate magnitude were generated for Minsmere Sluice as the closest standard port to SZC The ad pted as nomical tide base curve was obtained with minor translation of the best-match curve from Minsm re Sluice to give a peak of 1.50mOD and an MSL close to 0.16mOD (as shown in Figure 1).

In accordance with [20], th normalised design surge shape for Lowestoft was adopted for SZC and scaled to give a peak s ge magnitude o 3.70m for the present-day and reasonably foreseeable climate change cases. For the credib e maxim climate change case, the normalised design surge shape was scaled to give a peak surge magnitude of 70m in accordance with the allowance specified in Table 1. It is noted that the design surge shape is b ad with an overall duration of about 40 hours (as shown in Figure 1).

The overall tide curv s were then assembled as a simple summation of the astronomical tide curve and the corresponding design surge curve while incorporating increases in relative mean sea level for the reasonably foreseeable case (+0.75m) and the credible maximum case (+2.12m). For convenience and clarity, the peak of the design surge curve was taken to occur at the mid-point of the astronomical tide sequence and coincident with the largest astronomical high tide. This creates tide curves which are almost symmetrical about the central peak.

[3] TIDSIM is a computer programme developed by Atkins to predict astronomical tide levels at any standard port at any time of interest using tidal constituents published by the Admiralty.

The tide curves are defined over a duration of 96 hours covering eight astronomical tide cycles. This provides an ample duration (about 28 hours) of lead-in and tail-off before and after the storm surge. From inspection of the tide curves, the dominant contribution of the storm surge is evident.

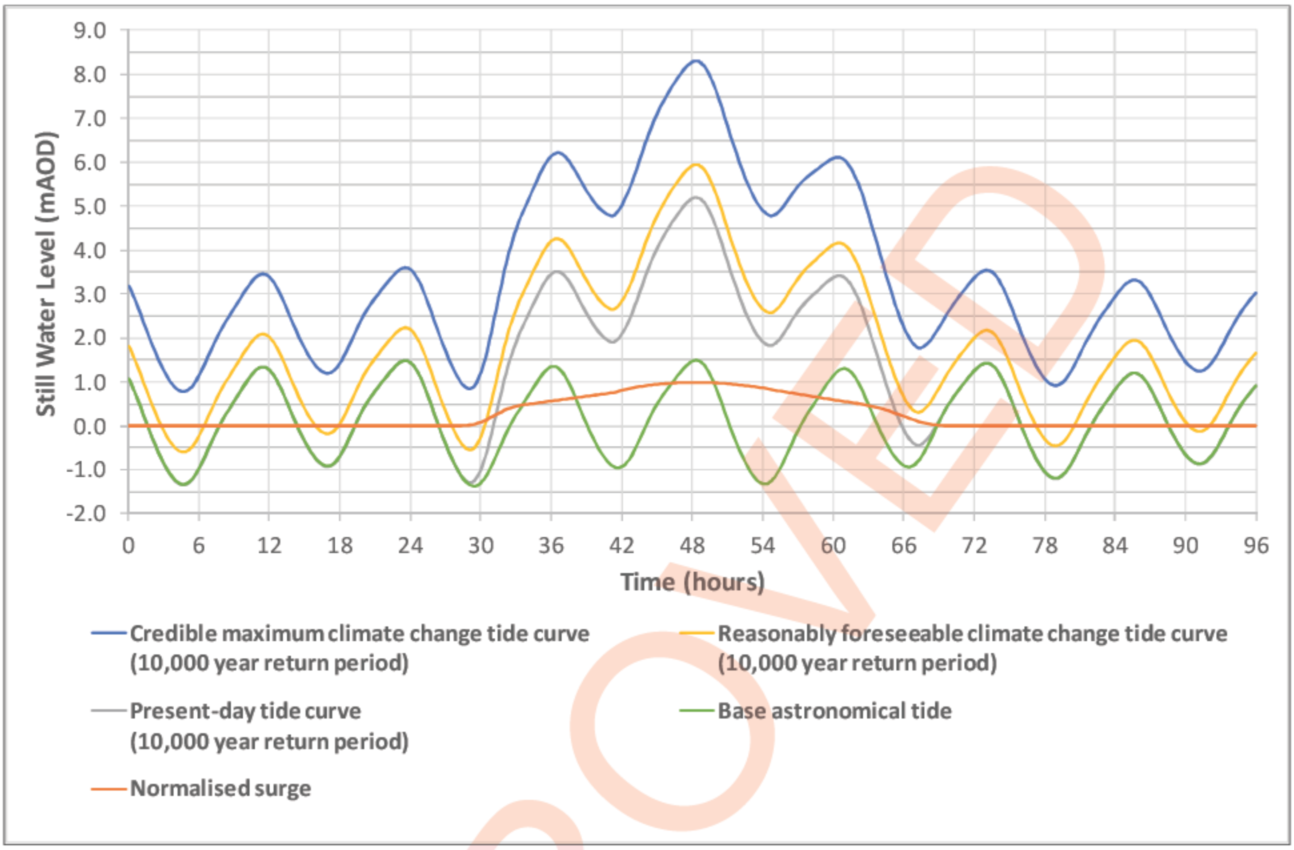


Figure 1. Tide Curves

3.2. Joint Probability of Offshore Waves and Still Water Levels

A joint probability analysis of offshore waves and still water levels has recently been performed by CEFAS for the SZC site [21]. The analysis covers two offshore wave direction sectors (S1 from north / north-east and S4 from south / south-east). The joint probability data has been used by CEFAS to define the offshore boundary condition in their TOMAWAC modelling of wave transformations to the nearshore as described in Section 3.3. Offshore waves are defined at the TOMAWAC model boundary approximately 10km east of the coastline [21].

The results of the joint probability analysis at 10,000 year return period for the two wave direction sectors and the three climate change scenarios being applied in this study are shown in Figure 2. The wave height values and still water level values are both given at 95% confidence level. The axis-crossing still water levels match the extreme still water levels specified in Table 5. It is apparent that offshore wave heights from the north / north-east bound those from the south / south-east over the full range of still water levels.

It should be noted that the joint probability curves in Figure 2 have been constructed by Atkins from a limited number of data points provided by CEFAS. The original labelling of the data points assigned by CEFAS is clearly marked for later reference since these points define the specific joint probability combinations which were run through the TOMAWAC model.

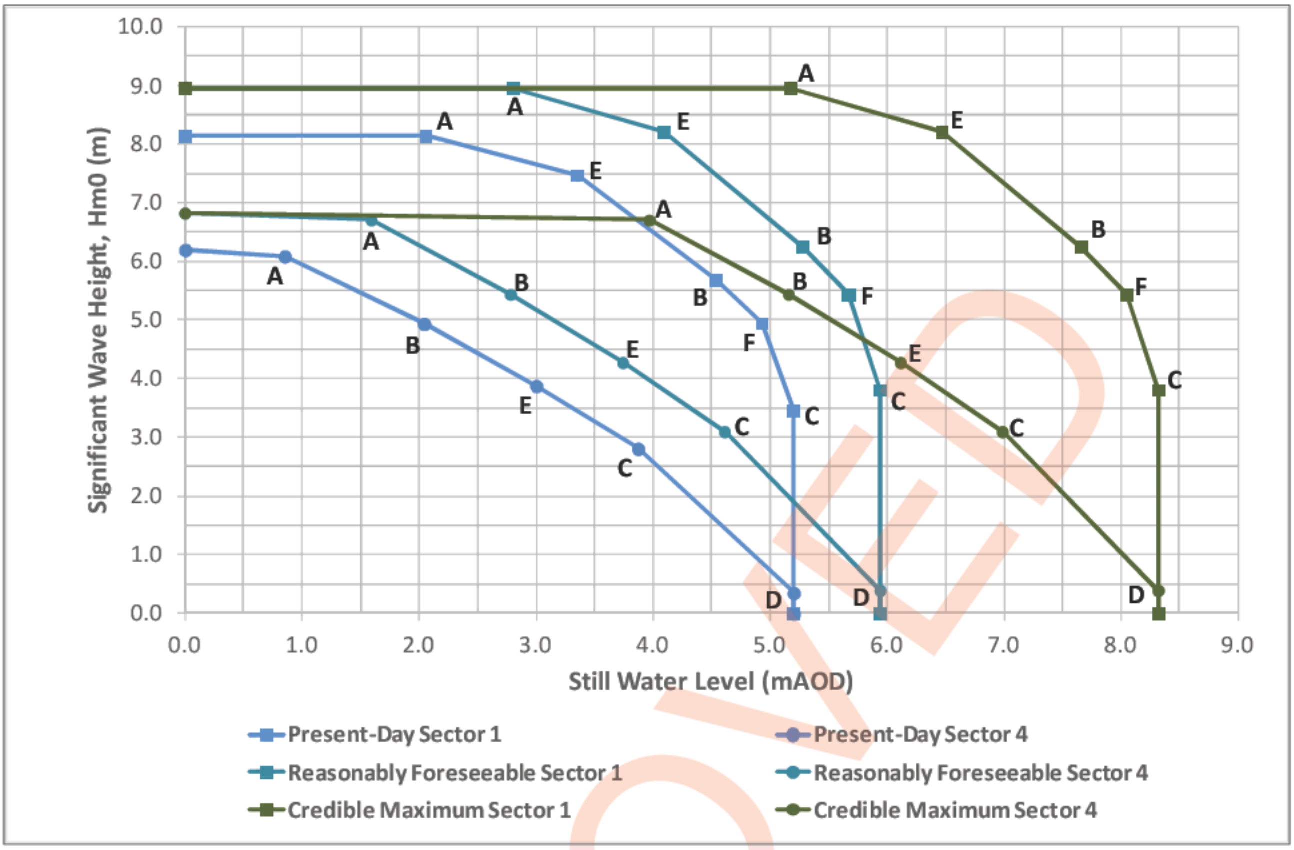


Figure 2. Joint Probability of Offshore Waves and Still Water Levels (10,000 year return period)

3.3. Nearshore Waves and Offshore Geomorphology

CEFAS has provided Atkins with output wave data from their TOMAWAC model [21] for the joint probability cases of offshore waves and still water levels defined in Section 3.2 and identified in Figure 2. The TOMAWAC model set-up and calibration are described in CEFAS report TR232 [22].

The output wave data is provided along eight transects extending due east from the shoreline of interest (from north Minsmere south to Sizewell Gap) for several kilometres. This has allowed Atkins to inspect and review the wave transformations through the nearshore zone and the predicted depth-limited wave behaviour at the shoreline. The shoreline location of the output transects is shown in Figure 3.

The output wave data from the TOMAWAC model [21] has been provided for both offshore wave directions (S1 and S4) and for two bathymetry scenarios (one present and one long-term). The second bathymetry scenario has been considered by CEFAS in their TOMAWAC modelling in recognition of the geomorphological changes that could occur to the offshore sandbanks (the Dunwich-Sizewell Banks) over the lifetime of SZC. The sandbanks run approximately parallel to the coastline about 1km to 2km from the shore with a crest level of -8m AOD to -6m AOD [21]. The second bathymetry scenario represents significant depletion and flattening of the sandbanks with the material being redeposited into the shoreward trough (the bottom of which is currently at about -11m AOD).

3.3.1. Factors affecting nearshore wave conditions

By inspection and comparison of the predicted significant wave heights along the TOMAWAC output transects for the various cases, it is found that:

- Nearshore wave conditions are more severe for offshore wave direction S1 (from north / north-east) than for S4 (from south / south-east). Nearshore wave conditions for offshore wave direction S1 (from north / north-east) only are taken forward in the assessment.
- Offshore waves from direction S1 (from north / north-east) refract to near-normal incidence approaching the shoreline (within about 10°). Therefore, no adjustments for wave obliquity are applied in the assessment.

- Inshore wave conditions are similar or more severe for the case with present-day bathymetry than for the case with depleted sandbank bathymetry and material redeposited shorewards. With the modified bathymetry, the greater nearshore wave heights passing the present location of the sandbanks are more than offset by the effect of the reduced water depths where the material has been redeposited in the shoreward trough and up towards the foreshore. Inshore wave conditions with present-day bathymetry only are taken forward in the assessment.

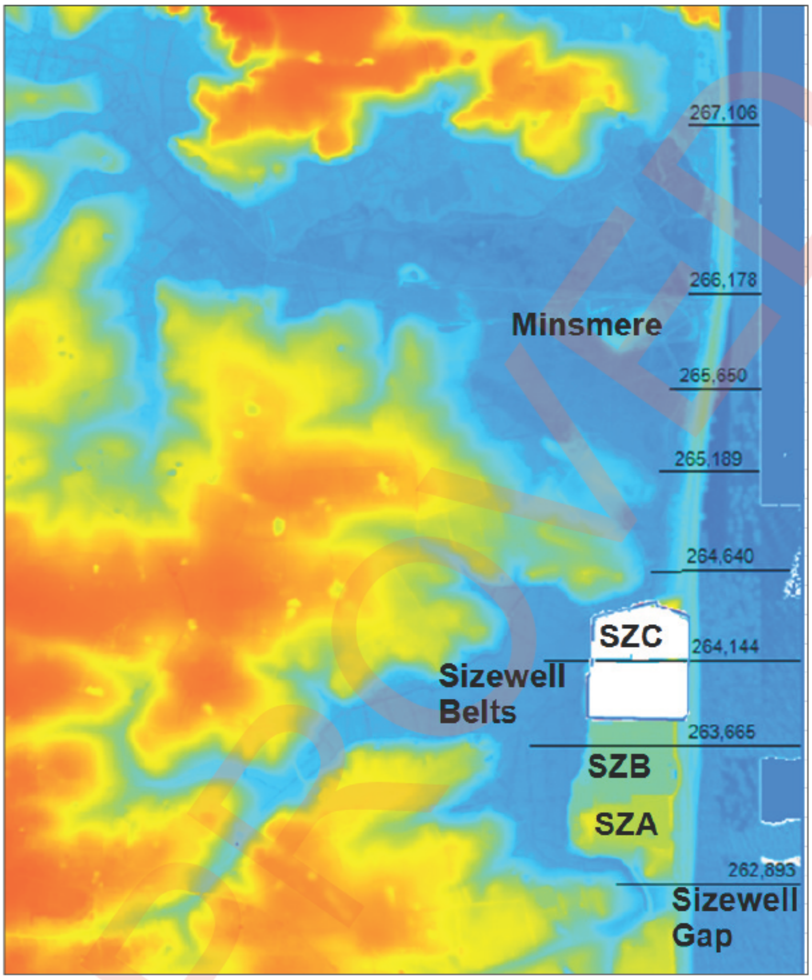


Figure 3. Transect Locations (OS National Grid Northings, overlaid on LiDAR topography map)

3.3.2. Wave conditions at shore

An understanding and evaluation of wave conditions at the shore is required to inform the assessment of (i) the storm response of dunes/banks along the Minsmere frontage and at Sizewell Gap (Section 4.2); (ii) run-up levels on the seaward slope east of the SZC site (Section 5.3); and (iii) overtopping volumes contributing to overland flooding (Sections 5.1 and 5.2).

The shoreline is at the very limit of the TOMAWAC model in terms of coverage and potentially with respect to the validity of the output wave conditions. The model area extends to between -1m AOD and 0m AOD in the present-day bathymetry case and the output wave conditions at the shore are sensitive to the particular formulation invoked for bathymetric breaking dissipation in the TOMAWAC model set-up (Roelvink's model (1993) [22]).

Two further methods, Goda (used in BS6349: 2000 [23]) and Van der Meer (used in EurOtop Manual, 2007 [24]) have been applied for the present study to obtain at-shore wave conditions above 0m AOD and to provide an independent calculation of at-shore wave conditions below 0m AOD against which the TOMAWAC outputs can be reviewed and the most robust values selected.

As detailed below, cross-shore elevation profiles for the present-day have been constructed using a combination of bathymetry data from the TOMAWAC model [21] and topography data from the LiDAR survey provided by NNB GenCo [25].

- TOMAWAC model bathymetry data extracted along the eight output transects.
  This covers the bathymetry below 0m AOD. Bathymetry data is provided at 5m easting intervals within about 1km of the shoreline (to easting 648,595) and at 50m easting intervals to the east of this point. The bathymetry data is shown in Figure 4 and 0.
- Sections taken from the LiDAR survey data [25] provided by NNB GenCo aligned with the eight output transects.
  This covers the topography above about 0.5m AOD. Ground elevation data below this level is not available from the LiDAR survey data [25] as it appears to have been flown close to high water.

The gap between the topography and bathymetry data sets around 0m AOD is small. There is generally good consistency between the two data sets in terms of foreshore level and gradient. On certain transects, there appears to be a mismatch of up to 50m in the cross-shore (easting) position of the foreshore between the two data sources. In such cases, the upper foreshore defined by the LiDAR survey is generally further seaward than the lower foreshore defined by the bathymetry data. On first inspection, this discrepancy is most likely due to some inaccuracy in the spatial positioning of the bathymetry data points. The positioning of the LiDAR data along the foreshore appears to be accurate and it does not seem credible for such large differences to be attributable to actual shoreline recession between the LiDAR survey date and the bathymetry survey date. Such discrepancies are not apparent in the detailed analysis of shoreline profile variability presented in [26].

As shown in Figure 4 and Figure 5, the average bathymetric slope from offshore to onshore is very gentle (about 0.004) and the typical inshore slope is steeper at about 0.02. The bathymetric profiles are reasonably consistent across all transects.

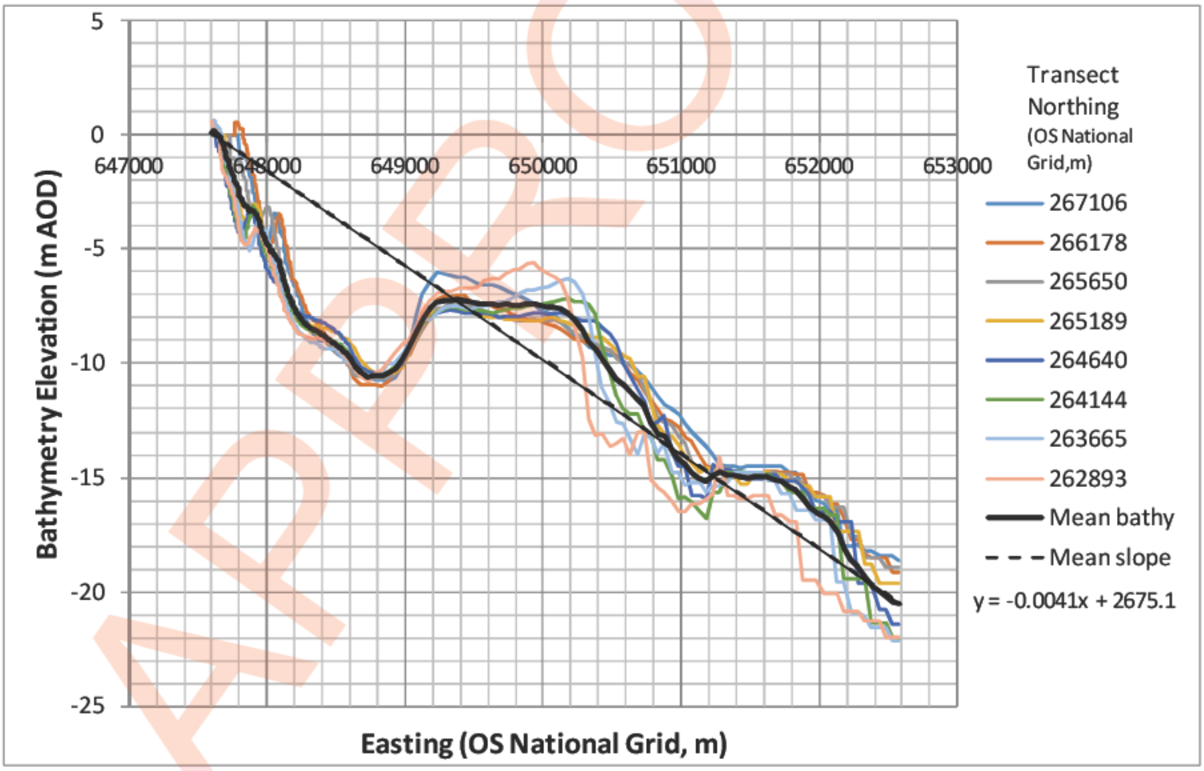


Figure 4. Bathymetry – Offshore to Inshore

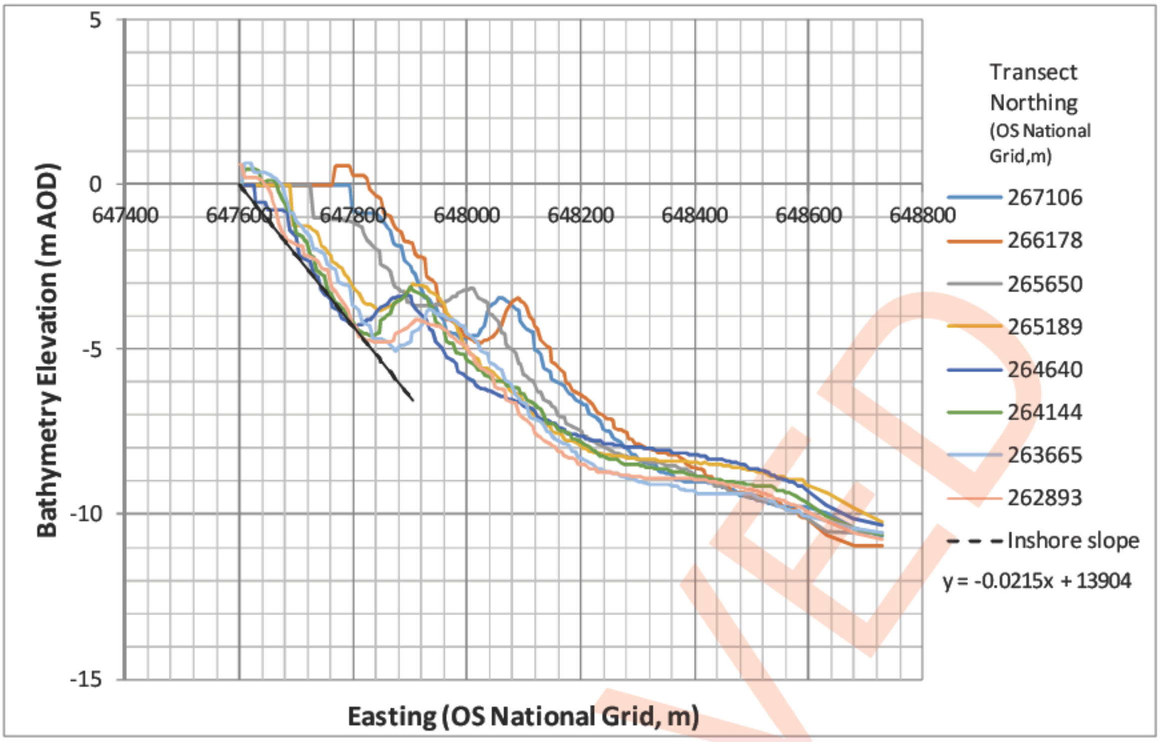


Figure 5. Bathymetry – Inshore

The review of at-shore wave data from the TOMAWAC model and comparison with other methods [23, 24] finds that:

