

Overstrand to Walcott Strategy Study

Do Nothing - Flood Probability and Flood Losses

Part II: Technical Support Information

**Report EX 4692
October 2004**

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This report describes work commissioned by North Norfolk District Council whose representative was Mr Peter Frew. The HR Wallingford job numbers were CDR3212 and CDR3214. The work was carried out by Vincent Grimm, Douglas Benwell, and Dr Cliff Ohl of the Engineering Systems and Management Group at HR Wallingford. The HR Wallingford project manager was Mr Paul Sayers.

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Summary

Overstrand to Walcott Strategy Study

Flood Probability and Flood Losses

Part II: Technical Support Information

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This report addresses the occurrence of overtopping, coastal flooding, and flood losses within the area covered by the Overstrand to Walcott Strategy Study, as indicated by the Environment Agency's indicative flood map. Existing information on sea defences was collated, and predictions of overtopping rates and volumes were then made. These calculations use the hydraulic loading conditions derived elsewhere within the strategy study, in terms of tables of extreme combinations of waves and water levels.

Coupled with a digital terrain model constructed from LiDAR data, the calculated overtopping rates were used as input to the numerical flood-spreading program LISFLOOD-FP. LISFLOOD-FP was used to estimate the flood extent and depth across the study area for several potential flood events, and the results are presented here graphically through GIS plots.

Economic damages have been calculated through a depth-damage relationship as given in the Multi-Coloured Manual published by Middlesex University's Flood Hazard Research Centre. The Present Value of the total flood damages over 100 years has been calculated as £4,176k. The expected annual damages are capped in year nine as the cumulative non-discounted losses reach the asset write off value of £5,025k, therefore, all expected annual damages in subsequent years are ignored

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1. INTRODUCTION

Within the area covered by the Overstrand to Walcott Strategy Study, coastal flooding is an issue of concern in the vicinity of the coastal town of Walcott, as indicated by the Environment Agency's indicative flood map (IFM). The sea defences at this location consist of a sloped concrete sea wall with a wave return wall at the crest, adjacent to the coastal road.

Calculations of overtopping for this area have been conducted through the following three steps:

1. Generation of the wave climate in the study area to obtain the wave condition at the toe of the structure;
2. Determination of the characteristic geometry of the coastal defence (usually through a site survey);
3. Overtopping calculations, dependent upon the type of wall considered.

The results of the overtopping calculations can be used to determine the following:

- Safety of a promenade during a storm;
- Potential risk for structures behind the coastal defence; and
- Volume of water overtopping the structure and flooding the protected area (for example, for flood inundation and damage calculations).

The numerical flood-spreading program LISFLOOD-FP, developed at Bristol University, was used to calculate the flood extent and depth generated by these overtopping volumes. Subsequently, guidance from the Multi-Coloured Manual (FHRC 2003) has been used in the estimation of flood damages and expected annual damages for the present day.

To assess the effects of future climate change, the initial overtopping calculations have been revisited to allow for the effects of rising sea levels. Finally, the effect of such increased water levels on flood damages up to 100 years into the future has been estimated for use in estimating the benefits of coastal defence along this frontage.

2. OVERTOPPING

2.1 Waves, Water Levels and Joint Probability

The generation of the offshore wave climate in the study area is described in the accompanying hydrodynamics report. This report describes prediction of offshore wave conditions using the numerical model HINDWAVE, which simulates the growth of waves under the action of offshore winds. The deformation of the offshore waves as they travel towards the shoreline can then be determined to produce a nearshore wave climate from the calculated offshore wave conditions. With a digital representation of the offshore seabed, the TELURAY numerical model was used to calculate wave conditions for a range of 10 nearshore points, all at -3.25mOD. The offshore wave conditions were assumed to occur (uniformly) along the seaward boundary of this grid, and the model output was the nearshore wave height, period, and direction at these 10 nearshore points.

From the nearshore wave conditions, the wave conditions at the toe of the structure may be obtained through formulas developed by Goda (1985). These formulas take into account the foreshore slope (in this case assumed to be 1 in 80) and the location of the toe of the structure (assumed to be at 2.45mOD, see Figure 3.1).