- The TOMAWAC model has been set-up to calculate depth-limited wave heights according to Roelvink's model (1993) [22].
- The TOMAWAC wave heights approaching the foreshore (in water depths of 10m and greater) are in reasonable agreement with wave heights calculated using the Goda and Van der Meer methods. The TOMAWAC model accounts for the effects of refraction and irregular bathymetry (e.g. over sandbanks) which are not accounted for in the breaker index graphs used in the application of the Goda and Van der Meer methods as these implicitly assume a plane slope. These effects would be expected to reduce the inshore wave energy intensity and it appears that the similarity in the wave heights may be due in part to the wind forcing included in the TOMAWAC model.
- The TOMAWAC output wave periods (T_{m0}) reduce from offshore (about 11s) to inshore (about 7s in 10m water depth and 6s in 7m water depth). Applying a multiplier of 1.1983 on T_{m0}, as suggested by CEFAS, the offshore peak wave periods are in reasonable agreement with those used in the previous analysis of extreme sea conditions by HR Wallingford [27]. Recently obtained information from the TOMAWAC model [28] indicates that peak wave periods (T_p) remain almost constant as the waves propagate inshore while the relative peak energy in that part of the spectrum is reduced. This is consistent with the expected flattening of the wave spectrum in increasingly shallow, depth-limited conditions as energy is transferred to shorter wave periods.
- In shallower water (depths less than 10m), there is increasing divergence between the depth-limited wave heights from the TOMAWAC model and from the Goda and Van der Meer methods. The variation in the ratio of significant wave height (H_{m0}) to local water depth (h) for reasonably foreseeable climate change is shown in Figure 6 along the nearshore section of the TOMAWAC output transects at SZC (T6) and at Sizewell Gap (T8). The pattern is similar for all transects and sea conditions. The TOMAWAC outputs show a general increase in the H_{m0}/h ratio as would be expected as the waves propagate into shallower water and over the nearshore bar. However, as the waves approach the foreshore, the H_{m0}/h ratio remains limited at 0.45 in all cases before reducing in very shallow water. This is contrary to the expected wave behaviour as the H_{m0}/h ratio should increase progressively in shallow water until wave breaking. In line with standard coastal engineering practice, the Goda and Van der Meer methods predict limiting H_{m0}/h ratios (breaker indices) of 0.6 to 0.7 even for the least steep category of slope (up to 0.01).

In view of these findings, at-shore wave conditions predicted by the Goda and Van der Meer methods have been adopted for the present study. The 0.01 slope case is adopted as it is lies between the gentle offshore to onshore mean slope (about 0.004) and the steeper inshore slope (0.02) characteristic of the actual bathymetry.

For a 0.01 slope, the significant wave heights reaching the foreshore (taken at MLWS -1.21m AOD) are calculated from the Van der Meer method to be up to 3.2m for present-day conditions, 3.9m for reasonably foreseeable climate change and 4.5m for credible maximum climate change. For comparison, the predicted wave heights would be about 25% higher for a 0.02 slope. In each case, the results are quoted for the joint probability combination of offshore waves and still water levels (A, E, B, F, C) giving the greatest inshore wave height. The most onerous joint probability combinations for inshore wave heights are found to be F and B towards the centre of the traces (for S1 wave direction) shown in Figure 2.

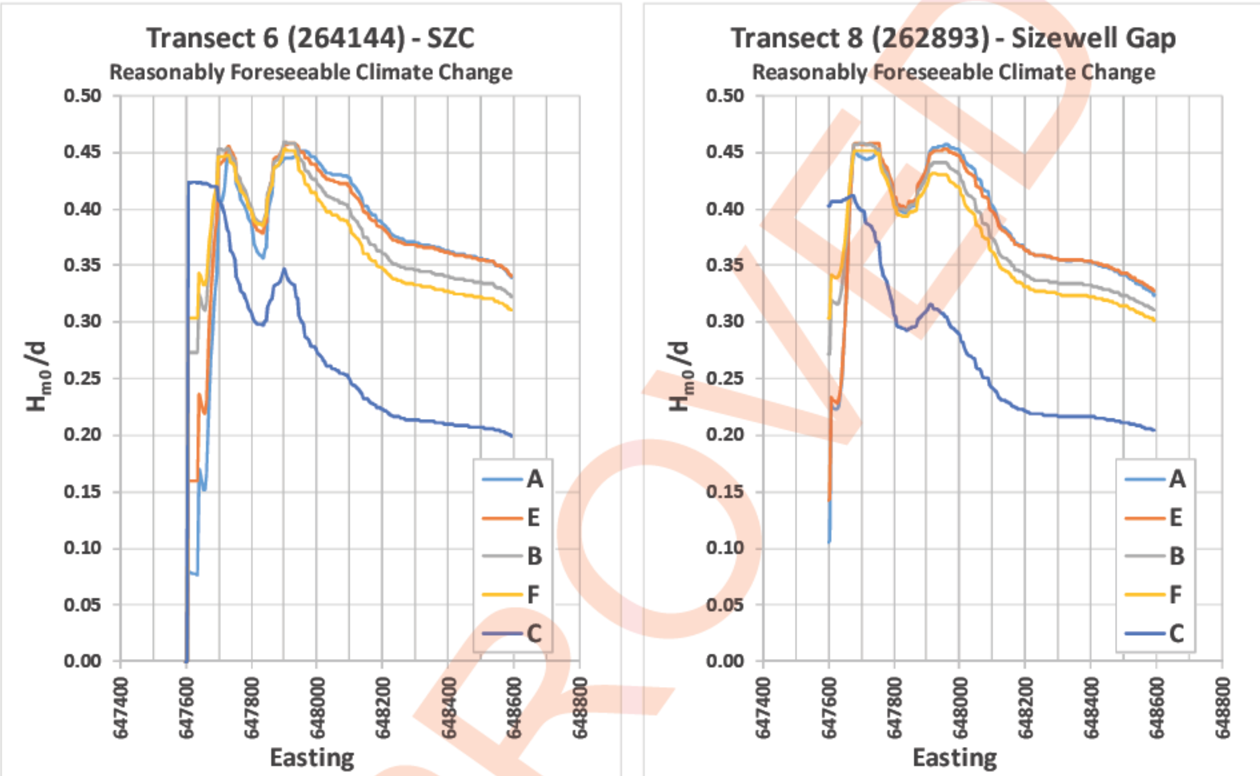


Figure 6. Nearshore Wave Height to Depth Ratios from TOMAWAC Model

4. Topography and Geomorphology of Shoreline Dune Banks

The topography of the shoreline banks along the Minsmere frontage and at Sizewell Gap dictates the overtopping and overflow volumes entering the northern and southern overland flood pathways under different sea conditions. The majority of the shoreline bank system along the Minsmere frontage consists of natural beach and dunes accompanied by longshore bars on the lower foreshore and some man-made earth embankments to the rear [26]. Minsmere Sluice acts as a 'hard point' within the dune system. At Sizewell Gap, there is a higher bank landward of the main dune bank which is understood to have been constructed or enhanced as part of the SZB development [7].

Beach and dune systems naturally exhibit variability on a range of time scales, with alternating periods of erosion and accretion due to variations in wind, wave and tidal conditions. They are also affected by human interventions in the coastal zone. The dune banks along the Minsmere frontage and at Sizewell Gap form soft sea defences which are subject to long-term geomorphological change and potential erosion or flattening during extreme storms.

This section of the report briefly reviews how the topography of the shoreline banks may change through long-term geomorphological trends and as a result of extreme storm response. Simple allowances for long-term and extreme storm-driven topographical change are derived to be applied as adjustments to the coastal boundary in the TUFLOW overland flood model (Section 4.3). The allowances for storm-driven topographical change account for extreme sea conditions at 10,000 year return period and different allowances are considered for each climate change scenario (present-day, reasonably foreseeable and credible maximum).

The present-day shoreline bank topography is visible on the LiDAR map shown in Figure A.1 and a selection of cross-shore profiles are shown in Figure 7. The highest bank along the Minsmere frontage has a general crest level of 4m AOD to 5m AOD increasing to 6m AOD locally south of Minsmere Sluice and decreasing to a minimum of 3m AOD locally along part of the northern Minsmere frontage. At Sizewell Gap, the two main banks are roughly parallel with the higher landward bank having a crest level of about 6m AOD and the seaward dune bank having a crest level of about 4m AOD.

The banks are understood to be generally well vegetated (down to typically 3m AOD on the seaward side) [26]. The sediment forming the dune banks and upper beach is generally gravel-dominated, overlain with wind-blown sand at the surface [26]. The sediment forming the lower beach (below about MLWS) and longshore bars is sand-dominated [26].

4.1. Long-Term Shoreline Change

The potential long-term changes to the dune banks over the timescale to 2110 have been reviewed with reference to the Shoreline Management Plan 2 (SMP2) for Suffolk [29], the coastal trends report for Suffolk produced by the EA [30], and CEFAS report TR223 on shoreline variability trends in Sizewell Bay [26].

The Suffolk SMP2 [29] identifies that the 'Minsmere floodplain' (PDZ4, MIN12.2-3) to the north of the SZC site is planned for managed realignment across all epochs. Detailed inspection indicates that the intention is to allow natural transition of cliffs/shingle banks (with the sluice remaining) rather than large-scale set-back of defences. The future estimates of shoreline change provided in Suffolk SMP2 (Appendix C Tables 3.13 and 3.14 of [29]) are wide-ranging with the following upper and lower projections of dune toe movement to 2105:

- North end of Minsmere frontage (near Minsmere Cliffs) (S1B1) — 10m to 75m recession in both scenarios
- South Minsmere frontage (south of sluice) (S1B4) — 10m to 26m recession in 'no active intervention' scenario; 26m to 77m recession in 'with present management' scenario
  (It is noted that the greater extent of erosion quoted in Suffolk SMP2 for the 'with present management' scenario is counter-intuitive and only occurs at four out of about 60 coastal chainage locations)
- Sizewell Gap (towards SZA) (S1B6) — 10 to 25m recession in both scenarios

It is noted that the SMP2 projections account for sea level rise of 1m (±20%) in line with Defra (2006) [31] which is the cause of some of the difference between the upper and lower projections.

The EA coastal trends report for Suffolk [30] identifies the following rates of shoreline change (between MLWS and MHWS contours) based on recent monitoring campaigns since 1991:

- North end of Minsmere frontage (near Minsmere Cliffs) (S1B1)
  Mean trend is 1.3m/yr erosion with slight beach steepening.
- North Minsmere frontage (north of sluice) (S1B2)
  Slight accretion trend to 2000 followed by a significant erosion trend of 3m/yr to 2010 with no beach rotation.
- Central Minsmere frontage (near sluice) (S1B3)
  Highly variable at all levels within a slowly accreting trend of 0.5m/yr with no beach rotation.
- South Minsmere frontage (south of sluice) (S1B4)
  Moderate erosion trends at all levels. An accelerated rate of erosion is evident from 2001, giving a mean erosion trend of 1.3m/yr.
- Sizewell Gap (towards SZA) (S1B6)
  Profile shows erosion to 1999/2000 followed by a period of accretion to 2010, resulting in no overall trend.

More detailed analysis of local shoreline change from a range of data sources is provided in CEFAS report TR223 [26]. It is evident that the patterns of shoreline change are complex both temporally and spatially with considerable variation between near-term and longer-term historical trends. Prior to 1925, there was persistent and spatially coherent erosion to the north of Minsmere Sluice and accretion to the south of Minsmere Sluice. This trend reversed during the period 1925 to 1940 and the shoreline change rates become more spatially and temporally variable. The shoreline trends since 1992 are summarised as follows with reference to [26] (from north to south):

- A strong erosional trend is apparent from the south of Minsmere Cliffs to the former Coney Hill, north of Minsmere Sluice (maximum erosion rate of 2.5 m/yr). As described in [26], part of this frontage was severely affected by storms in 2006 and 2007 when waves breached the frontal dune ridge and partially overtopped the secondary earth embankment behind. Improvements to this defence have since been made by the Environment Agency as part of the Minsmere Sea Defence Scheme.
- Minsmere Sluice continues to act as a 'hard point' and, while shoreline change rates fluctuate near the sluice, the net trend is accretional.
- A strong erosional trend is apparent from about 500m south of Minsmere sluice to just north of the SZC site (maximum erosion rate of 1.7 m/yr).
- The SZC site lies between an eroding area to the north and an accreting/stable area to the south. The central part of the SZC frontage currently experiences low shoreline change rates but the erosional zone over south Minsmere extends as far as the 'northern mound' directly to the north-east of the SZC site.
- There is seaward movement of the beach along the frontage of the existing power station sites (SZB, SZA) which is greatest in front of SZB (maximum 2.1 m/yr).
- The shoreline position at Sizewell Gap has been relatively stable although losses in beach volume occurred following storm events in 1993 and 1996.

Closer inspection of the shoreline change data (post-1992) for the Minsmere catchment frontage (Table 8 and Figures 13-16, 33, 34 of [26]) indicates mean recession rates in the order of 1 to 1.5 m/yr over a length of about 1km for each of the two erosion zones to the north and south of the sluice. The available shoreline profiles (S1B2 for north Minsmere, S1B4 for south Minsmere (Figures 33, 34 of [26])) both show about 20m of recession of the upper beach and seaward dune face from 1992 to 2007. The profiles indicate that the dune banks have experienced net erosion based on the observation that recession of the seaward face is not matched by roll-back on the landward side of the dunes. As would be expected, these recession rates are in close agreement with that in the EA coastal trends report for Suffolk [30] and in reasonable agreement with the upper projections from Suffolk SMP2 [29].

If the recession and erosion trends of the Minsmere shoreline experienced over the last 20 years were to continue for some decades into the future, the volume of the dunes would be substantially reduced and the residual dune banks would be less stable, generally lower and more susceptible to direct overflow, overtopping and breach in severe storms. It is acknowledged in CEFAS report TR223 [26] that under the present shoreline management strategy, erosion to the north of the SZC site could expose parts of the northern site boundary to the sea during the lifetime of the power station. As discussed in CEFAS report TR223 [26], the future pattern of shoreline change along the Minsmere frontage is uncertain as it is sensitive to several variable and interacting factors including offshore sandbank morphology, inshore wave climate and longshore sediment transport.

A simple and reasonably conservative scenario is adopted in the present study for the purpose of estimating the future sea defence performance of the shoreline banks. The assumed scenario, which covers the lifetime of SZC to 2110, gives greater weight to recent trends and is characterised by:

- Net recession of the upper beach and seaward face of the dune banks by 50m to 75m along the majority of the Minsmere catchment frontage. It is assumed that recession on the seaward side of the dunes is not accompanied by roll-back of the landward side resulting in significant loss of dune width and volume. It is accepted that the structure of Minsmere Sluice may continue to act as a hard point promoting more stable or accretional shoreline conditions for several hundred metres to either side.
- No significant change to the upper beach, dune banks and landward bank at Sizewell Gap.

A further geomorphological scenario involving the formation of a Minsmere tidal inlet was considered for inclusion in the overland flood modelling by Atkins in consultation with CEFAS and NNB GenCo. However, it was deduced that the presence of a Minsmere tidal inlet would not promote significantly greater overland flooding of Sizewell Belts in an extreme coastal flooding event. The reasons for this are that:

- Even without the formation of a tidal inlet or breach / over-wash of the shoreline banks, there would be flooding of the lower Minsmere catchment as a result of the hydraulic connectivity through the crag strata between the sea and the groundwater / surface water system. Long-term groundwater / surface water levels would be higher than at present due to mean sea level rise and would be further elevated as a result of the initial storm surge build-up. The groundwater / surface water conditions assumed in the overland flood modelling before arrival of the coastal flood event are described in Section 7.1.
- As sea levels rise above about 3m AOD during an extreme coastal flooding event, tidal overflow of the Minsmere frontage would increasingly dominate landward flow through a tidal inlet (predicted to be in the order of 100m wide at its narrowest point [32]). Potential lowering of the Minsmere frontage during an extreme coastal flooding event is examined in Section 4.2.
- The TUFLOW model results presented in Section 8.4 show that peak overland flood levels attained in an extreme coastal flooding event are not particularly sensitive to the initial water levels across the Minsmere catchment.
- A tidal inlet would aid the drain-back of overland floodwater to the sea at times of low astronomical tide and as the storm surge abates. Hence, the assumption of a tidal inlet could have had a non-conservative effect on calculated peak overland flood levels.

4.2. Response to Extreme Storms

A number of methods for predicting and quantifying dune and beach erosion are identified in the scientific literature (CIRIA Beach Management Manual [33], United States Coastal Engineering Manual [34] and Pye et al, 2007 [35]). The present study applies the following two methods to assess the extreme storm response of the shoreline banks along the Minsmere frontage and at Sizewell Gap:

- The method of Vellinga (1986) [36] and subsequent extensions and amendments (noted in Van Rijn, 2013 [37]). The Vellinga (1986) equation[4] provides a means of predicting the storm response of sand dunes in terms of a re-configured profile seaward of the crest. The method is intended for sand dunes rather than shingle barriers or beaches.
- The Bradbury method (2000 [38], 2005 [39]) provides a means of predicting breach of a shingle barrier beach. Breach is defined as the short-term lowering of the barrier crest with wave induced over-washing. The method accounts for the effective inertia of the barrier under wave attack based on the freeboard and cross-sectional area above the still water level.

The two methods are regarded as complementary for the assessment of the Minsmere frontage and Sizewell Gap because of the intermediate gravel / sand composition of the dunes and beach sediment. For the Vellinga method, an intentionally coarse particle size of 0.5mm (moderately coarse sand) was selected which is considered to be towards the upper end of the model validity for sand dunes. Both methods were applied to a set of present-day cross-shore profiles provided by NNB GenCo [40] for the Minsmere frontage and Sizewell Gap subject to a range of 10,000 year return period sea conditions (water level and wave combinations) taken from the offshore joint probability analysis (Section 3.2).

4.2.1. Vellinga Method Results

A selection of results from the Vellinga method for present-day sea conditions and cross-shore profiles are shown in Figure 7.

Even with present-day cross-shore profiles and sea conditions (no climate change), the Vellinga method predicts significant storm response with erosion and partial flattening of the seaward facing dunes. The degree of penetration and flattening along the Minsmere frontage depends on the local breadth of the dune system in the cross-shore direction and whether there is a second bank to the rear of the seaward dune. Where the seaward dune bank is narrow (less than about 30m cross-shore dimension above 3m AOD), the results suggest almost complete loss of the bank crest above about 3m AOD. Where the seaward bank is broad (greater than about 50m cross-shore dimension above 3m AOD), the results suggest about 20m of erosion of the seaward face tapering from the crest down to about 1m AOD. The predicted erosion is greatest on the upper part of the dune and typically about 1m loss of crest height is predicted. The method incorporates volume balance and shows the material eroded from the dune bank and upper beach being re-deposited on the lower

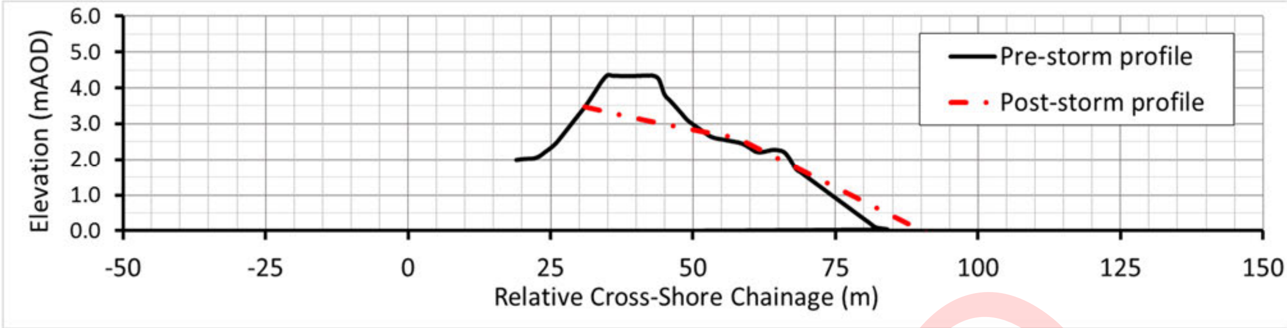
[4] The Vellinga (1986) equation is also referred to as the 'empirical DUROS+ method'.

foreshore below 1m AOD. At Sizewell Gap, the method predicts flattening of the lower seaward bank with the higher rear bank remaining intact.