Both water level and wave height have important impacts on the overtopping discharge. As various return periods are associated with each parameter, it is necessary to use the joint probability return period associated with a pair of wave height and water level values. For a given joint probability return period there are several combinations which range from low water level with high waves to high water level with small waves. The return periods used for this study are 1, 10, 100, 1000 years and the number of combinations for each return period varies between 6 and 11 for the 1 year and 1000 year joint probability return periods, respectively. The wave data used in the overtopping calculations are listed in Table 2.1. For cases where the water level was lower than the assumed toe of the structure (2.45mOD) no further calculations were made.

Table 2.1 Joint probability wave data for return periods used for the overtopping study.

Joint Probability Return Period (years)	Water level (mOD)	Mean Wave Period, T_m (s)	Nearshore Significant Wave Height, H_{sn} (m)	Significant Wave Height at structure toe, H_{st} (m)
1	2.13	6.80	4.10	N/A
	2.25	6.60	3.80	N/A
	2.39	6.40	3.50	N/A
	2.49	6.20	3.30	0.34
	2.61	5.80	2.90	0.37
	2.71	5.50	2.60	0.39
10	2.35	7.60	5.00	N/A
	2.49	7.30	4.60	0.46
	2.61	7.10	4.30	0.50
	2.71	6.70	4.00	0.52
	2.88	6.50	3.70	0.59
	3.07	6.20	3.30	0.66
	3.24	5.90	3.00	0.72
100	2.49	8.20	5.80	0.58
	2.61	8.00	5.50	0.62
	2.71	7.80	5.30	0.65
	2.88	7.60	5.00	0.71
	3.07	7.30	4.60	0.78
	3.24	7.00	4.40	0.85
	3.39	6.80	4.10	0.90
	3.58	6.50	3.70	0.97
	3.79	6.30	3.40	1.06
1000	2.61	8.80	6.60	0.72
	2.71	8.60	6.30	0.75
	2.88	8.40	6.10	0.82
	3.07	8.20	5.80	0.90
	3.24	8.10	5.60	0.97
	3.39	7.90	5.40	1.03
	3.58	7.60	5.00	1.10
	3.79	7.40	4.70	1.19
	3.94	7.00	4.40	1.23
	4.12	6.80	4.10	1.30
	4.37	6.60	3.80	1.42

2.2 Overtopping calculations

A typical wall cross-section for the study area is shown in Figure 2.1, with approximate dimensions.

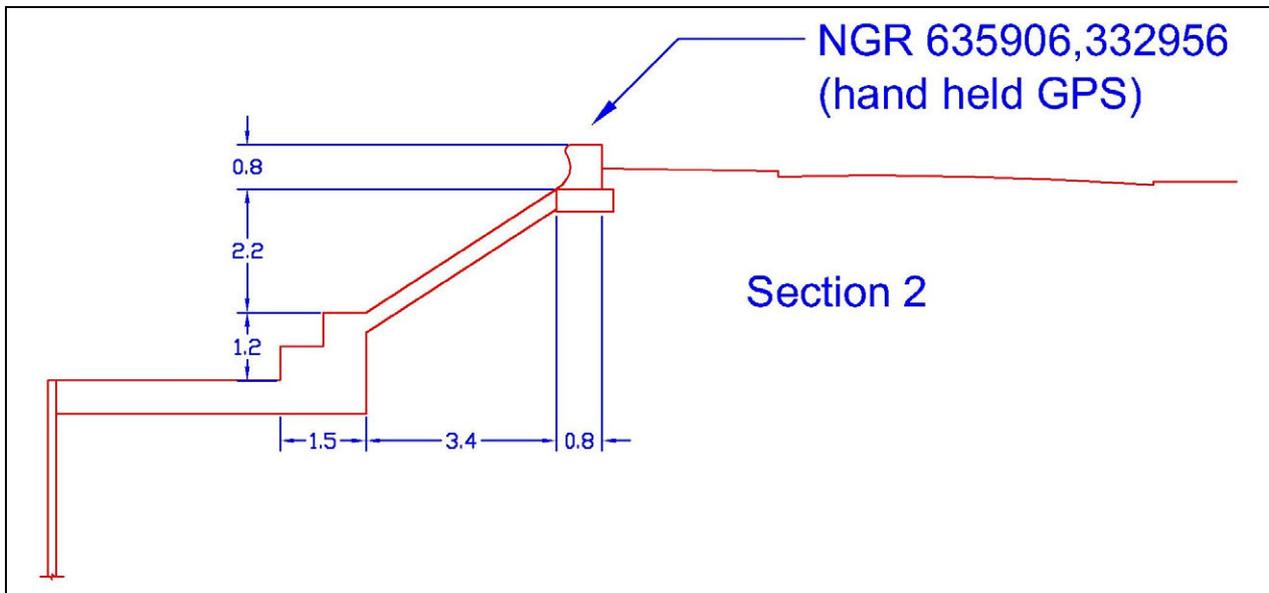


Figure 2.1 Typical seawall cross-section at Walcott

The dimensions of the seawall and the return wall are required for calculation of the overtopping variables. The dimensions used in the overtopping calculations are listed in Table 2.2. As the seawall is made of concrete, the seawall was considered impermeable with a roughness of one, indicating a relatively smooth surface.

Table 2.2 Seawall dimensions used for the overtopping calculations.

Variable	Value	Units
Seawall crest level	5.85	mOD
Return wall height	0.8	m
Seawall toe level	2.45	mOD
Seawall slope	1 in 1.43	
Foreshore slope	1 in 80	

The mean overtopping rate or discharge rate, Q ($\text{m}^3/\text{s}/\text{m}$), and peak or maximum overtopping volume, V_{max} (m^3/m), are required to estimate the likelihood of damage to infrastructure or the general public. The values for these variables for a simply sloping seawall with a return wall have been calculated using the data in Table 2.2 and the approach described by Besley (1999) in the overtopping manual. For completeness, the overtopping equations used to derive the results given in this report are given below.

The first parameter calculated was the mean discharge rate of the seawall. The case of the return seawall is quite complex and was therefore solved in steps. The first step was to calculate the discharge rate at the base of the return wall. This discharge rate, Q_b , (m^3/s per metre run of the seawall) was calculated using the following equation

$$Q_b = Q_b * T_m g H_s,$$

such that

$$Q_b^* = A \exp(-BA_c^*),$$

and

$$A_c^* = A_c / (T_m (gH_s)^{0.5}),$$

where

- A_c is the freeboard of the top of the slope (m);
- H_s is the significant wave height at the toe of the seawall (m);
- T_m is the mean wave period at the toe of the seawall (s);
- g is the acceleration due to gravity (m/s^2);
- A, B are empirical coefficients dependent upon the cross-section of the seawall.

Having calculated the discharge rate incident on the return wall, the influence of the return wall was then considered. The effects of the return wall height (and of its location relative to the sloping seawall crest) in reducing the overtopping discharge were included through adjustment factors obtained from tables within the overtopping manual (Besley, 1999). The mean overtopping discharge rate of the sloping seawall with return wall, Q , was then calculated using

$$Q = Q_b D_f,$$

where D_f is the discharge factor calculated from the tables in the overtopping manual (Besley, 1999).

The other variable calculated for the study was the overtopping volume of the maximum individual overtopping event within a storm, V_{max} , which is used to determine risk of damage or injury. This was calculated as shown by Besley (1999) and is a function of the number of overtopping waves within the storm, N_{ow} , such that

$$V_{max} = a (\ln[N_{ow}])^{1/b},$$

where a and b are empirical parameters dependent on the structure of the wall and the offshore wave steepness. The value of N_{ow} was calculated using an empirical formula based on the mean discharge rate of the seawall.

2.3 Overtopping rates

Overtopping calculations have been made for two cross sections which only vary from that shown in Figure 2.1 by 13mm in height. Since there is only this small difference between the wall sections only the results for wall section 1 are given in Table 2.3. The mean discharge rate values given for each of the return periods are the maximum values for discharge rates obtained from the range of water level and wave height combinations given in Table 2.1. On average, the results for wall section 2 were 5% less than the results for section 1, while the maximum difference was 10%.