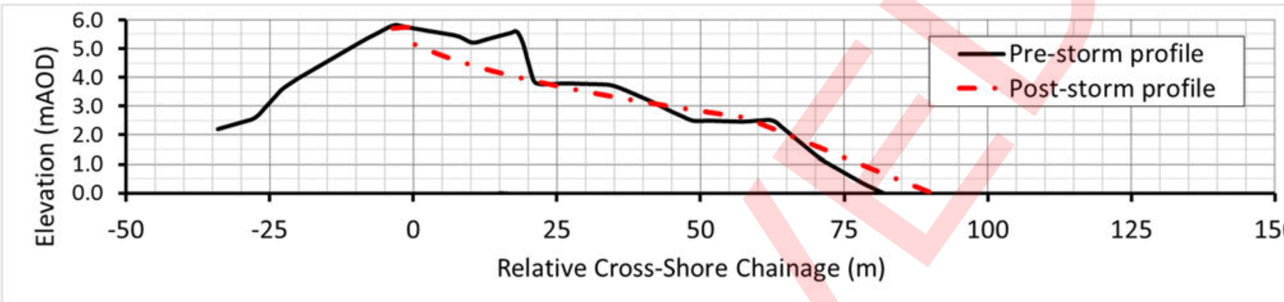
The storm response predicted by the Vellinga method for future 10,000 year return period sea conditions incorporating reasonably foreseeable has been briefly assessed for comparison with the present-day results. As expected, the response is even more severe with typically an additional 15m to 20m of erosion cut into the dune crest. The response to credible maximum climate change has not been calculated as the pattern is already clear and the degree of overflow would probably exceed the validity range for the Vellinga method. At Sizewell Gap, the future extreme storm erosion accounting for climate change is predicted to reach the seaward face of the higher rear bank resulting in about 1m loss of crest height to a level of about 5m AOD.

The combined effect of long-term recession of the Minsmere frontage (50m to 75m, Section 4.1) followed by extreme storm response has not been explicitly calculated. In is estimated that this degree of long-term recession would leave the majority of the Minsmere frontage with narrow dune banks having a total cross-shore dimension above 3m AOD of less than 30m. Applying the above Vellinga results suggests that the subsequent extreme storm response over the majority of the Minsmere frontage would result in complete loss of the bank crest above about 3m AOD with present-day sea conditions and above about 2m AOD with future sea conditions (reasonably foreseeable or credible maximum).

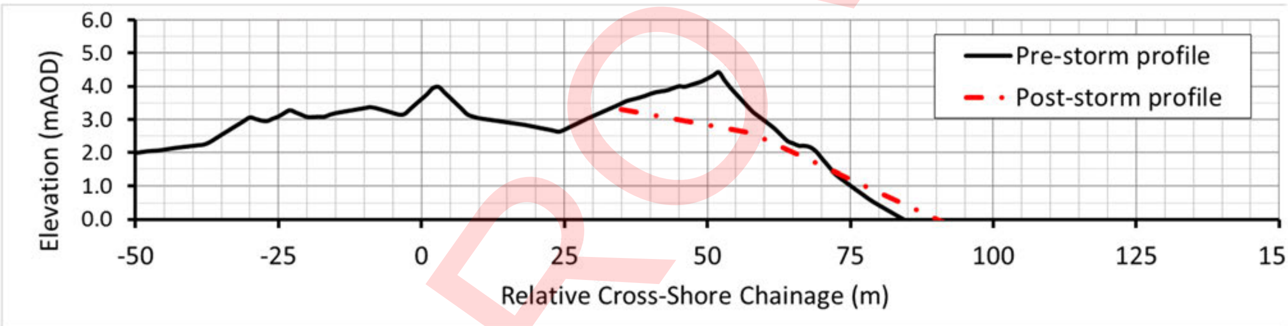
The main limitations of the Vellinga method for the present application are (i) that it is intended for sand dunes (rather than gravel / shingle) and (ii) that it applies volume-balance to the seaward face and does not cover the erosive effect of overtopping and overflow on the landward side. The first of these factors is expected to make the results pessimistic for gravel-dominated banks as coarser material promotes greater stability. The second factor suggests that the method may underestimate the degree of erosion and flattening where the still water level approaches or exceeds the dune crest. Given the recorded response of the Minsmere frontage to much smaller storms [26], the results on balance appear to be reasonable and not too pessimistic.



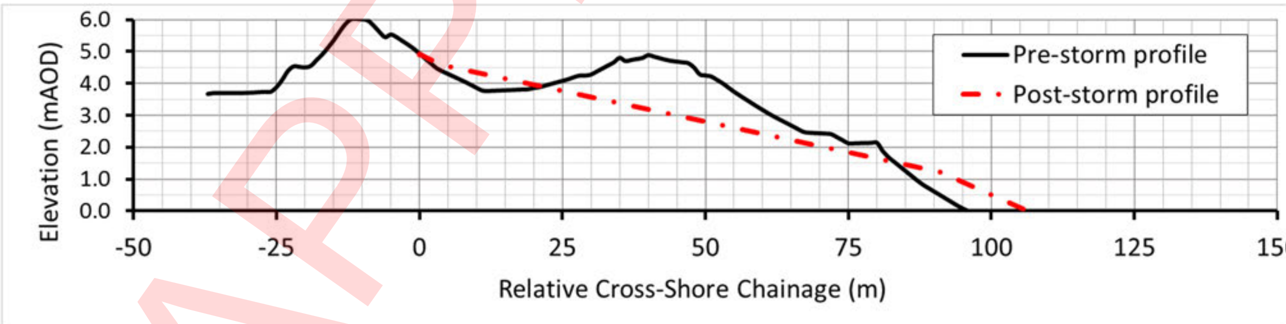
(a) North Minsmere frontage (north of sluice), 267000 N, near section S1B2



(b) Central Minsmere frontage (near sluice), 266000 N, near section S1B3



(c) South Minsmere frontage (south of sluice), 265000 N, near section S1B4



(d) Sizewell Gap (towards SZA), 262869 N, near section S1B6

Figure 7. Results of Storm Response Dune Profile Analysis (Vellinga method, 1986)

4.2.2. Bradbury Method Results

The results of the Bradbury analysis for present-day sea conditions and cross-shore profiles are shown in Figure 8. Output points below the threshold lines (lower barrier inertia / lower wave steepness) indicate a predicted breach of the barrier beach for that cross-shore profile. The further the points lie below the threshold line, the greater the degree of predicted over-wash breaching and associated recession / lowering of the residual crest.

The Bradbury results are in broad agreement with the Vellinga method results. Breach is predicted for all of the assessed profiles along the Minsmere frontage (for all joint probability combinations of waves and still water levels). Breach is marginally predicted at Sizewell Gap but to a much lesser extent. Over half of the joint probability points lie below the Bradbury 2000 threshold and all but one of the joint probability points lie below the Bradbury 2005 threshold.

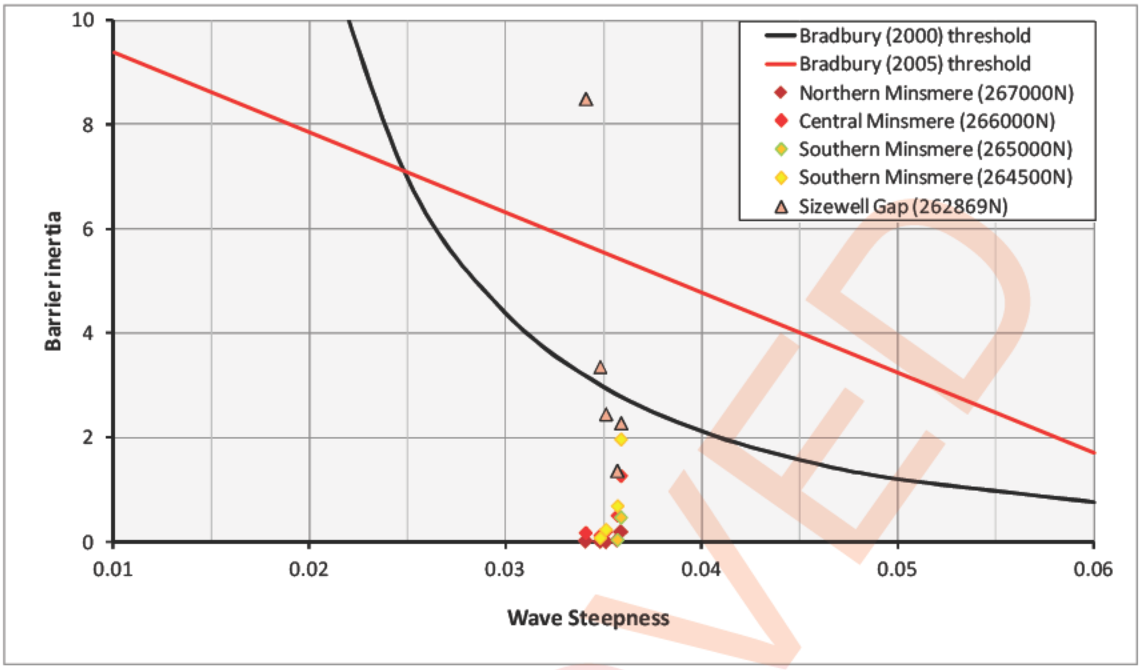


Figure 8. Results of Storm Response Breach Analysis (Bradbury method, 2000 & 2005)

4.3. Shoreline Bank Topography Assumed for Flood Modelling

The shoreline topography used for the overland flood modelling accounts for the predicted changes due to the extreme storm response in all cases and for long-term geomorphological change prior to the extreme storm in the future Minsmere frontage cases incorporating climate change. The shoreline topography adopted in the TUFLOW model is defined in Table 6 based on the findings of Section 4.2. The defined topography (including the storm response effect) was applied at the start of the TUFLOW model runs. This is considered reasonable as the majority of the response is expected to occur as the storm conditions escalate during the rising surge.

Case	Present-Day No climate change	Future to 2110 Reasonably Foreseeable Climate Change	Future to 2110 Credible Maximum Climate Change	Notes
Minsmere Frontage	Partial flattening of the dunes/banks due to extreme storm response (only) to a level mid-way between the current topography and 2mOD. This is consistent with a loss of crest above 3m AOD for the narrower banks (Section 4.2.1).	Flattening of the dunes/banks down to 2mOD level due to long-term recession and extreme storm response.	Flattening of the dunes/banks down to 2mOD level due to long-term recession and extreme storm response.	The height of the dunes/banks is adjusted in the model. For simplicity, the cross-shore position and dimensions are not changed to reflect recession and cut-back.
Sizewell Gap	Partial flattening of seaward dune bank due to extreme storm response (only). Full height of landward bank retained (approximately 6m AOD).	Partial flattening of the dunes/banks down to 5mOD due to extreme storm response (only).	Partial flattening of the dune/bank down to 5mOD due to extreme storm response (only).	The height of the higher dune/bank is adjusted in the model. For simplicity, the cross-shore position and dimensions are not changed to reflect cut-back.

Table 6. Shoreline Bank Topography for Flood Modelling

5. Tidal Overflow, Wave Run-Up and Overtopping

This section summarises the coastal inputs and boundary conditions applied in the overland flood modelling for the Minsmere frontage and Sizewell Gap based on the sea conditions (waves and still water levels) described in Section 3 and the shoreline topography described in Section 4. The relative contribution of wave overtopping and direct overflow is considered. Scoping calculations demonstrate that direct overflow dominates wave overtopping where the still water level exceeds the shoreline crest level by a metre or more.

The wave run-up levels calculated on the seaward slope to the east of the SZC site are presented in Section 5.3.

5.1. Minsmere Frontage

Even with present-day conditions (no climate change or long-term recession), the shoreline crest height along the Minsmere frontage allowing for storm response (from about 2.5m to 4m AOD) is considerably less than the peak of the tide curve (5.20m AOD). By inspection of the present-day tide curve (Figure 1), the still water level exceeds 3.5m AOD for 7 hours, 4.5m AOD for 4.25 hours and 5.0m OAD for 2 hours. Given the degree and duration of exceedance, it is evident that direct overflow dominates wave overtopping along the Minsmere frontage in present-day conditions. The dominance of direct overflow is even more pronounced in future conditions where the still water levels are greater and the predicted shoreline crest level is lower. As a result, the boundary condition along the Minsmere frontage in the TUFLOW model is defined directly by the tide curves (stage-time boundary) and no additional input due to wave overtopping is applied.

5.2. Sizewell Gap

With present-day conditions (no climate change), the majority of shoreline crest height at Sizewell Gap (about 6m AOD) is greater than the peak of the tide curve (5.20m AOD) although close inspection of the LiDAR data suggests short sections which are as low as 5m AOD. Hence, little direct overflow is predicted. Accounting for the joint probability of waves and water levels, maximum flow-rates from wave overtopping are calculated (EurOtop [24] Equation 5.9) to be in the order of 0.01 $m^3/s/m$ near the peak of the tide curve. This flow-rate is negligible in comparison to the corresponding overflow rates along the Minsmere frontage and would not contribute significantly to overland flood levels across Sizewell Belts. As a result, no additional input due to wave overtopping at Sizewell Gap is applied in the TUFLOW model.

For reasonably foreseeable future conditions, the crest height at Sizewell Gap allowing for storm response (5m AOD) is exceeded by still water level (tide curve peak of 5.95m AOD) by up to 1m for a duration of 5.5 hours. Hence, direct overflow would dominate wave overtopping at Sizewell Gap in the predicted future conditions (reasonably foreseeable and credible maximum). The boundary conditions at Sizewell Gap in the TUFLOW model are defined directly by the tide curves (stage-time boundary) and no additional input due to wave overtopping is applied.

5.3. Run-Up on Seaward Slope to the East of SZC

The wave run-up on the seaward slope to the east of SZC has been calculated using the Van der Meer method [24] for all joint probability combinations of waves and still water levels at 10,000 year return period (present-day, reasonably foreseeable and credible maximum – see Figure 2). The gradient and roughness of the seaward slope and toe are assumed to be similar to those of the preferred concept design option for the sea protection embankment (Option 2.1 in [15], as shown in Figure 9). The inputs to the Van der Meer run-up calculation are presented in Table 7:

The results of the run-up calculation are presented in Table 8. It should be noted that, in accordance with convention, the calculated run-up levels are nominally those that would be exceeded by 2% of waves for the sea state at the peak of the tide curve. The selection of the dry site platform level (above the run-up level) for credible maximum (and reasonably foreseeable) climate change is presented below with a margin to allow for control of overtopping and reasonable variation in the key input parameters.

Item	Baseline Value Adopted	Reference	Notes
Slope of nearshore sea bed	1:100	Derived in Section 3.3.2	The sensitivity of the calculated run-up to a steeper sea bed slope of 0.02 is estimated below.
Level of toe (residual, during extreme storm)	3.0m AOD	A toe level of 3.0m AOD is defined as a fundamental assumption in the design basis for the sea protection embankment and in the associated overtopping analysis [15].	The level of the toe is important as it controls the local water depth, wave height and run-up height. Erosion of the toe could destabilise the rock armour and lead to slope failure. In line with [15], it is understood that measures will be undertaken to secure a residual toe level of at least 3.0m AOD. NNB GenCo has advised Atkins that the current beach management plan is to maintain the normal beach level immediately seaward of the embankment at 4.0m AOD with buried rock armour at 3.0m AOD (to top of rock). The results of the Vellinga method in Section 4.2.1 indicate that allowance should be for made for erosion in response to an extreme storm unless the toe material is stabilised. The sensitivity of the calculated run-up to minor additional toe erosion (to a level of 2.5m AOD) is estimated below.
Seaward slope geometry	1:3 simple slope	A 1:3 slope is consistent with that adopted for the main slope in Option 2.1 [15].	The berm at 5.0m AOD and 1:4 lower slope in Option 2.1 are conservatively neglected (it is noted that Option 2.1 was selected as the preferred sea protection option after the run-up calculation had been completed).
Seaward slope roughness	Exposed rock armour (run-up roughness coefficient 0.55)	A rough seaward face is required to limit run-up heights. This is consistent with the exposure of the rock armour assumed for Jacobs' Stage 2 design [15].	With reference to [15], a rock armour size (median mass) of 2.5 tonne is estimated to be required for stability in reasonably foreseeable conditions and a rock armour size (median mass) of 11 tonne is estimated for credible maximum conditions. The sensitivity of the calculated run-up to a small reduction in roughness (to 0.65) is estimated below.

Table 7. Inputs for Calculation of Run-Up on Seaward Slope to East of SZC

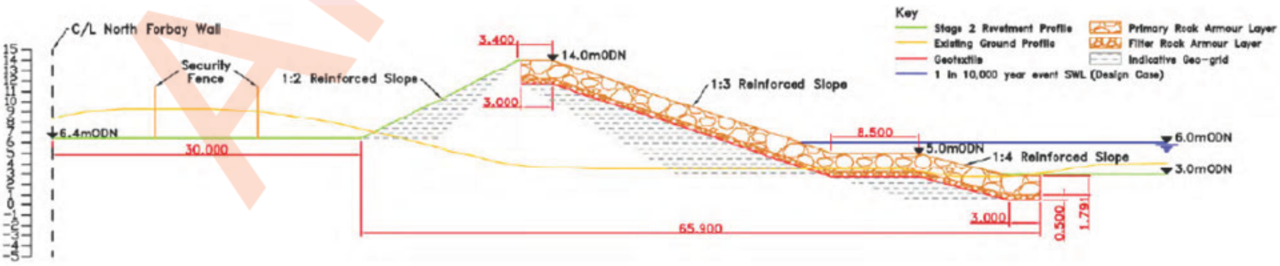


Figure 9. Cross-Section of SZC Sea Protection Embankment – Option 2.1, Stage 2 [15]

Sea Condition / Climate Change Case	Joint Probability Case (Figure 2)	Still Water Level, (SWL) (m AOD)	Offshore Wave Height (H_{m0}) (m)	Water Depth at Toe (h) (m)	Breaker Index (H_b/h)	Wave Height at Toe (m) (lesser of H_{m0}, H_b)	Run-Up Height 2% (m) (above SWL)	Run-Up Level 2% (m AOD)
Present-day	A	2.06	8.14	< 0	N/A	N/A	N/A	N/A
	E	3.35	7.46	0.35	0.65	0.23	0.77	4.12
	B	4.54	5.68	1.54	0.65	0.99	2.20	6.74
	F	4.93	4.94	1.93	0.65	1.24	2.52	7.45
	C	5.20	3.46	2.20	0.60	1.32	2.47	7.67
Reasonably foreseeable	A	2.80	8.95	< 0	N/A	N/A	N/A	N/A
	E	4.09	8.21	1.09	0.65	0.71	1.86	5.95
	B	5.28	6.25	2.28	0.65	1.47	3.03	8.31
	F	5.67	5.43	2.67	0.63	1.68	3.26	8.93
	C	5.94	3.81	2.94	0.58	1.71	3.09	9.03
Credible maximum	A	5.18	8.95	2.18	0.64	1.40	3.17	8.35
	E	6.47	8.21	3.47	0.64	2.22	4.40	10.87
	B	7.66	6.25	4.66	0.58	2.70	4.90	12.56
	F	8.05	5.43	5.05	0.55	2.78	4.90	12.95
	C	8.32	3.81	5.32	0.52	2.74	4.54	12.86

Table 8. Results of Calculation of Run-Up on Seaward Slope to East of SZC

From the results in Table 8, a baseline (2%) run-up level of about 9m AOD is predicted for reasonably foreseeable condition and a baseline (2%) run-up level of about 13m AOD is predicted for credible maximum conditions. The allowances in Table 9 are considered in order to derive a reasonably robust dry site platform level which would be subject to minimal overtopping.

Item	Sensitivity
Effect of non-uniform slope of nearshore sea bed	Increasing the slope from 1:100 to 1:50 (i.e. 0.02) to match the steepest inshore slope would increase the wave height at the toe by up to 15% and the run-up height by up to 10%. This equates to an increase in run-up level of about 0.3m for the reasonably foreseeable case and 0.5m for the credible maximum case.
Effect of reduction in toe level	Reducing the toe level from 3.0m AOD to 2.5m AOD increases the wave height at the toe by about 0.25m and the run-up height by about 0.4m in the reasonably foreseeable case and the credible maximum case.
Effect of reduction in slope roughness	Increasing the slope roughness coefficient from 0.55 to 0.65 (NB. a smooth slope has a coefficient of 1.0) increases the run-up height by about 18%. This equates to an increase in run-up level of about 0.5m for the reasonably foreseeable case and 0.8m for the credible maximum case.
Allowance to minimise overtopping accounting for random sea state	A margin of 10% is sought on the 2% run-up height to minimise overtopping recognising the effect of larger random waves. This equates to a margin on run-up level of about 0.3m for the reasonably foreseeable case and 0.5m for the credible maximum case.