While overtopping calculations provide maximum mean discharge rate (m^3/s) per metre run of seawall, the maximum mean discharge rate for the total wall length can be calculated based on the length of seawall in the study area (778m). Furthermore, the assumption of the duration of a storm can lead to the calculation of the total overtopping volume during that storm. Thus, in Table 2.3, the total volume of water overtopping the seawall structure during a storm has been estimated assuming three potential storm durations:

- 2.5 hours for the minimum storm;
- 3 hours for a typical storm; and
- 3.5 hours for the maximum storm.

These three durations are based on the typical tidal cycle at the study area, with allowance for typical storm surge duration enhancing the standard high water duration.

Table 2.3 Overtopping results

Return period (years)	Maximum mean discharge rate, Q ($m^3/s/m$)	Maximum event overtopping volume, V_{max} (m^3/m)	Total overtopping volume for storm (m^3)		
			Minimum storm (2.5 hours)	Mean storm (3 hours)	Maximum storm (3.5 hours)
1	1.7×10^{-5}	5.3×10^{-5}	120	150	170
10	1.0×10^{-3}	6.6×10^{-4}	7,000	8,400	9,800
100	9.7×10^{-3}	2.7×10^{-3}	68,000	81,000	95,000
1000	3.7×10^{-2}	6.3×10^{-3}	260,000	310,000	360,000

The values given in Table 2.3 can be interpreted in terms of potential damage to seawalls, buildings, or infrastructure, defined by Simm (1991) as a function of the mean discharge rate. For the case of a revetment seawall (which approximates the seawall structure in this study), the limits with respect to the seawall structure itself are:

$Q < 0.05$	No damage
$0.05 < Q < 0.2$	Damage if promenade not paved
$Q > 0.2$	Damage even if promenade paved

Thus, the results in Table 2.3 suggest that no damage to the seawall is to be expected for any of the return period storms investigated.

In calculating the impact of overtopping discharges on buildings, Jensen and Sorensen (1979) recognised a dependence on distance from the seawall. While the rates shown in Table 2.3 are overtopping discharge rates at the seawall structure itself, the buildings present along this section of seawall are at distances of the order of 25m behind the seawall or greater. Thus, based on the exponential decrease defined by Jansen and Sorensen (1979), a value of between 0.01% and 0.1% of the predicted maximum mean overtopping discharge rates given in Table 2.3 can be expected to affect local buildings.

The guidelines on tolerable mean discharge rate limits for buildings (Simm 1991) are:

$Q < 1 \times 10^{-6}$	No damage
$1 \times 10^{-6} < Q < 3 \times 10^{-5}$	Minor damage to fittings, etc...
$Q > 3 \times 10^{-5}$	Structural damage

Therefore, the values in Table 2.3, reduced to account for distance from the seawall structures, indicate that no damage to existing buildings is likely for the 1 and 10 year return period storm condition. However, the 100 and 1000-year return period storms can be expected to cause minor damage to existing buildings (of the order of 25m from the seawall). Furthermore, any other structures (e.g. fences, walls, and sheds) located closer to the seawall may be damaged dependent on their distance from the wall and specific structural properties.

While the tolerable overtopping limits have traditionally been specified in terms of mean discharge rates, for the safety of vehicles and pedestrians it is more relevant to use the peak discharge. This peak discharge is the biggest discharge event likely to happen during the storm. Considering the limits of safety for an individual behind a sea defence, Herbert (1996) conducted tests on volunteers and concluded that a peak discharge of $0.04m^3/m$ for any structure constituted a danger to pedestrians. Herbert also conducted tests on vehicles and established that a peak discharge of greater than $0.06m^3/m$ for any structure was dangerous to moving vehicles. Comparing the values developed by Herbert (1996) to the overtopping results in Table 2.3, the peak overtopping volumes (i.e. maximum event overtopping volume) for each storm do not present a danger to either pedestrians or vehicles.

3. FLOOD SPREADING

With overtopping rates from Table 2.3 and a digital terrain model (DTM) developed from the Environment Agency's Lidar data, the aforementioned program LISFLOOD-FP was used to calculate flood extents and depths.