Table 9. Run-Up on Seaward Slope to East of SZC – Allowances for Variation in Input Parameters

From Table 9, an overall allowance of 1.5m is proposed on the baseline run-up level for reasonably foreseeable conditions and an overall allowance of 2m is proposed on the baseline run-up level for credible maximum conditions. These allowances are based on judgement and it is considered overly conservative to apply the sum of all four items. This gives the following dry site platform levels:

- Dry site level for reasonably foreseeable climate change: 10.5m AOD
- Dry site level for credible maximum climate change: 15m AOD

6. Model Area and Onshore Topography

The model area for the overland flood analysis covers the Minsmere and Sizewell Belts catchments up to a level exceeding the peak tide level (8.32m AOD), as indicated in Figure A.1.1.

The model topography is selected to be representative of the permanent SZC development during operation of the power station. The general model terrain is taken from filtered LiDAR survey data provided by NNB GenCo [25]. The received LiDAR data set is a composite of several surveys flown since 1999 with different resolutions (2m, 1m and 0.5m).

The SZA and SZB sites are incorporated in the model at their current levels from the LiDAR data. It is noted that the SZA and SZB safety cases will ensure adequate protection of the SZA and SZB sites from coastal flooding over their appropriate lifetimes which may be exceeded by the 2110 end-of-life climate change timescale adopted in this report for SZC. The SZC site (defined by the raised site plot plan area [41]) is set at a high level above the modelled flood levels.

The levels of the access road and adjoining land along the eastern part of Goose Hill will be a function of the platform level. It is assumed that the level of the access road where it meets the permanent bridge is the same as the platform level option being considered. Details of the planned permanent (post-construction) topography along the eastern part of Goose Hill (for a 6.4m AOD platform level) were not available from NNB GenCo at the start of the flood levels analysis. It was agreed with NNB GenCo at the time that a simplified method could be used to define the levels between the permanent access bridge and the 10m AOD contour on Goose Hill. Three variants of the developed Goose Hill topography were generated each with a near-uniform gradient along the line of the access road to suit platform levels at 6.4m AOD[5], 7.5m AOD and 8.5m AOD (Figure A.1.5). It is noted that this method gives higher topography levels (relative to the platform level) along the east end of Goose Hill (within about 750m of the access bridge) than does the construction plot plan [42] (relative to a 6.4m AOD platform level).

7. Onshore Hydrological Conditions

While the overland flooding is predominantly driven by the extreme sea conditions, the modelling also accounts for the effect of coincident fluvial flows and groundwater / surface water conditions.

Minsmere and Sizewell Belts are low-lying with ground levels less than 1m AOD over the majority of their area. The topography is shown in Figure A.1.1. Fluvial flows from the small surrounding catchments are channelled through a series of watercourses to outfall through the barrier beach at Minsmere Sluice.

Minsmere Sluice is included in the TUFLOW model based on a simple representation of the internal levels, cross-sections and lengths for the outfalls taken from the previous reinstatement works drawings [43] and the recent refurbishment drawings [44]. Partial landward flow is permitted during high tides to reflect the design of the refurbished sluice which replicates the seawater leakage of the old sluice. The modelling shows that operation of the sluice has no effect on overland flooding driven by extreme sea conditions as the flow-rates are so small. The results of the present assessment are therefore not sensitive to the long-term operation of the sluice (or otherwise).

7.1. Groundwater and Surface Water Conditions

The hydrology of Minsmere and Sizewell Belts is strongly influenced by the hydraulic connectivity between the groundwater / surface water system and the sea through the highly permeable crag deposits which underlie

[5] In this case, the lower end of the smoothed topography was modelled at 6.5m AOD rather than 6.4m AOD. This difference has a negligible effect on the overland flood modelling as the associated TUFLOW runs had Goose Hill shoulder closed or the northern flood pathway either fully open or fully closed.

local peat and clay alluvium. From the recent groundwater monitoring and conceptual models described in [45], the character of the hydrological regime over Sizewell Belts may be summarised as follows:

- Groundwater levels in the peat and crag deposits maintain a small hydraulic gradient over mean sea level (0.16m AOD, present-day) indicating flow to the sea via the crag deposits.
- Groundwater contours are aligned broadly parallel with the shoreline (i.e. north-south) indicating flow to the east.
- Groundwater levels are influenced by tidal conditions for up to 1km inland with a temporary reversal in the normal hydraulic gradient being observed during the December 2013 storm surge.
- Typical groundwater levels are close to ground level (within about 0.15m) over the low-lying areas and rising groundwater levels reach ground level in response to rain events (about 1 day delay).
- Standard groundwater levels in the north-eastern part of Sizewell Belts to the north-west of the SZC site are around 0.6m AOD. The groundwater levels rise to the west at a rate of about 0.1m every 100m.

Similar groundwater conditions are expected to exist over Minsmere given the similar geology (although monitoring information is not available). In order to account for high groundwater conditions and associated surface water, the TUFLOW model domain is pre-wetted to a defined water level before applying the coastal inputs. As shown in Table 10, a different pre-wetted surface water level is adopted for each climate change scenario recognising the direct link between mean sea level and groundwater levels. For the future climate change scenarios, it is assumed that Minsmere Sluice would have insufficient capacity to regulate surface water levels through drainage to sea at low tides.

Climate Change Scenario	Mean Sea Level	Mean Sea Level Rise (above present)	Pre-Wetted Surface Water Level in TUFLOW Model	Notes
Present-day	0.16m AOD	0m	0.6m AOD	Based on standard monitored groundwater level in Sizewell Belts to north-west of SZC site.
Reasonably foreseeable climate change	0.91m AOD	0.75m	1.35m AOD	Mean sea level rise added to present-day value.
Credible maximum climate change	2.28m AOD	2.12m	2.0m AOD	Mean sea level rise added to present-day value would give 2.72m AOD. However, this would exceed the modified crest level of the Minsmere frontage in the TUFLOW model accounting for long-term recession and extreme storm response (Section 4.3). Hence, the pre-wetted surface water level has been capped at 2m AOD.

Table 10. Pre-Wetted Surface Water Levels in TUFLOW Model

7.2. Rainfall and Fluvial Conditions

Fluvial inputs are included in the TUFLOW model to represent watercourse flows from the surrounding watershed into the model area which could reasonably occur concurrently with a 10,000 year return period coastal flooding event. No direct rainfall is applied to the TUFLOW model as it is considered to be adequately covered by the fluvial inputs.

Strong winds and precipitation would reasonably be expected in the Sizewell area in association with the extreme sea conditions (storm surge and waves). The storm surge would be driven by a deep low pressure system and associated rainfall would be frontal rather than convective. While this type of rainfall could be heavy, it would rarely be exceptionally intense and the timing of the greatest rainfall and subsequent fluvial flows may not coincide with the arrival of the peak storm surge or the largest waves (generated offshore). It is considered that a rainfall event with a return period of 1 year would be sufficient as a combined case. However, to be conservative, the fluvial inputs are based on a rainfall event of 10 year return period. A rainfall event duration of 6 hours is assumed in keeping with frontal rainfall and the anticipated response time for the moderately small catchment area (78 km^2).

The total rainfall (mm) over the catchment was taken from the Flood Estimation Handbook Depth-Duration-Frequency model [46] and the flow-rate hydrographs (m^3/s versus time) for the watercourses feeding into the model area were calculated using the Revitalised Flood Estimation Hydrograph (ReFH) method [47]. An allowance of +20% was added for climate change to 2110 in line with the change factor case in [13]. The total 10 year return period rainfall over the 6 hour event was calculated to be 35.5mm and was uniformly applied.

The peak hydrograph flow-rate for the dominant Yoxford catchment (about 85% of total) was found to be about 10 m^3/s occurring about 16 hours after the start of the rainfall event within an overall hydrograph duration of 48 hours. The timing of the hydrograph inputs to the TUFLOW model gave peak fluvial flows during the shoulder tide on the rising storm surge. The total volume of the fluvial hydrograph flows is three or four times greater than the volume of direct rainfall on the model floodplain area. In other words, the maximum flood effect of direct rainfall and fluvial flows combined is to increase overland flood levels by about 0.15m (conservatively neglecting all drainage). It is evident that rainfall and fluvial inputs have a negligible effect on the overland flood levels in comparison to the tidal inputs.

8. Overland Flood Modelling

8.1. Model Software and Set-Up

The overland flood model has been built in TUFLOW Classic (version TUFLOW.2013-12-AC). A fixed grid model forms the basis of the main two dimensional (2D) model domain. The TUFLOW analysis software is designed to solve the free-surface flow equations to simulate tidal and overland flood propagation (www.tuflow.com). Dynamic linking is provided between one dimensional (1D) elements and the 2D digital elevation model to allow accurate integrated representation of linear hydraulic features such as watercourses, culverts and narrow weirs. TUFLOW is a well-established industry standard software that has been benchmarked by the Environment Agency [48].

The present TUFLOW model uses a 10m fixed grid for the 2D terrain generated from LIDAR data [25]. Trial model runs confirmed the suitability of this grid size with respect to accuracy, stability and run times.

Adjustments to the 2D topography for the SZC development are described in Section 6. Modifications to the model for the representation of flood barriers (and any associated openings) are described in Section 8.2.

1D elements are introduced in the TUFLOW model to represent:

- Minsmere Sluice outfalls with flow direction control
- V-notch fluvial opening through Northern Flood Barrier (see Section 8.2)

8.2. Representation of Flood Barriers in Model

In accordance with Table 4, the following two flood barriers are represented in the relevant model runs for the control of still water levels in Sizewell Belts adjacent to the SZC site (see Figure A.1.3):

- The Northern Flood Barrier (between the Northern Mound and Goose Hill) limits ingress into Sizewell Belts via the northern flood pathway.
- The Southern Flood Barrier (at Sizewell Gap) prevents ingress into Sizewell Belts via the southern flood pathway.

Both barriers were represented in the TUFLOW model by deactivating a continuous line of 2D cells along the crest position (indicated by white colouring on maximum flood level maps in Appendix B). This prevented any overflow or through-flow except where specific openings were trialled. The representation of the Northern Flood Barrier accounting for openings is described below. The position of the deactivated cells used to represent the effect of the Southern Flood Barrier is shown in Figure A.1.6. It should be noted that the positioning and geometry of the barrier features in the TUFLOW model is simplified and approximate. It is nevertheless sufficiently accurate to represent their hydraulic performance. Further consideration will be given to barrier positioning and alignment in the developed definition of the options in the ALARP assessment.

The Northern Flood Barrier comprises the following three openings which are variously open or closed in the different TUFLOW model runs (see Figure A.1.3):

- V-notch opening for fluvial channel.
  This was modelled as a 1D open culvert element representing the geometry shown in Figure A.1.4. For model runs where the V-notch channel was closed to prevent overland flows from the sea into Sizewell Belts, a one-way (e.g. flap) valve was placed on the opening as fluvial flows could not be obstructed in normal conditions.
- Opening for BLF road near abutment to Northern Mound.
  This was modelled by manually lowering the 2D terrain level along the line of the road where the barrier abuts the northern flank of the Northern Mound. The invert level of the BLF road opening was simply taken as 6.4m AOD in all TUFLOW model runs. This is slightly conservative where higher platform level options are being considered as this part of the BLF road would be close to the platform level.
- Based on the developed topography of Goose Hill, the shoulder of Goose Hill at the north-west end of the barrier would remain open above the level of the platform being considered unless the crest of the barrier is specifically extended to close the opening.
  For model runs where the shoulder of Goose Hill was closed, a continuous line of 2D cells were deactivated extending to a level on Goose Hill above the maximum flood levels.
  It is noted that the developed topography levels adopted in the TUFLOW model along the east end of Goose Hill (Section 6) are higher (relative to platform level) than those indicated in the construction plot plan [42] (relative to a 6.4m AOD platform level). This point should be accounted for when interpreting the results of the TUFLOW modelling with this flow path open.

8.3. TUFLOW Model Runs

Two sets of TUFLOW model runs were performed:

- Series 1 — Without any flood barriers
  With present-day conditions
  With reasonably foreseeable conditions at end-of-life (2110)
  With credible maximum conditions at end-of-life (2110)
- Series 2 — With various combinations of flood barriers and openings in the Northern Flood Barrier
  With credible maximum conditions at end-of-life (2110), only (as explained in Section 2.6).

The model runs are listed and the results presented in Section 8.4 with reference to the outputs shown in Appendix B and Appendix C.

8.4. TUFLOW Model Results

The following forms of TUFLOW model output were extracted for each model run:

- Maximum flood level maps (Appendix B).
- Maximum flow velocity maps (reviewed by Atkins, not presented in report).
- Flood level and velocity time histories at selected output points identified in Figure 10.
  Points 1, 2, 3 and 4 are within Sizewell Belts.
  Point 5 is to the north of the Northern Mound on the seaward side of Northern Flood Barrier position.
- Flow-rate time-histories and total flow volumes over selected output lines (Appendix C).
  Output lines were placed across flood pathways and openings in barriers.

Figure 10. TUFLOW Model Output Points

8.4.1. Series 1 – Without flood barriers

The results of Series 1 are presented in Table 11, Section B.1 (Figures B.1.1, B.1.2 and B.1.3).

Without barriers, it is evident in all cases that the overland flood levels across Sizewell Belts closely approach or reach the peak still water level on the input tide curve. The peak flood levels are very uniform (within a few centimetres) over the entire flooded area. In present-day conditions, the overland flood levels across Sizewell Belts do not quite reach the peak of the tide curve due (i) the lower pre-wetted surface water levels, (ii) the very low flows into the southern flood pathway at Sizewell Gap, and (iii) the more limited duration of overflow along the Minsmere frontage.

Run / Case	Description	Sea Condition		Maximum Flood Water Level in Sizewell Belts adjacent to SZC (Point P4) (m AOD)
		Climate Change Scenario (m AOD)	Maximum Still Water Level (m AOD)	
1.1	• No flood barriers • Goose Hill developed topography tapered to 6.4mOD	Present-day	5.20	5.04
1.2	• No flood barriers • Goose Hill developed topography tapered to 6.4mOD	Reasonably foreseeable (2110)	5.95	5.95
1.3	• No flood barriers • Goose Hill developed topography tapered to 6.4mOD	Credible maximum (2110)	8.32	8.32
Note: All cases run with 10,000 year return period sea conditions.				

Table 11. TUFLOW Model Results – Series 1: Without Flood Barriers

8.4.2. Series 2 – With flood barriers

The results of Series 2 are presented in Table 12, Section B.2 (Figures B.2.1 to B.2.6) and Appendix C. The peak flow-rates in Table 12 give an indication of the relative flows though each flood pathway and each opening in the Northern Flood Barrier. Further details of the flows and volumes with reference to the output time histories are given in Appendix C.

A commentary on the results of Series 2 and the key deductions from each model run is provided in Table 12. As for Series B.1, the peak still water flood levels are very uniform (within a few centimetres) over the entire Sizewell Belts area except in the immediate vicinity of the openings in the Northern Flood Barrier.

<table>
<tr><th rowspan="3">Run / Case</th><th rowspan="3">Detailed Description</th><th rowspan="3">Max Water Level adj. to SZC (P4)</th><th colspan="4">Maximum Flow-Rate via Flood Pathway / Opening (m^3/s)</th></tr>
<tr><th colspan="3">Northern</th><th>Southern</th></tr>
<tr><th>Goose Hill Shoulder</th><th>V-Notch Channel</th><th>BLF Road</th><th>Sizewell Gap</th></tr>
<tr><td rowspan="2">2.1</td><td rowspan="2">• Goose Hill developed topography tapered to 6.4mOD
• Northern flood barrier in place
o V-notch channel closed
o Goose hill shoulder closed
o BLF road closed
• Southern flood barrier omitted</td><td rowspan="2">8.32m AOD</td><td>Closed</td><td>Closed</td><td>Closed</td><td>682.8</td></tr>
<tr><td colspan="4">Only Southern Flood Pathway Open
This result determines that:
• A Southern Flood Barrier is required at Sizewell Gap for the lower two platform level options and is hence retained for all subsequent model runs.
• The level of the third platform level option should be 8.8m AOD to provide a 0.5m margin above the still water level.</td></tr>
<tr><td rowspan="2">2.2</td><td rowspan="2">• Goose Hill developed topography tapered to 6.4mOD
• Northern flood barrier in place
o V-notch channel open
o Goose hill shoulder closed
o BLF road closed
• Southern flood barrier in place</td><td rowspan="2">6.21m AOD</td><td>Closed</td><td>170.2</td><td>Closed</td><td>Closed</td></tr>
<tr><td colspan="4">Only V-Notch Fluvial Channel in Northern Barrier Open
This result determines that:
• A V-notch fluvial channel of the assumed size would not be acceptable with the 6.4m AOD platform option (insufficient margin) but it would acceptable with the second platform level option.</td></tr>
<tr><td rowspan="2">2.3</td><td rowspan="2">• Goose Hill developed topography tapered to 7.5mOD
• Northern flood barrier in place
o V-notch channel closed
o Goose hill shoulder open
o BLF road closed
• Southern flood barrier in place</td><td rowspan="2">2.19m AOD</td><td>34.2</td><td>Closed</td><td>Closed</td><td>Closed</td></tr>
<tr><td colspan="4">Only Shoulder of Goose Hill at Northern Barrier Open
This result shows that:
• Flow volumes through the open Goose Hill shoulder (tapered to 7.5m AOD) are relatively low with no more than a 0.19m contribution to flood levels adjacent to SZC.</td></tr>
<tr><td rowspan="2">2.4</td><td rowspan="2">• Goose Hill developed topography tapered to 6.4mOD
• Northern flood barrier in place
o V-notch channel closed
o Goose hill shoulder closed
o BLF road open
• Southern flood barrier in place</td><td rowspan="2">2.29m AOD</td><td>Closed</td><td>Closed</td><td>26.4</td><td>Closed</td></tr>
<tr><td colspan="4">Only BLF Road through Northern Barrier Open
This result shows that:
• Flow volumes through the BLF road opening are relatively low with no more than a 0.29m contribution to flood levels adjacent to SZC.</td></tr>
<tr><td rowspan="2">2.5</td><td rowspan="2">• Goose Hill developed topography tapered to 7.5mOD with an even gradient along the access road
• Northern flood barrier in place
o V-notch channel open
o Goose hill shoulder open
o BLF road closed
• Southern flood barrier in place</td><td rowspan="2">6.31m AOD</td><td>32.7</td><td>171.9</td><td>Closed</td><td>Closed</td></tr>
<tr><td colspan="4">Only Shoulder of Goose Hill and V-Notch Channel in Northern Barrier Open
This result determines that:
• An open Goose Hill shoulder (tapered to 7.5m AOD and with an even gradient along the access road) would be acceptable together with the open fluvial channel for the second platform level option.</td></tr>
<tr><td rowspan="2">2.6</td><td rowspan="2">• Goose Hill developed topography tapered to 7.5mOD with an even gradient along the access road
• Northern flood barrier in place
o V-notch channel open
o Goose hill shoulder open
o BLF road open
• Southern flood barrier in place</td><td rowspan="2">6.45m AOD</td><td>33.5</td><td>172.6</td><td>26.1</td><td>Closed</td></tr>
<tr><td colspan="4">Shoulder of Goose Hill, BLF Road and V-Notch Channel in Northern Barrier Open
This result determines that:
• An open Goose Hill shoulder (tapered to 7.5m AOD with an even gradient along the access road) and an open BLF road would be acceptable together with the open fluvial channel for the second platform level option.
• A margin of 0.75m is sought above the still water level to allow for waves and turbulent flow conditions downstream of openings in the Northern Flood Barrier.
• A platform level of 7.3m AOD is selected to achieve the required margin while allowing for increased flow through the open Goose Hill shoulder (tapered to 7.3m AOD to suit platform level).</td></tr>
<tr><td colspan="7">NB. All cases are for 10,000 year return period credible maximum conditions to 2110.</td></tr>
</table>

Table 12. TUFLOW Model Results – Series 2: With Flood Barriers

9. Definition of Options

The platform levels and flood barriers for each option to be taken forward in the ALARP assessment are presented in Table 13.

Option No.	Option 1	Option 2	Option 3	Option 4
Platform Level	6.4m AOD	7.3m AOD	8.8m AOD	15m AOD
Northern flood barrier	Northern flood barrier would be required for credible maximum (2110) climate change: - Fluvial channel open - Goose hill shoulder closed - BLF road closed A minimum crest level of about 10m AOD is estimated to prevent direct overflow and to limit wave transmission for credible maximum climate change. Other measures may be required from the start of station life to control wave transmission towards the north side of the SZC site, and associated run-up / overtopping, before construction of the northern flood barrier.	Northern flood barrier required for credible maximum (2110) climate change: - Fluvial channel open - Goose hill shoulder open (with even gradient along access road) - BLF road open A minimum crest level of about 10m AOD is estimated to prevent direct overflow and to limit wave transmission for credible maximum climate change. Other measures may be required to control wave transmission towards the north side of the SZC site, and associated run-up / overtopping, before construction of the northern flood barrier.	Northern flood barrier not required for credible maximum (2110) climate change. Measures may be required in future to control wave transmission towards the north side of the SZC site, and associated run-up / overtopping.	Northern flood barrier and other measures not required for credible maximum (2110) climate change.
Southern flood barrier	Southern flood barrier for credible maximum (2110) climate change. A minimum crest level of about 9m AOD is estimated for credible maximum climate change to prevent direct overflow and to avoid excessive overtopping volumes	Southern flood barrier for credible maximum (2110) climate change. A minimum crest level of about 9m AOD is estimated for credible maximum climate change to prevent direct overflow and to avoid excessive overtopping volumes.	Southern flood barrier not required for credible maximum (2110) climate change.	Southern flood barrier not required for credible maximum (2110) climate change.
Eastern sea protection barrier	As ongoing sea protection embankment design.	As ongoing sea protection embankment design.	As ongoing sea protection embankment design.	Incorporated into platform seaward slope. 15m AOD platform level assumes maintenance of the SZC foreshore to secure a residual toe level of 3.0m AOD.

Table 13. Definition of Options for ALARP Assessment - Platform Levels and Flood Barriers

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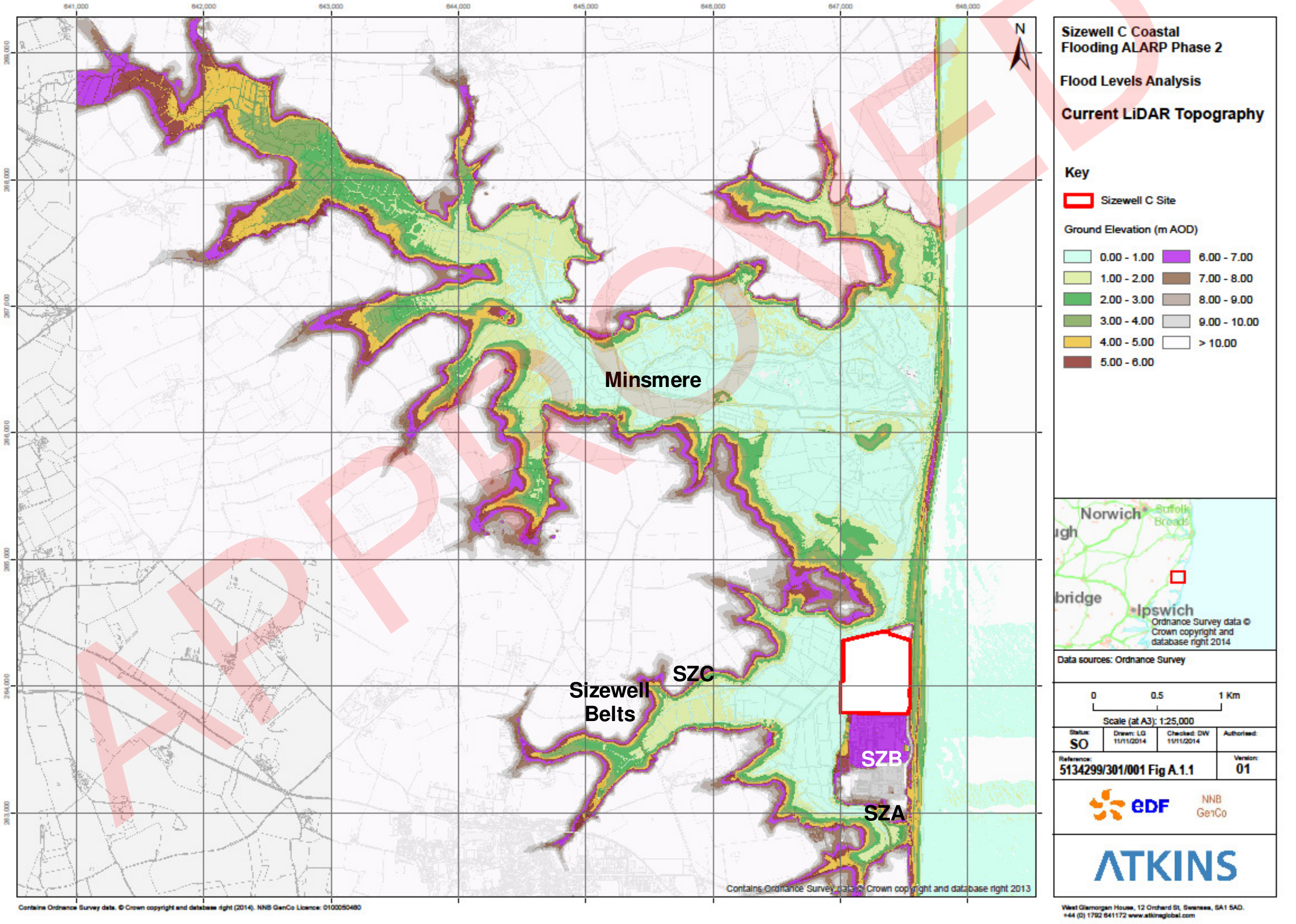
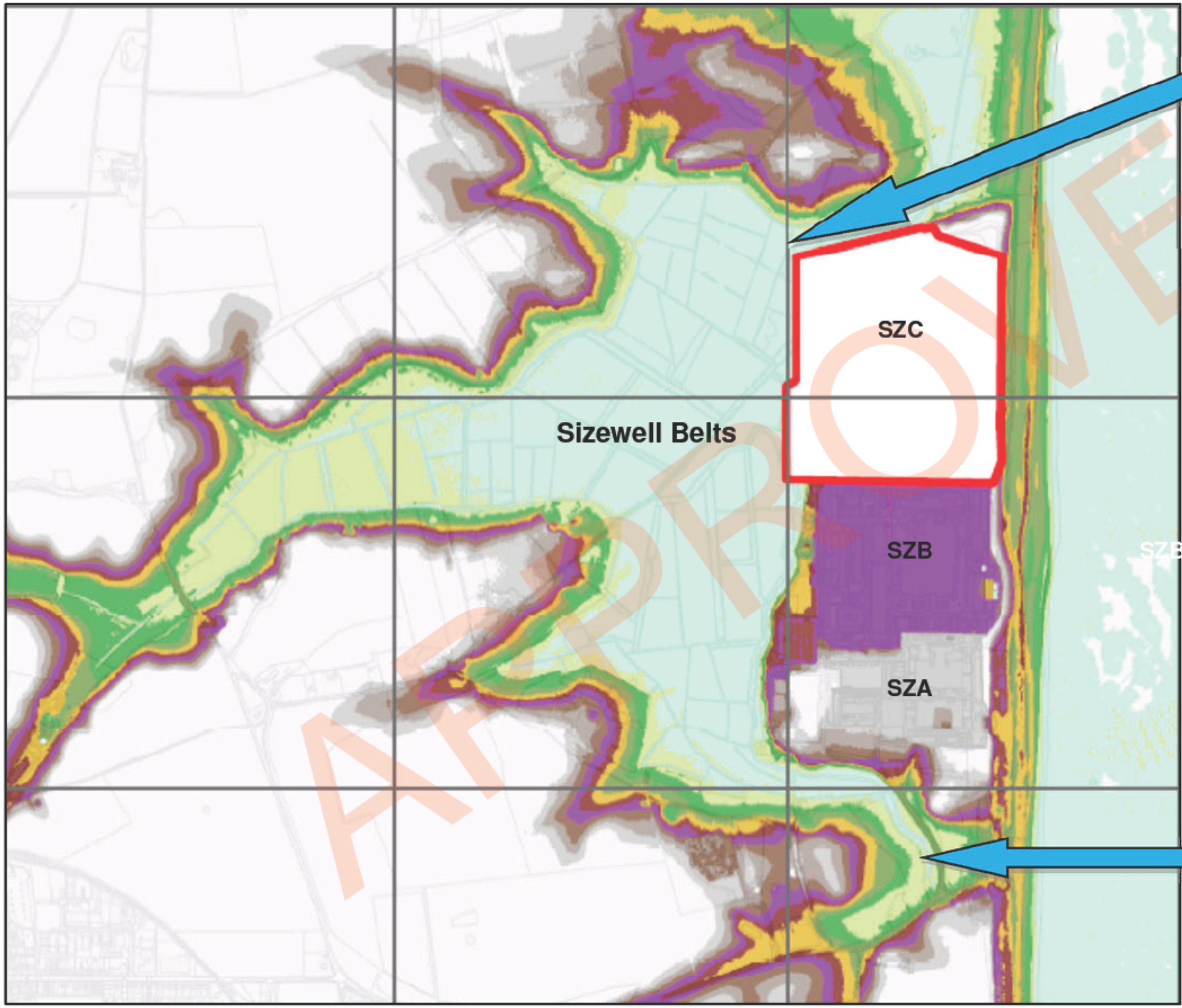
Appendix A. Topography, Flood Pathways and Barriers

A.1.1. Current Topography from LiDAR



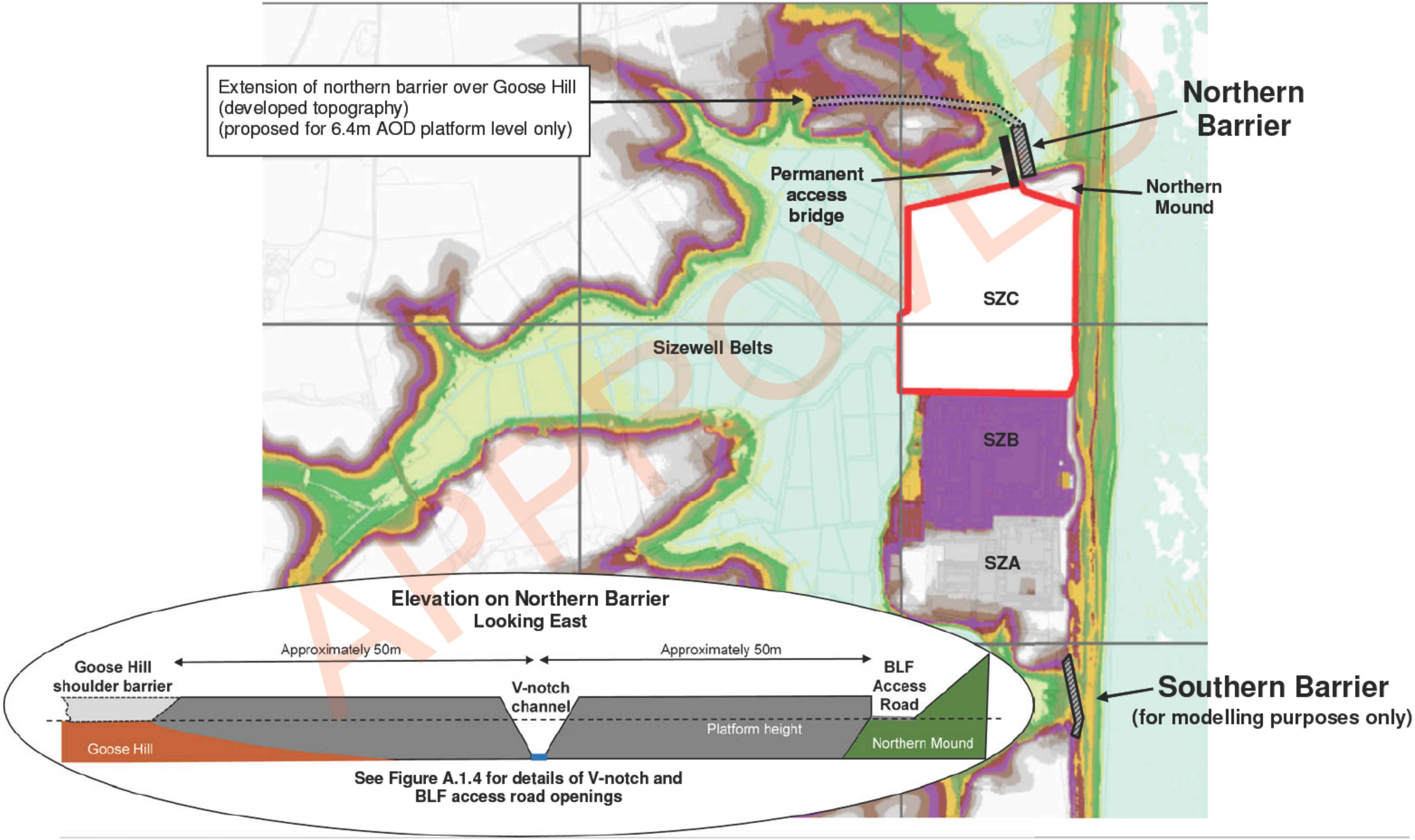
A.1.2. Flood Pathways (overlaid on current LiDAR topography)



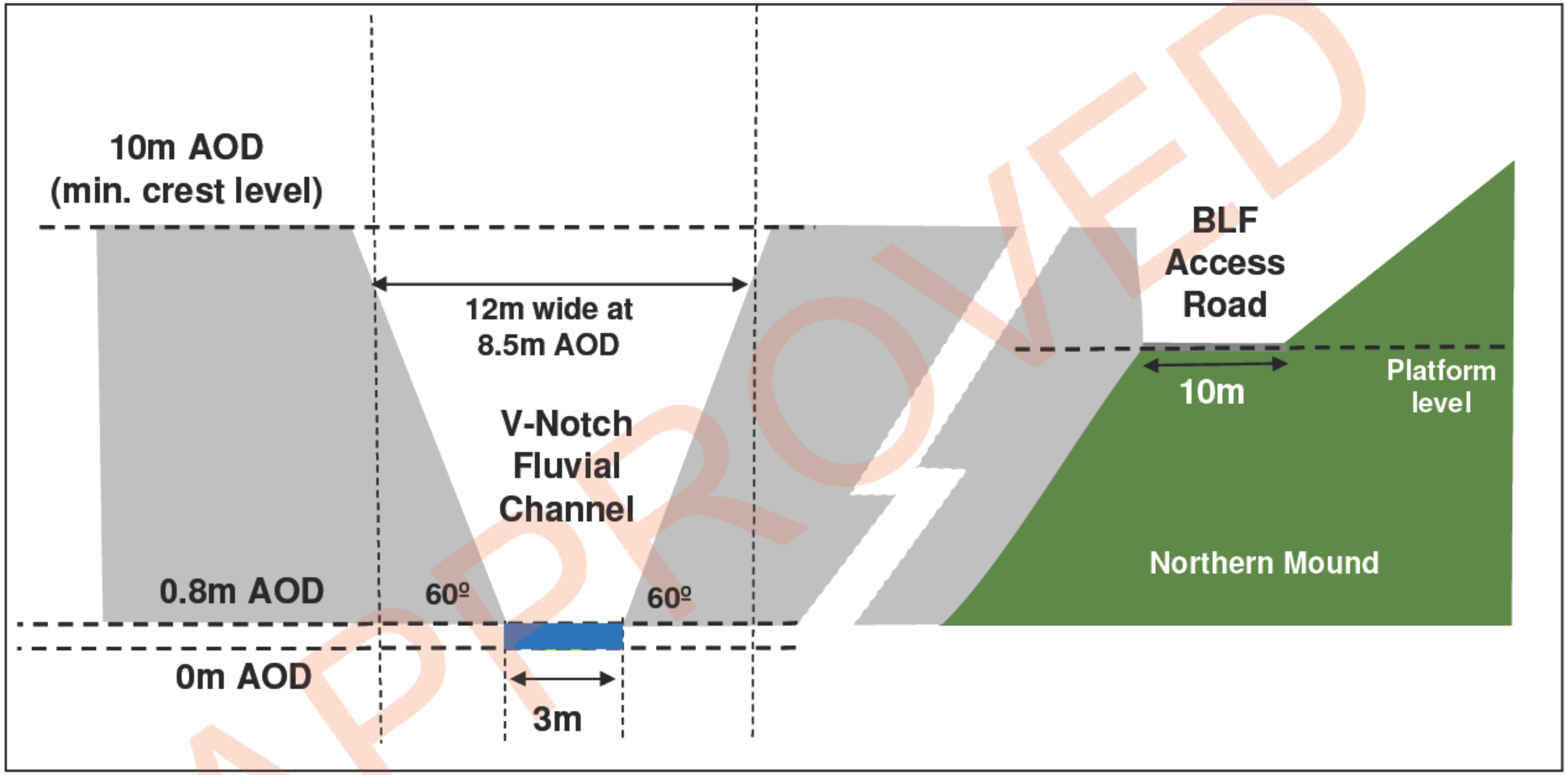
Northern flood pathway
Overflow / overtopping of the coast to the north of SZC (Minsmere frontage) leading to overland flooding to the north-west and west of the SZC site (Sizewell Belts).

Southern flood pathway
Overflow / overtopping of the coast to the south of SZA (Sizewell Gap) leading to overland flooding to the west and north-west of SZC site (Sizewell Belts).

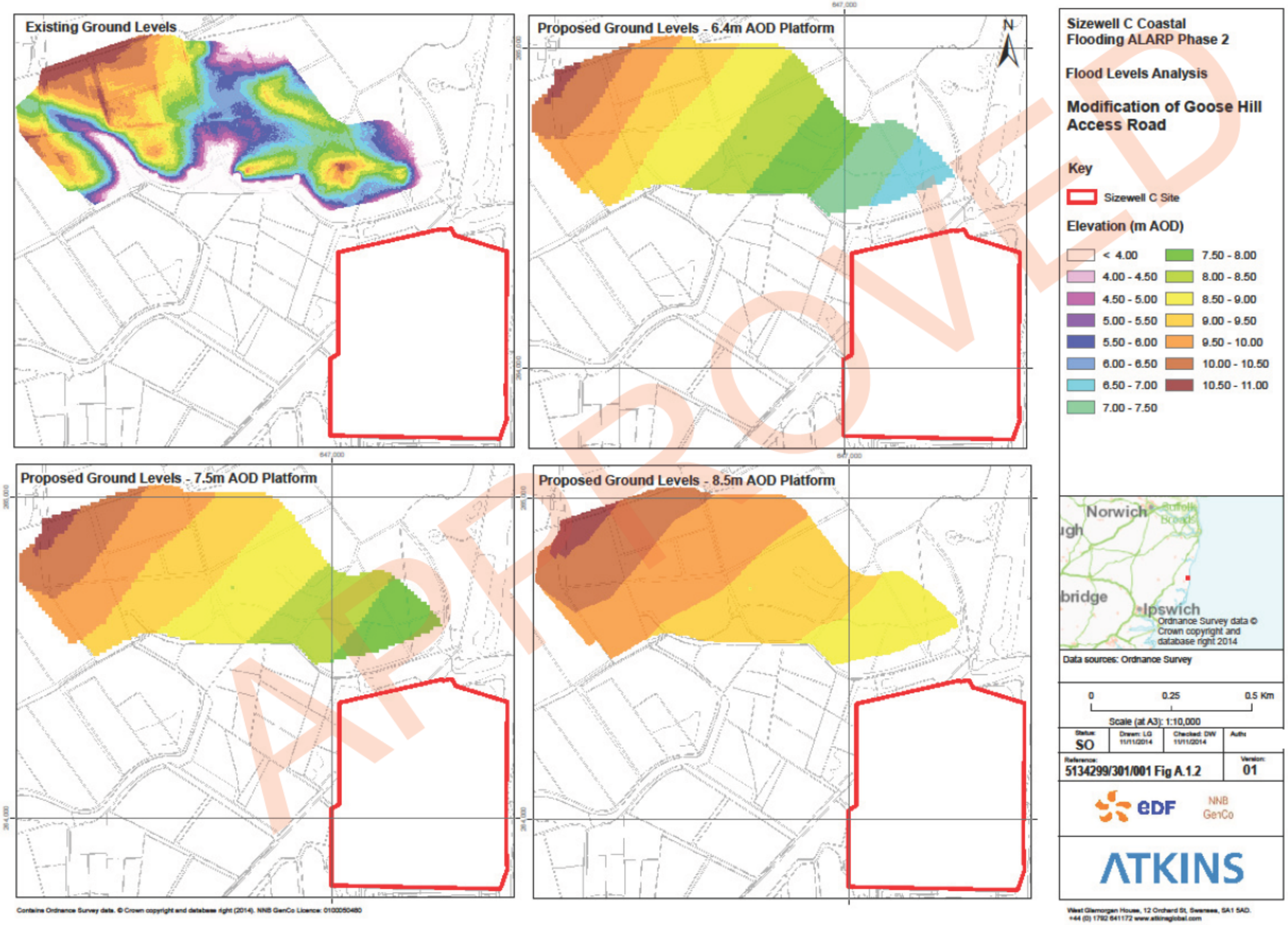
A.1.3. Flood Barriers Key Plan (overlaid on current LiDAR topography)