3.1 Flood extent and depth – LISFLOOD-FP

The raster-based flood modelling code LISFLOOD-FP has been developed at Bristol University as a high performance two-dimensional hydraulic model for predicting flood inundation. Originally developed as a dynamic GIS model in collaboration with Ad de Roo of the EU Joint Research Centre (Bates and De Roo 2000), the program has since been re-coded in c++ and refined (Horritt and Bates 2001). Based on a simple representation of dynamic, flood inundation processes, the model uses high-resolution raster Digital Elevation Models (increasingly available for many lowland floodplain rivers).

To provide a range of results for flood extent and depth and test the sensitivity to storm duration, LISFLOOD-FP was run for 12 scenarios, covering 1, 10, 100, and 100-year return periods with minimum, mean, and maximum storm duration. The resulting geo-referenced flood depths were placed into a GIS database, holding locations of residential properties in the coastal floodplain, which has been used to produce the plots of flood extent and depth in Figures 3.1 to 3.12. As the flood extents and depths given in the figures are based on potential present day overtopping rates and the geometry of the present day defences, the figures represent flood extent and depth at present day.

3.2 Flood damages and expected annual damage

The recently issued Multi-Coloured Manual (FHRC 2003) provides damage estimates for five categories of residential property in the UK (e.g. semi-detached or terraced for various periods of construction). In addition, the manual provides guidance for the estimation of damages and benefits associated with strategies for the prevention or reduction of flooding. Providing damage estimates in £k for inundation depths up to 3m, Table 3.1 provides a summary of the depth damage data used in this study, with lower bound, best estimate, and upper bound values. As all the properties subjected to coastal flooding in the study area are detached residential properties, the damage values used are an average of the damage values quoted in the Multi-Coloured Manual for all property ages in the detached category.

Using the GIS database with geo-referenced flood depths and property locations, flood damages were calculated for all 12 scenarios described above. Flood depth at each property was determined using the average flood depth calculated within a 4m wide buffer zone around each property. This flood depth was then used to derive a damage estimate from Table 3.1, whereby interpolation was used to derive damage estimates for depths between the index values provided by FHRC (2003).

As a quantitative measure of losses due to flood damage, expected annual damage (EAD) has been calculated in accordance with the recommendations of the Multi-Coloured Manual (FHRC 2003). EAD is the area under the loss-probability curve, i.e. the graph of flood losses plotted against exceedance probability (the reciprocal of the return period in years). A sample calculation of EAD is given in Table 3.2, using the mean storm duration and best estimate damage levels. Finally, EAD for all 12 scenarios is given in Table 3.3, indicating approximately 15% difference between the middle value (mean storm / best estimate damage) and the upper and lower extremes (maximum or minimum storm / upper or lower bound damage). Given this relatively small difference in results, mean storm duration and best estimate damage values have been adopted for the remainder of the analysis.

Table 3.1 Flood damage (£k) for detached properties, inundation less than 12 hours duration (FHRC 2003)

Damage Estimate	Depth above ground floor (m)														
	-0.3	0	0.05	0.1	0.2	0.3	0.6	0.9	1.2	1.5	1.8	2.1	2.4	2.7	3
Lower Bound	0.44	0.44	9.9	12	22	25	28	30	32	34	37	39	40	42	42
Best Estimate	0.50	0.50	11	14	24	28	32	34	37	39	41	43	45	47	48
Upper Bound	0.56	0.56	12	15	27	31	35	38	41	43	46	48	50	52	53

Table 3.2 Calculation of EAD (£k) for mean storm duration and best estimate damage levels

Return period (yr)	Exceedance probability (yr ⁻¹)	Damages (£k)	Risk increment (£k)
1	1	49	44
10	0.1	298	27
100	0.01	1316	12
1000	0.001	2292	2

EAD (£k): 85

Table 3.3 EAD (£k) for storms of three durations, using three damage estimate levels

Damage Estimate	Storm Duration (hours)		
	2.5	3	3.5
Lower Bound	73	76	79
Best Estimate	82	85	89
Upper Bound	91	95	99