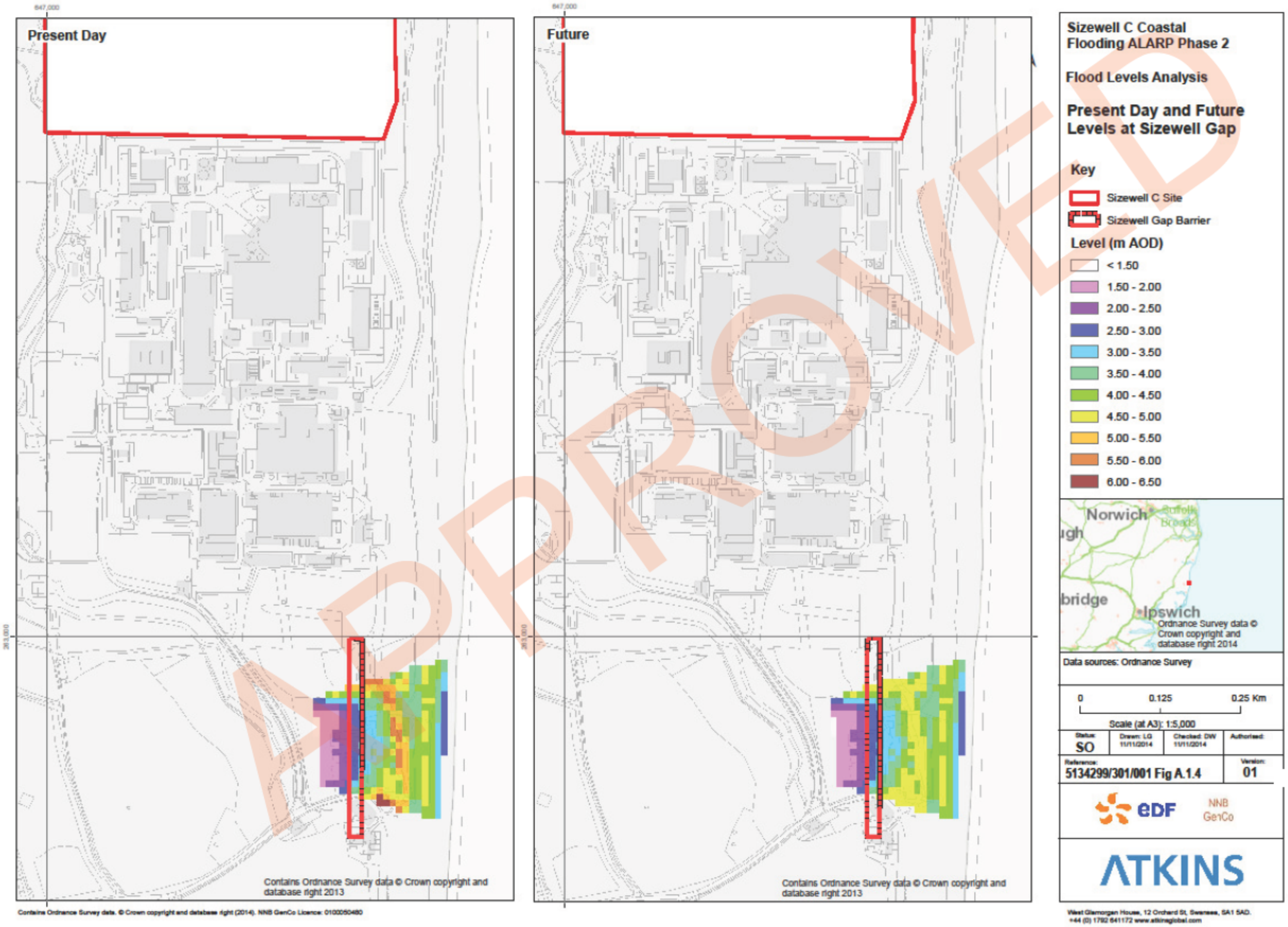
A.1.4. Details of V-Notch and BLF Access Road Openings in Northern Barrier



A.1.5. Goose Hill Developed Topography for TUFLOW Model



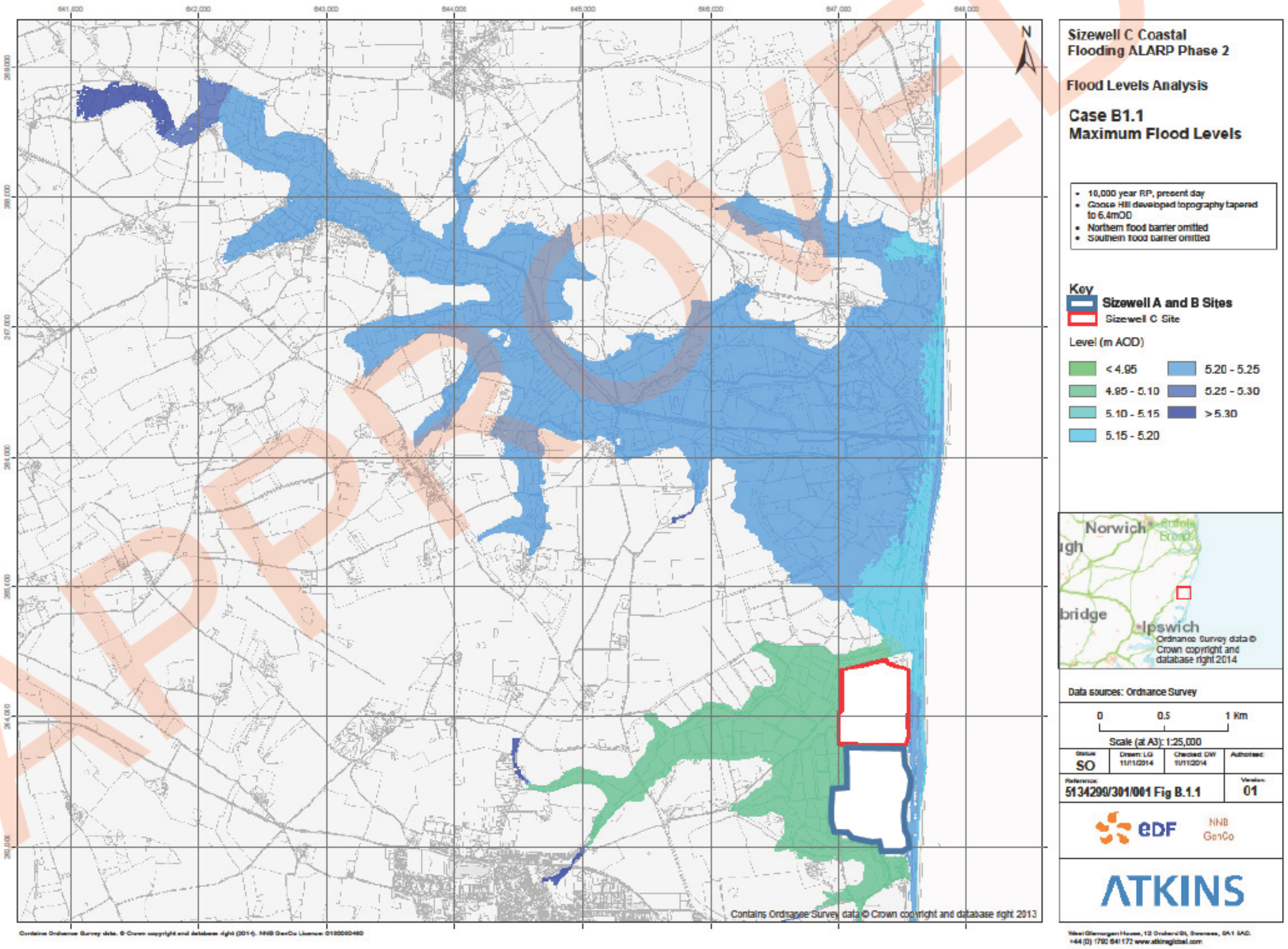
A.1.6. Sizewell Gap Topography for TUFLOW Model (left = present-day as LiDAR; right = future, 5m AOD crest)



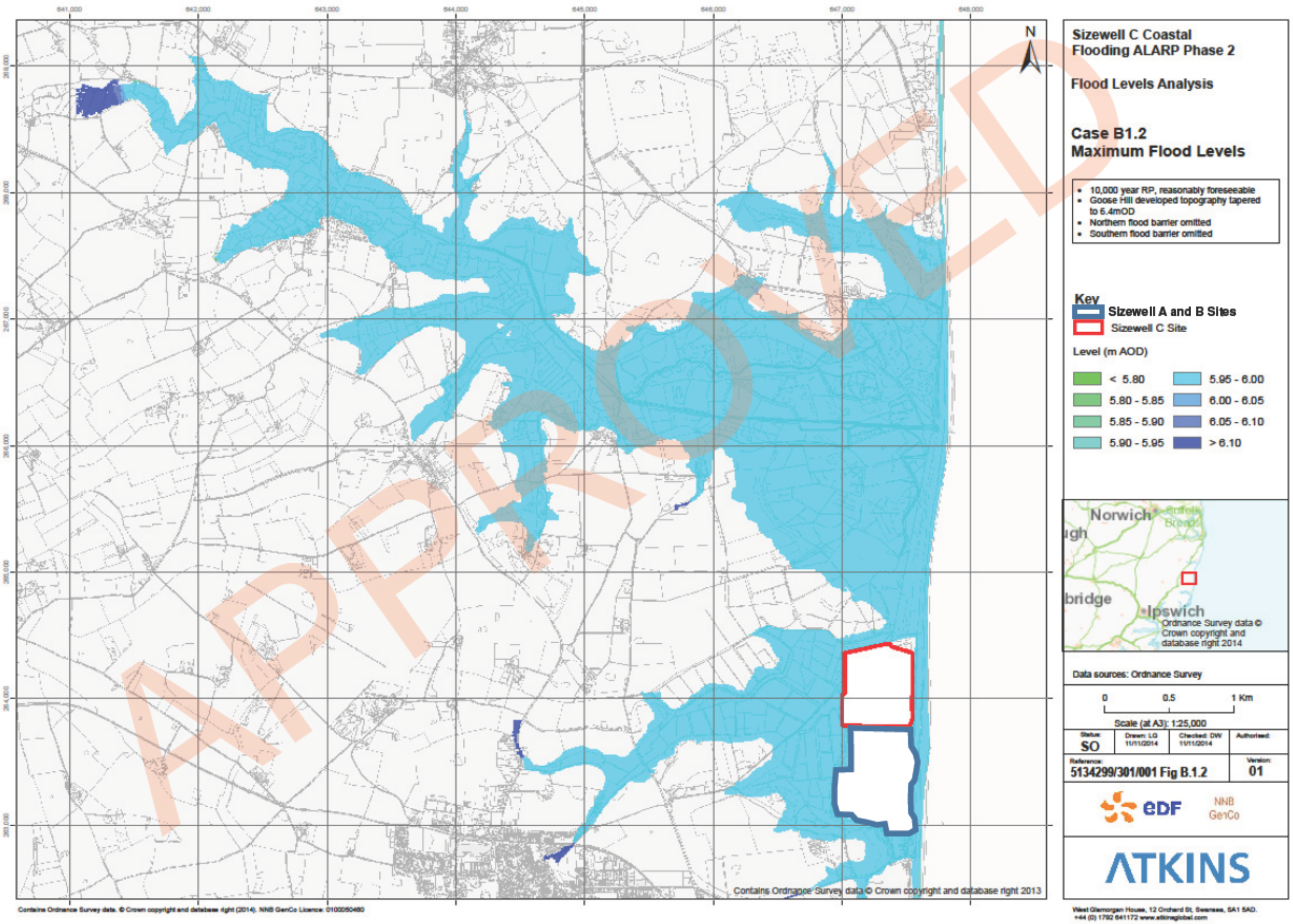
Appendix B. TUFLOW Model Output Maps

B.1. Maximum Flood Levels without Barriers (all 10,000 year return period)

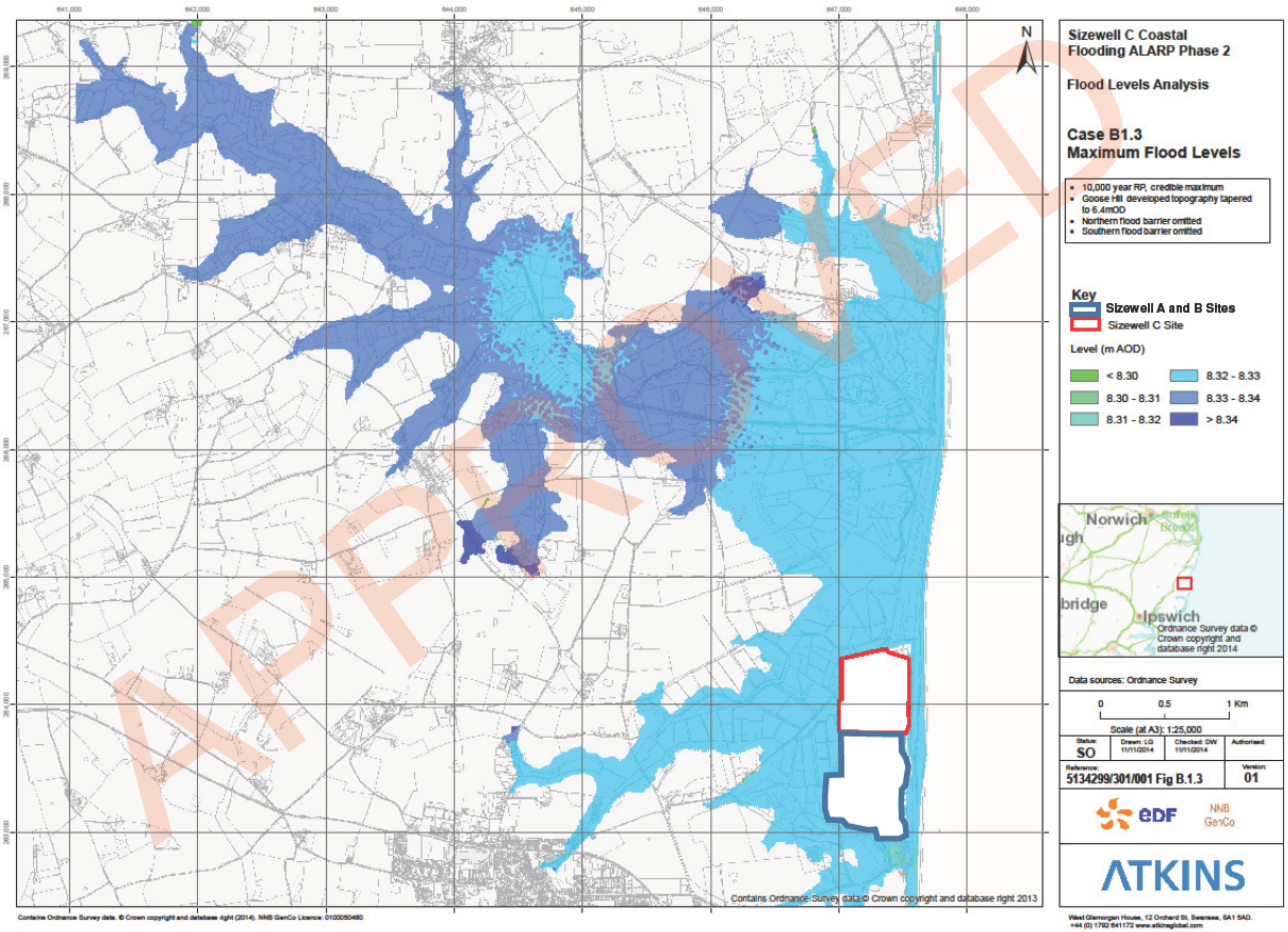
B.1.1. Present-Day (2008)



B.1.2. Reasonably Foreseeable Climate Change (2110)

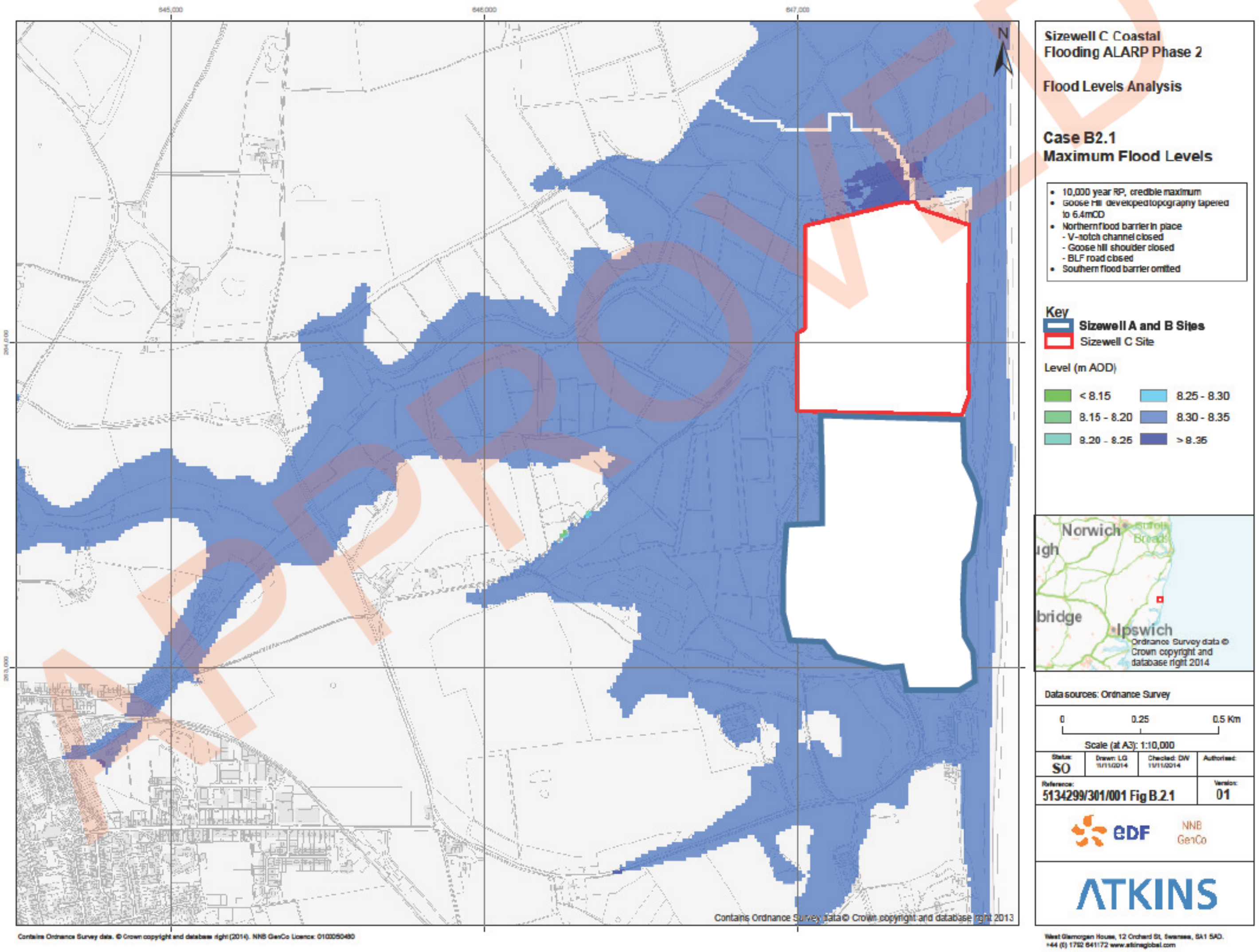


B.1.3. Credible Maximum Climate Change (2110)

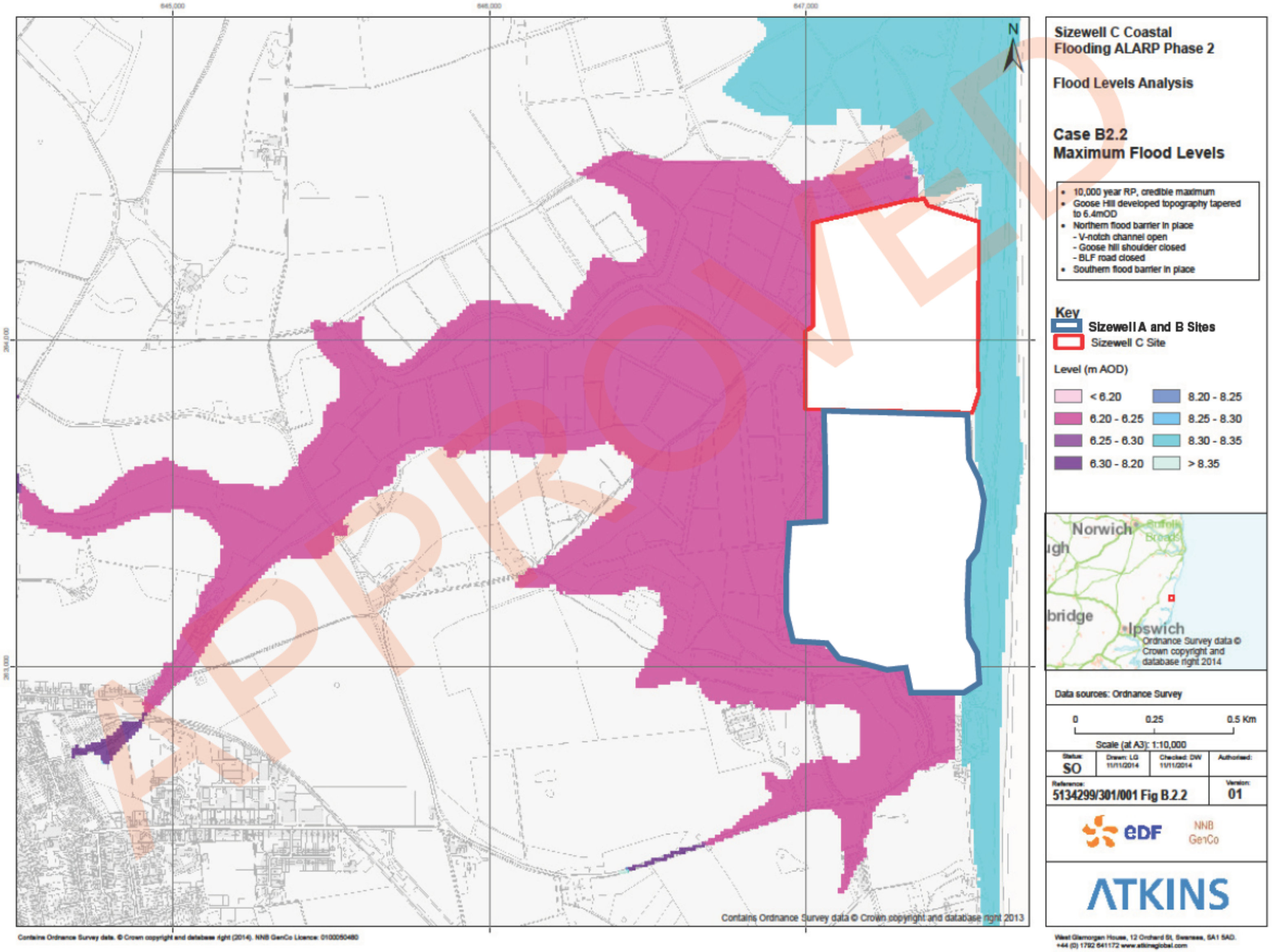


B.2. Maximum Flood Levels with Barriers (all 10,000 year return period with credible maximum climate change to 2110)