Figure 3.1 Flood extent and depth, 1-year return period event, minimum storm duration



Figure 3.2 Flood extent and depth, 1-year return period event, mean storm duration



Figure 3.3 Flood extent and depth, 1-year return period event, maximum storm duration

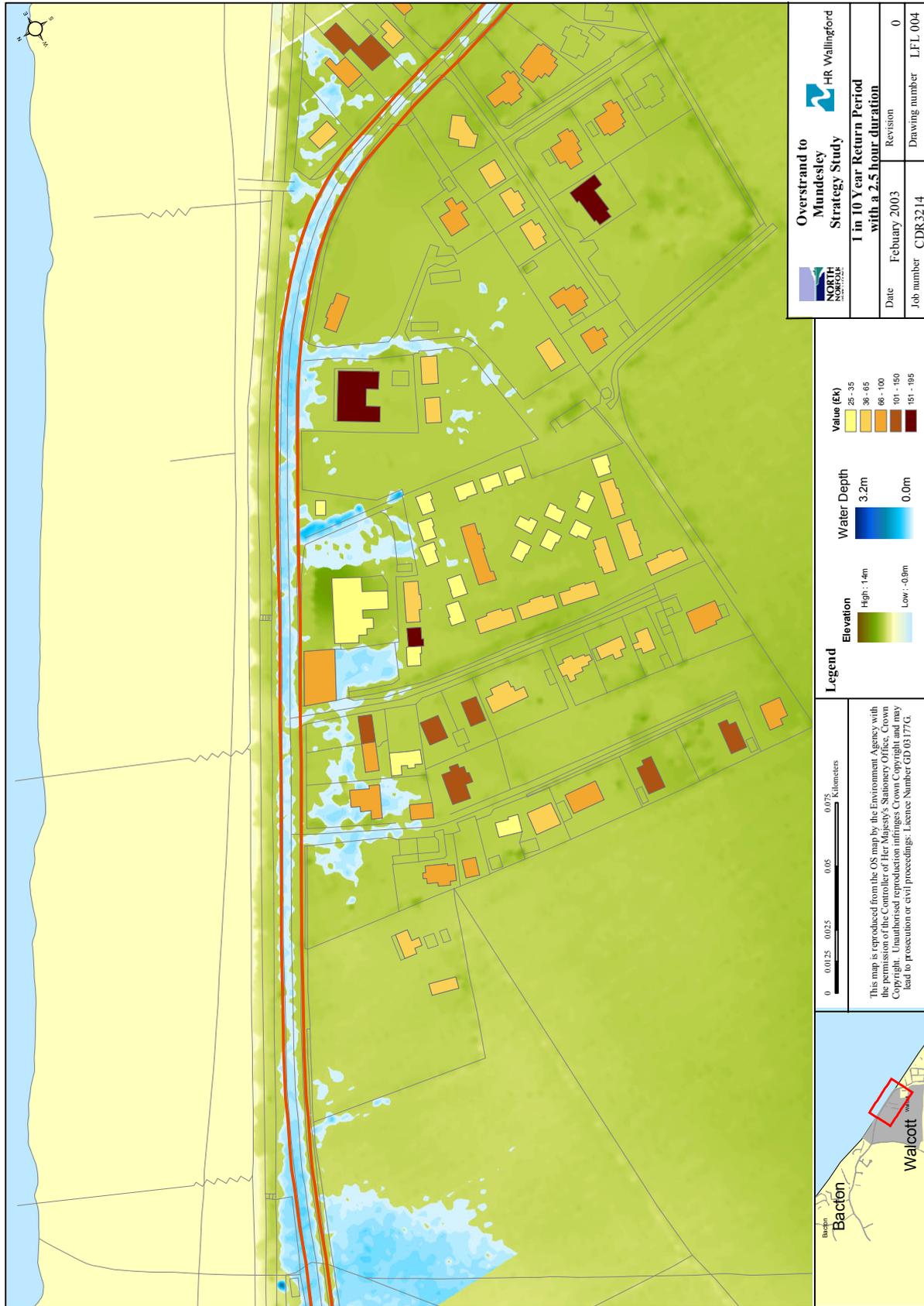


Figure 3.4 Flood extent and depth, 10-year return period event, minimum storm duration