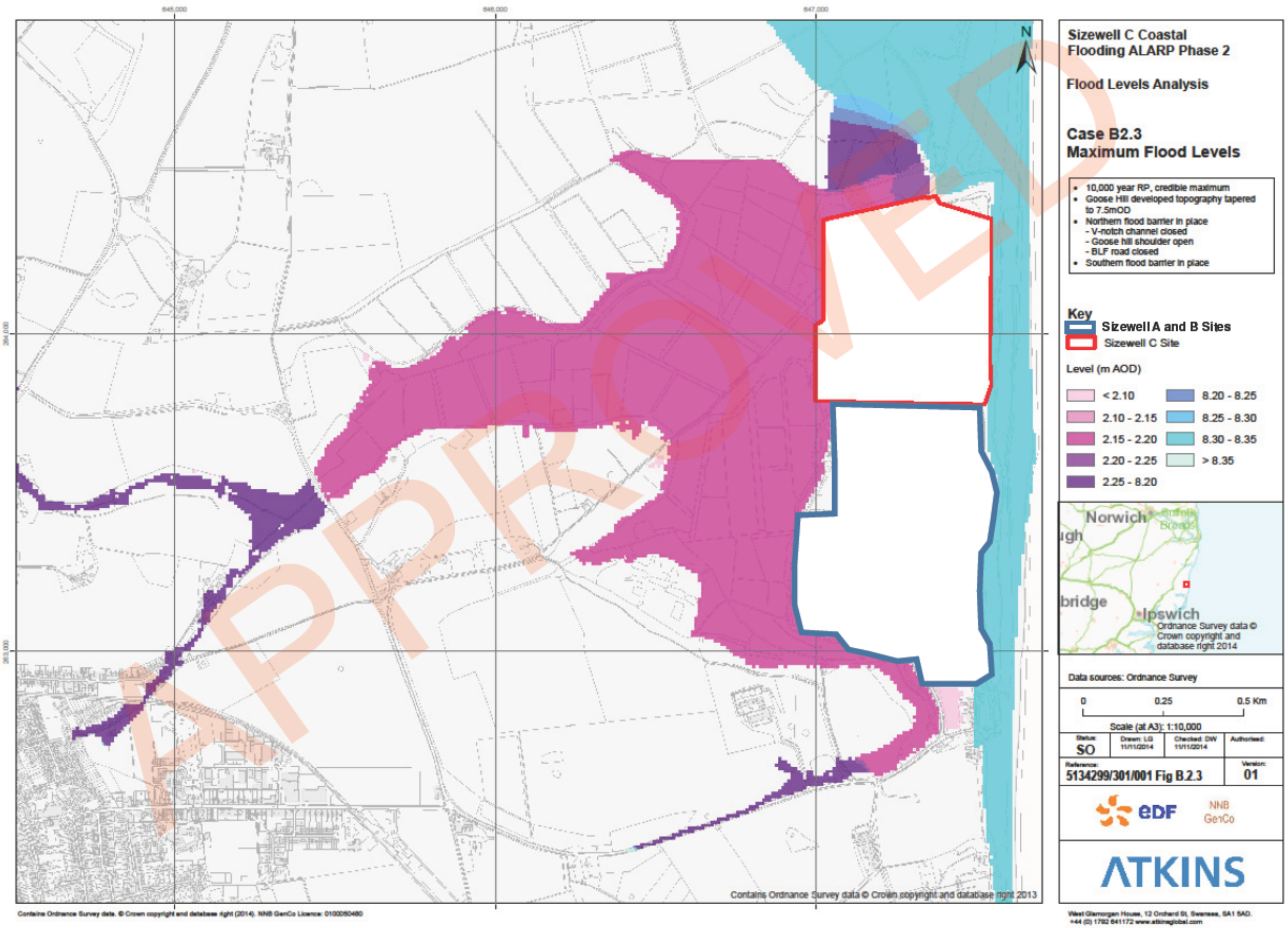
B.2.1. Only Southern Flood Pathway Open



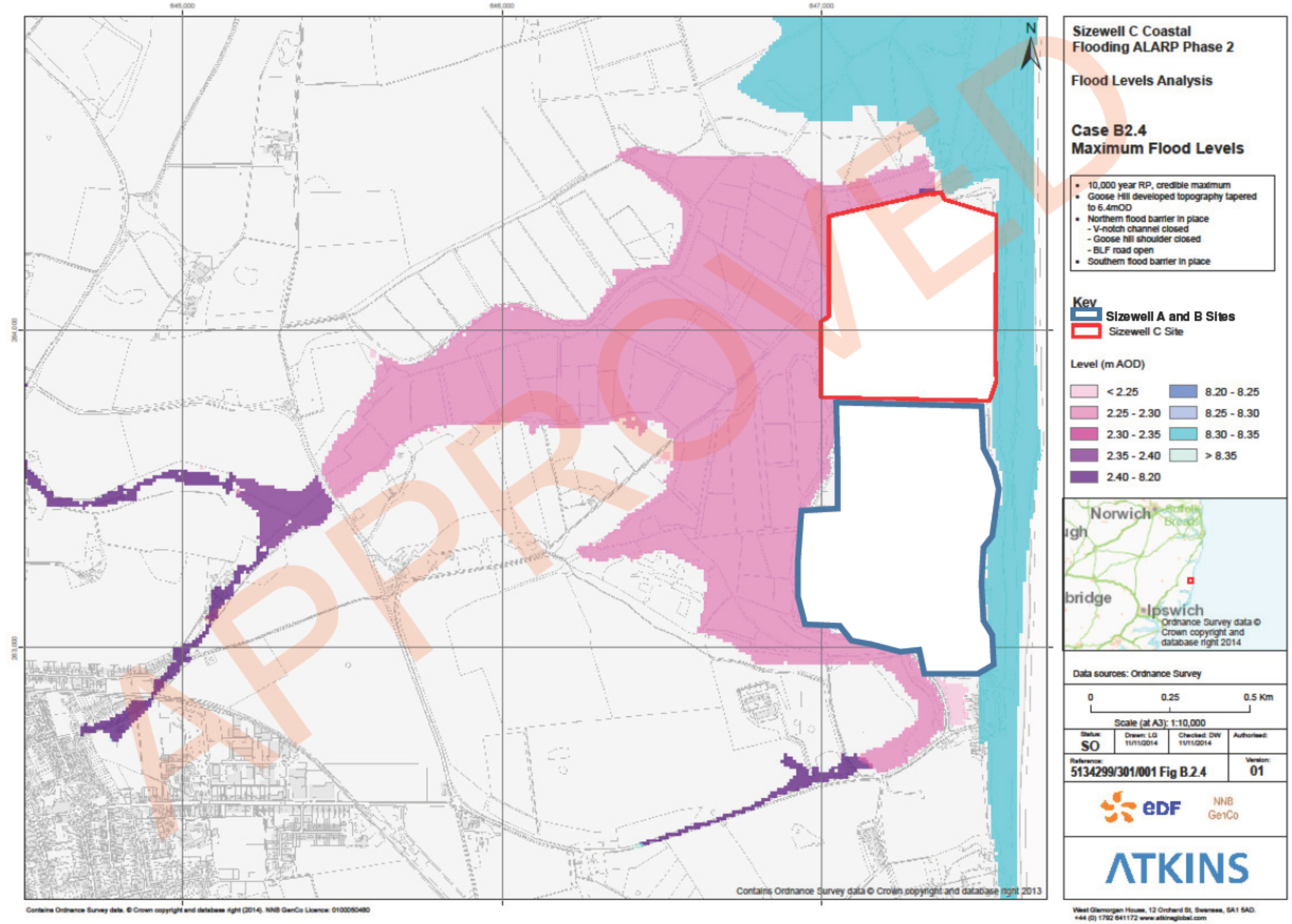
B.2.2. Only V-Notch Fluvial Channel in Northern Barrier Open



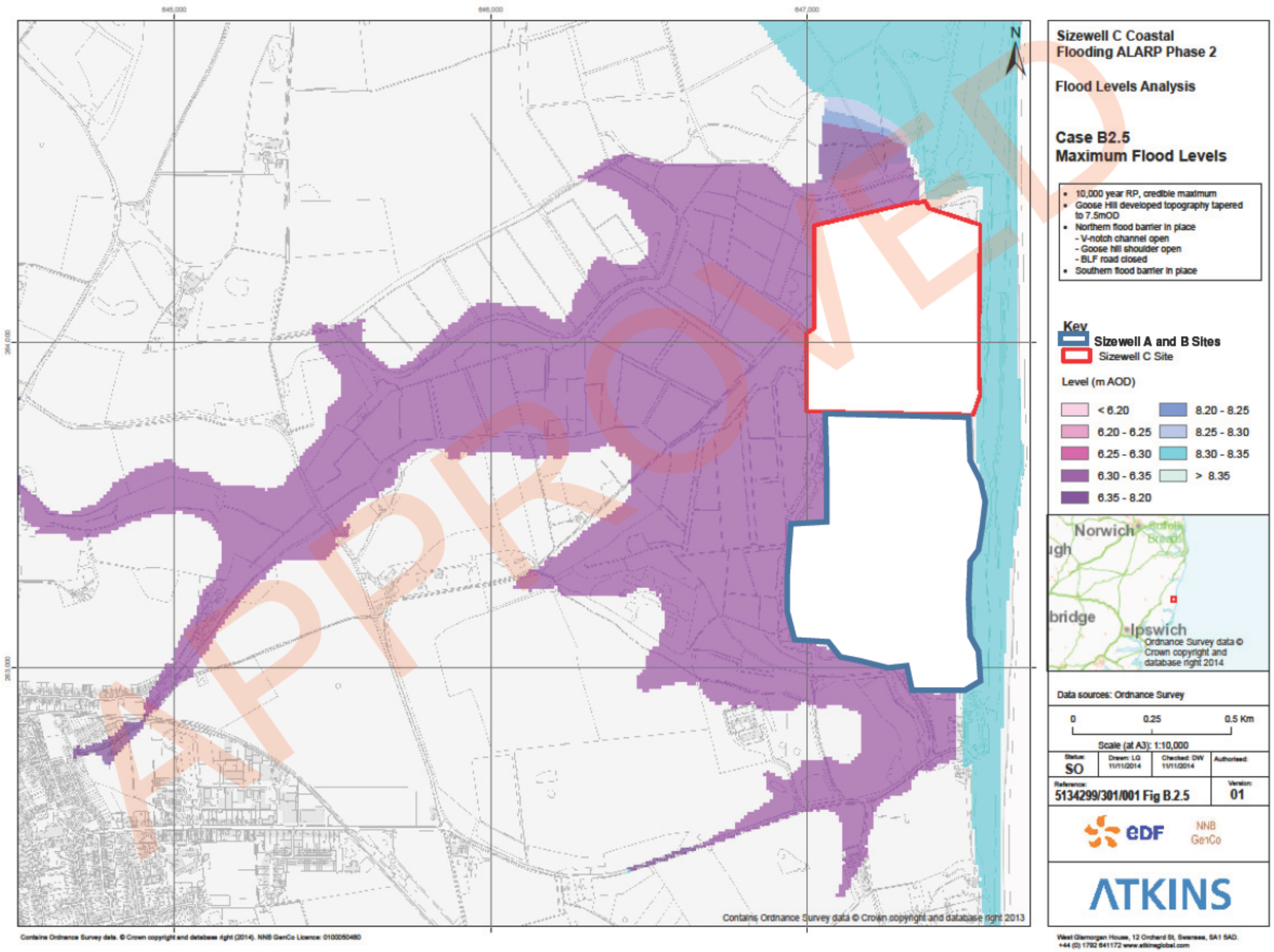
B.2.3. Only Shoulder of Goose Hill at Northern Barrier Open (tapered to 7.5m AOD at bridge)



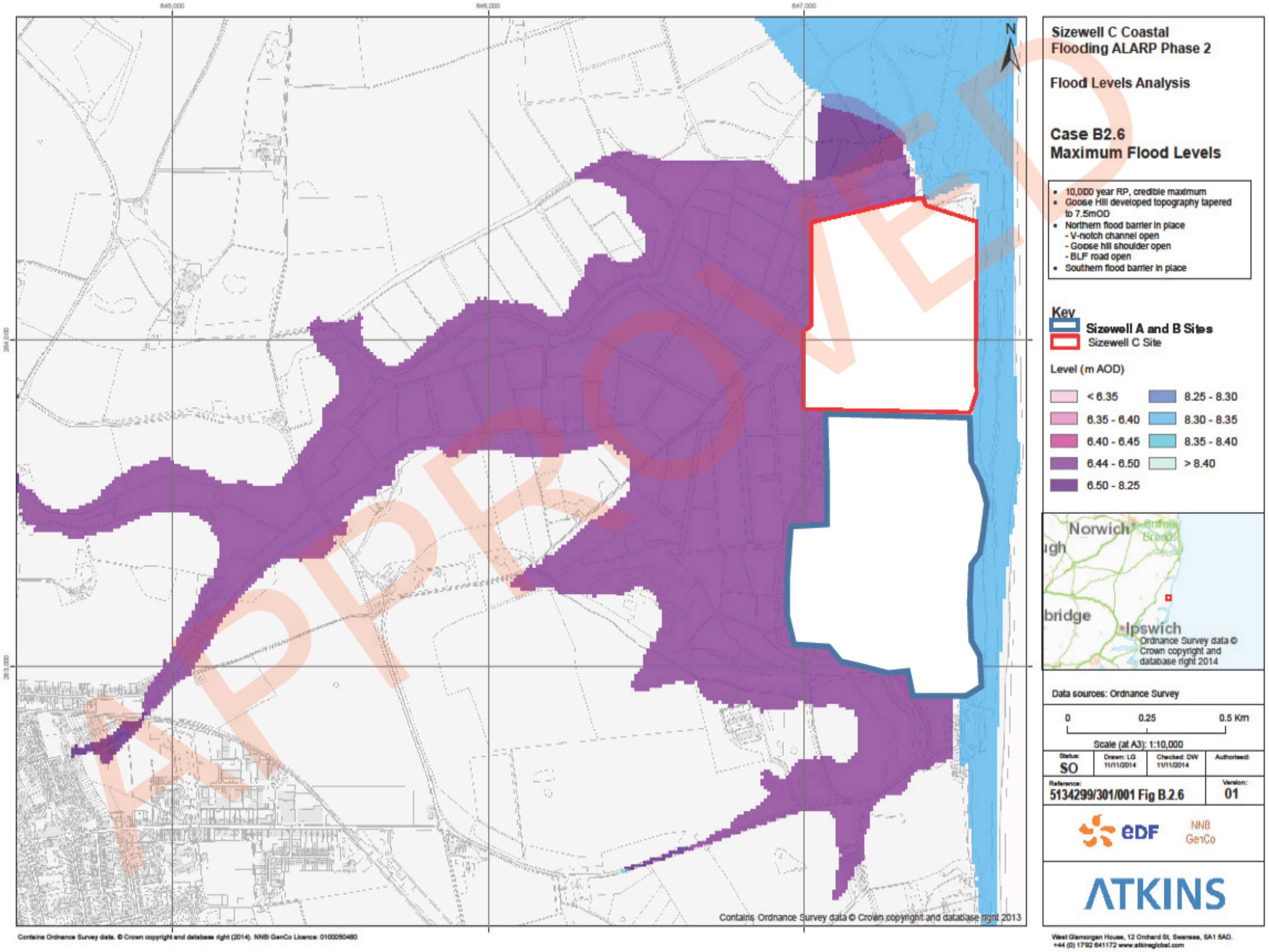
B.2.4. Only BLF Road through Northern Barrier Open



B.2.5. Only Shoulder of Goose Hill (tapered to 7.5m AOD at bridge) and V-Notch Channel in Northern Barrier Open



B.2.6. Shoulder of Goose Hill (tapered to 7.5m AOD), BLF Road and V-Notch Channel in Northern Barrier Open



Appendix C. TUFLOW Model Output Time Histories (with Barriers)

Case numbering (2.1, 2.2 etc.) matches that in Appendix B.

All outputs are for 10,000 year return period sea conditions with credible maximum climate change to 2110.

C.1. Case 2.1 - Only Southern Flood Pathway Open

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 6.4mAOD
- Northern flood barrier in place
  - V-notch channel closed
  - Goose hill shoulder closed
  - BLF road closed
- Southern flood barrier omitted

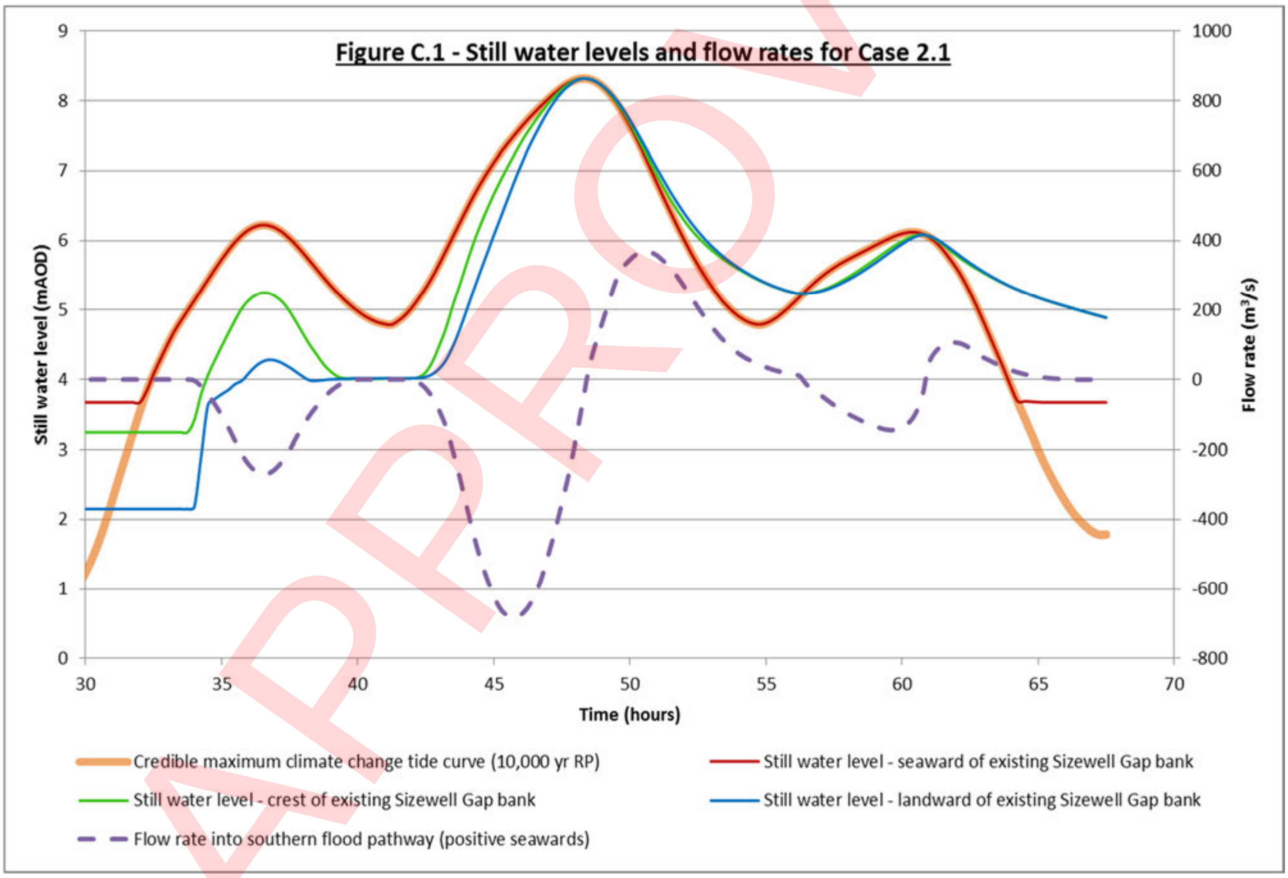


Figure C.1 - Still water levels and flow rates for Case 2.1

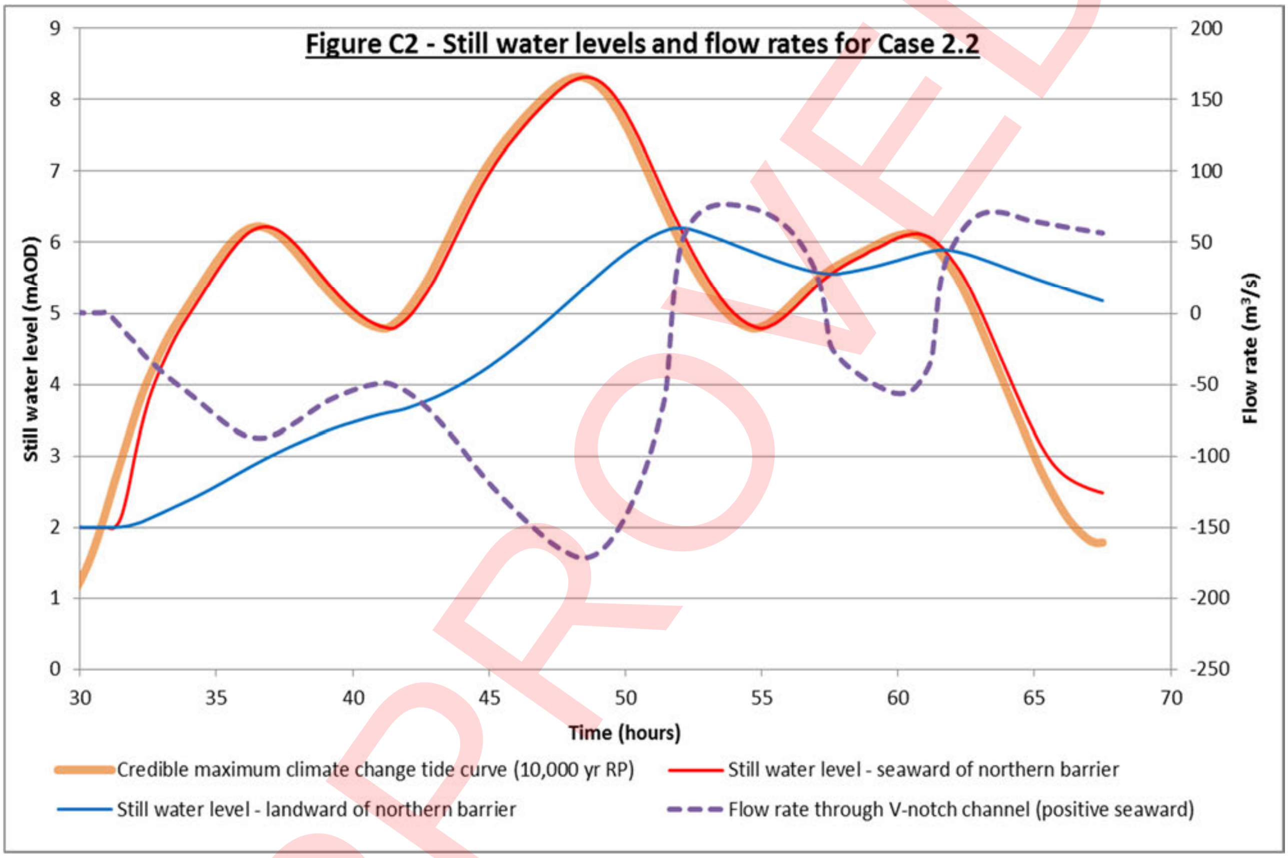
Key points and observations

- The peak landward flow rate through Sizewell Gap is 683m^3/s.
- The total volume flowing landward through Sizewell Gap is 12,978,073m^3.
- The maximum flood level over Sizewell Belts matches the peak tidal level.
- Flow velocities through Sizewell Gap reach 1.4m/s.

C.2. Case 2.2 - Only V-Notch Fluvial Channel in Northern Barrier Open

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 6.4mAOD
- Northern flood barrier in place
  - V-notch channel open
  - Goose hill shoulder closed
  - BLF road closed
- Southern flood barrier in place



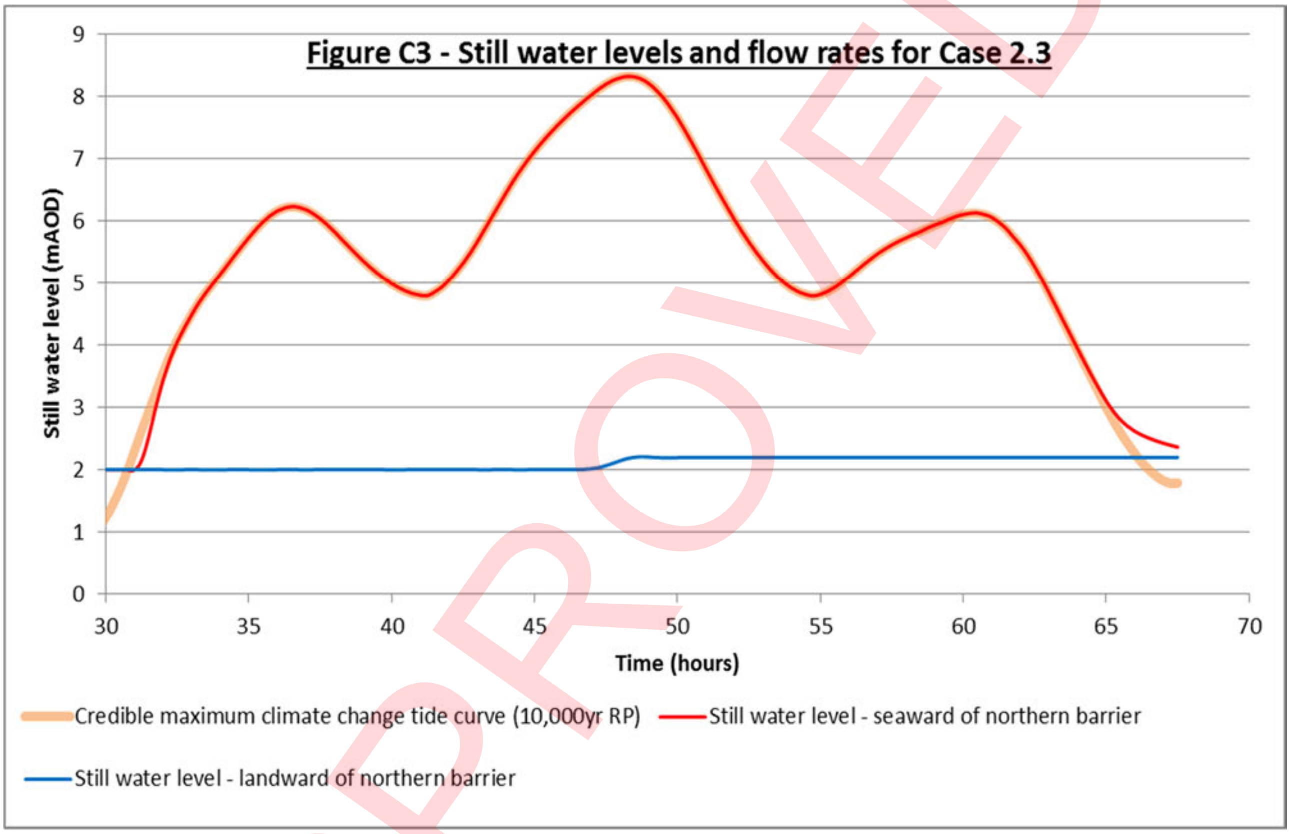
Key points and observations

- The peak landward flow rate through the V-notch channel is 170m^3/s.
- Flow velocities through the V-notch channel reach 5.2m/s.

C.3. Case 2.3 - Only Shoulder of Goose Hill at Northern Barrier Open (tapered to 7.5m AOD at bridge)

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 7.5mAOD
- Northern flood barrier in place
  - V-notch channel closed
  - Goose hill shoulder open
  - BLF road closed
- Southern flood barrier in place



Key points and observations

- The peak landward flow rate over Goose Hill shoulder is 34m^3/s.
- The total volume flowing landward over Goose Hill shoulder is 72,526m^3.
- Flow velocities over Goose Hill reach 0.6m/s.

C.4. Case 2.4 - Only BLF Road through Northern Barrier Open

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 6.4mAOD
- Northern flood barrier in place
    - V-notch channel closed
    - Goose hill shoulder closed
    - BLF road open
- Southern flood barrier in place

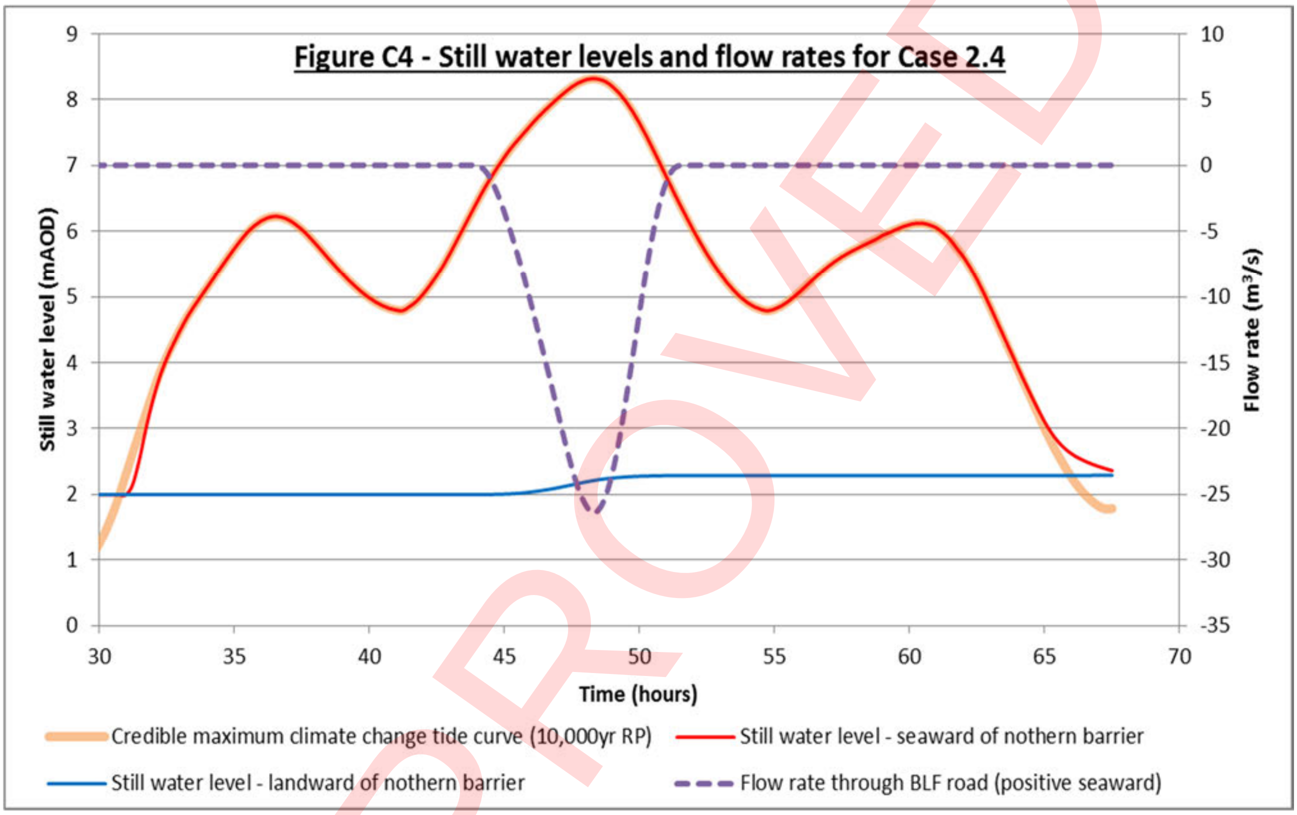


Figure C4 - Still water levels and flow rates for Case 2.4

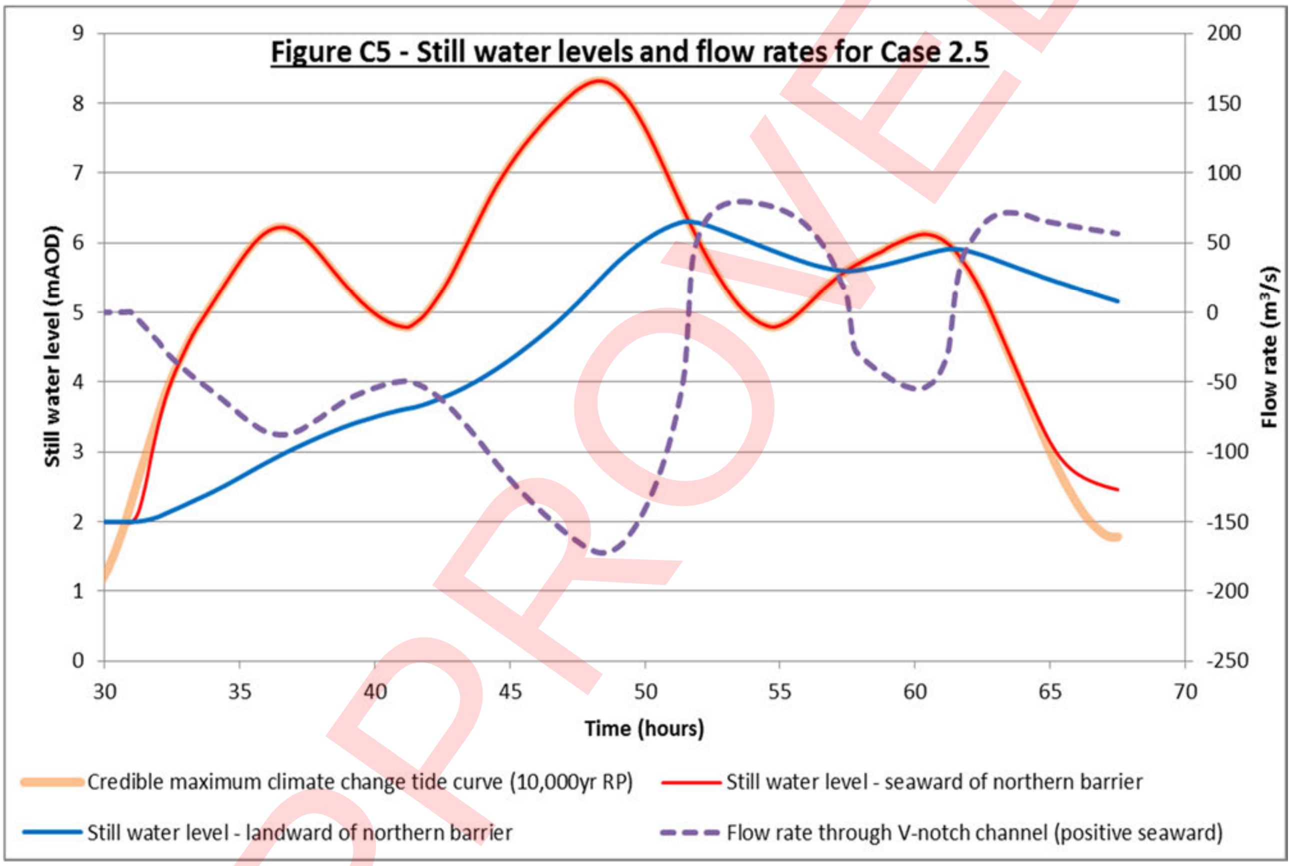
Key points and observations

- The peak landward flow rate through the BLF road is 26m^3/s.
- The total volume flowing landward through the BLF road is 341,240m^3.
- Flow velocities through the BLF road reach 2.7m/s

C.5. Case 2.5 - Only Shoulder of Goose Hill (tapered to 7.5m AOD at bridge) and V-Notch Channel in Northern Barrier Open

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 7.5mAOD
- Northern flood barrier in place
  - V-notch channel open
  - Goose hill shoulder open
  - BLF road closed
- Southern flood barrier in place



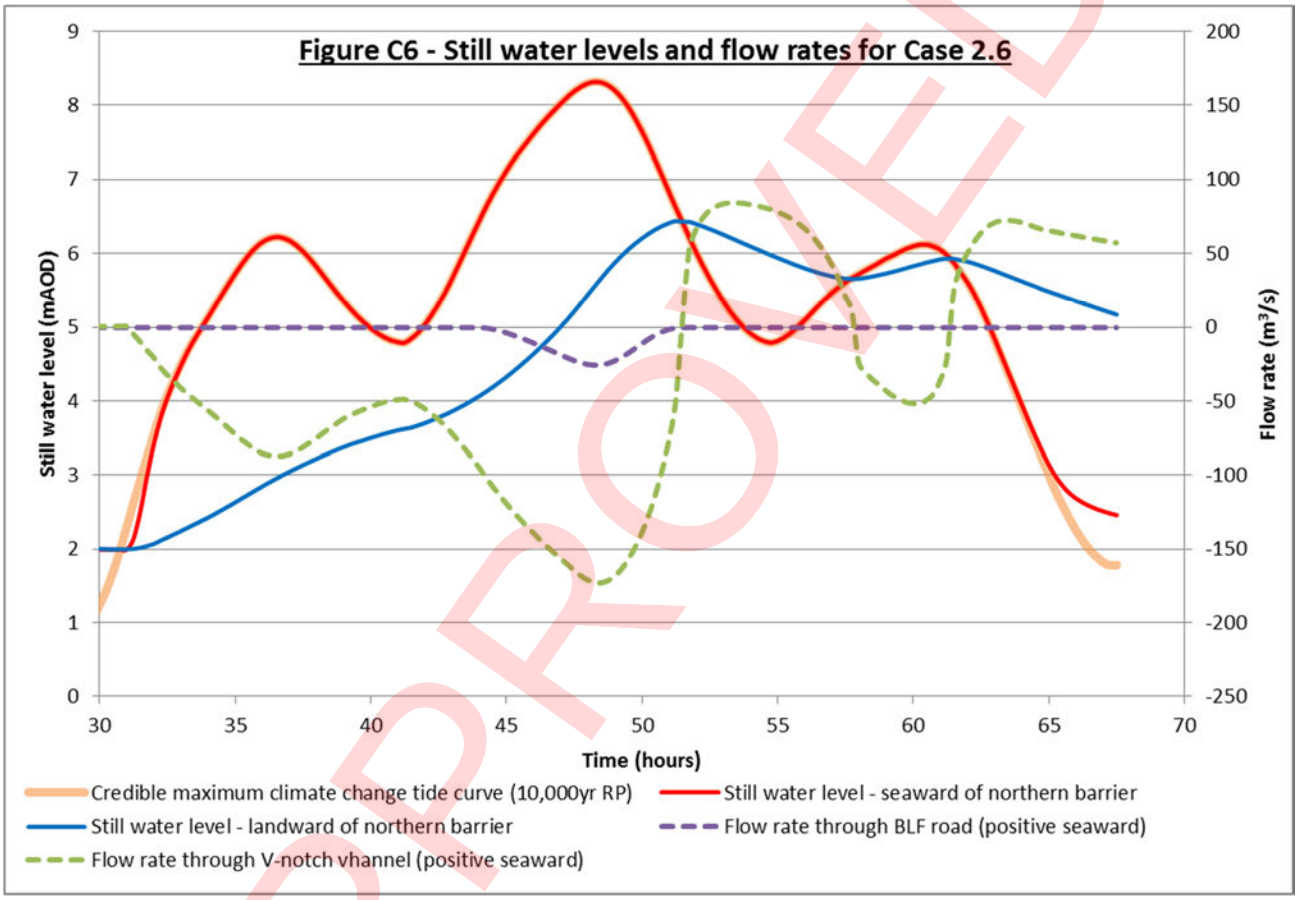
Key points and observations

- The peak landward flow rate through the V-notch channel is 172m^3/s.
- The total volume flowing landward through the V-notch channel is 7,051,122m^3.
- Flow velocities through the V-notch channel reach 5.7m/s.
- The peak landward flow rate over Goose Hill shoulder is 34m^3/s.
- The total volume flowing landward over Goose Hill shoulder is 120,858m^3.
- Flow velocities over Goose Hill reach 0.6m/s.

C.6. Case 2.6 - Shoulder of Goose Hill (tapered to 7.5m AOD), BLF Road and V-Notch Channel in Northern Barrier Open

Case description

- 10,000 year return period, credible maximum climate change
- Goose Hill developed topography tapered to 7.5mAOD
- Northern flood barrier in place
    - V-notch channel open
    - Goose hill shoulder open
    - BLF road open
- Southern flood barrier in place



Key points and observations

- The peak landward flow rate through the V-notch channel is 173m^3/s.
- The total volume flowing landward through the V-notch channel is 7,051,122m^3.
- Flow velocities through the V-notch channel reach 5.7m/s.
- The peak landward flow rate over Goose Hill shoulder is 34m^3/s.
- The total volume flowing landward over Goose Hill shoulder is 120,858m^3.
- Flow velocities over Goose Hill reach 0.6m/s.
- The peak landward flow rate through the BLF road is 26m^3/s.
- The total volume flowing landward through the BLF road is 341,240m3.

[redacted]
Atkins Limited
The Hub, 500 Park Avenue, Aztec West, Almondsbury, BS32 4RZ

Email: [redacted]
Telephone: [redacted]