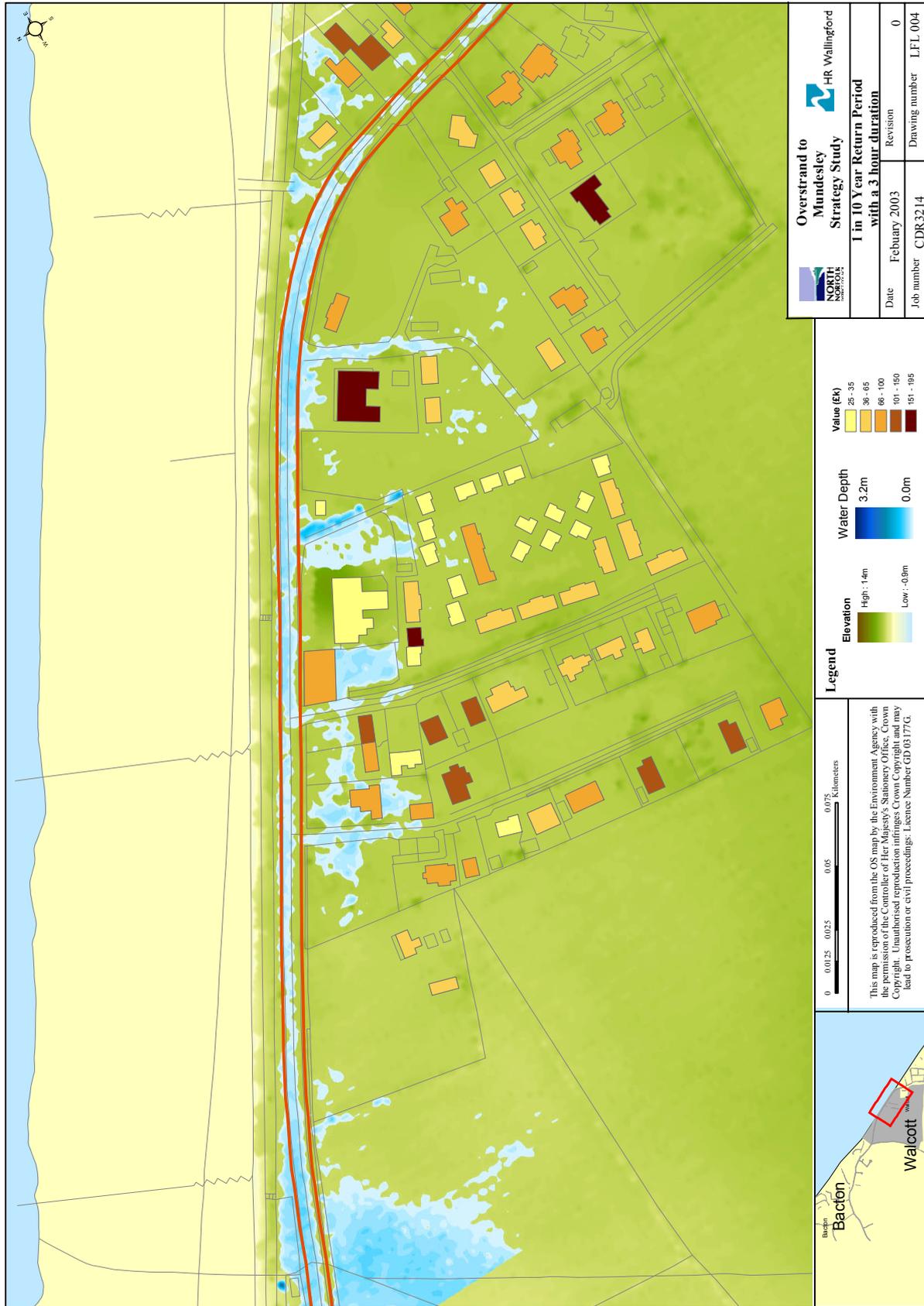


Figure 3.5 Flood extent and depth, 10-year return period event, mean storm duration

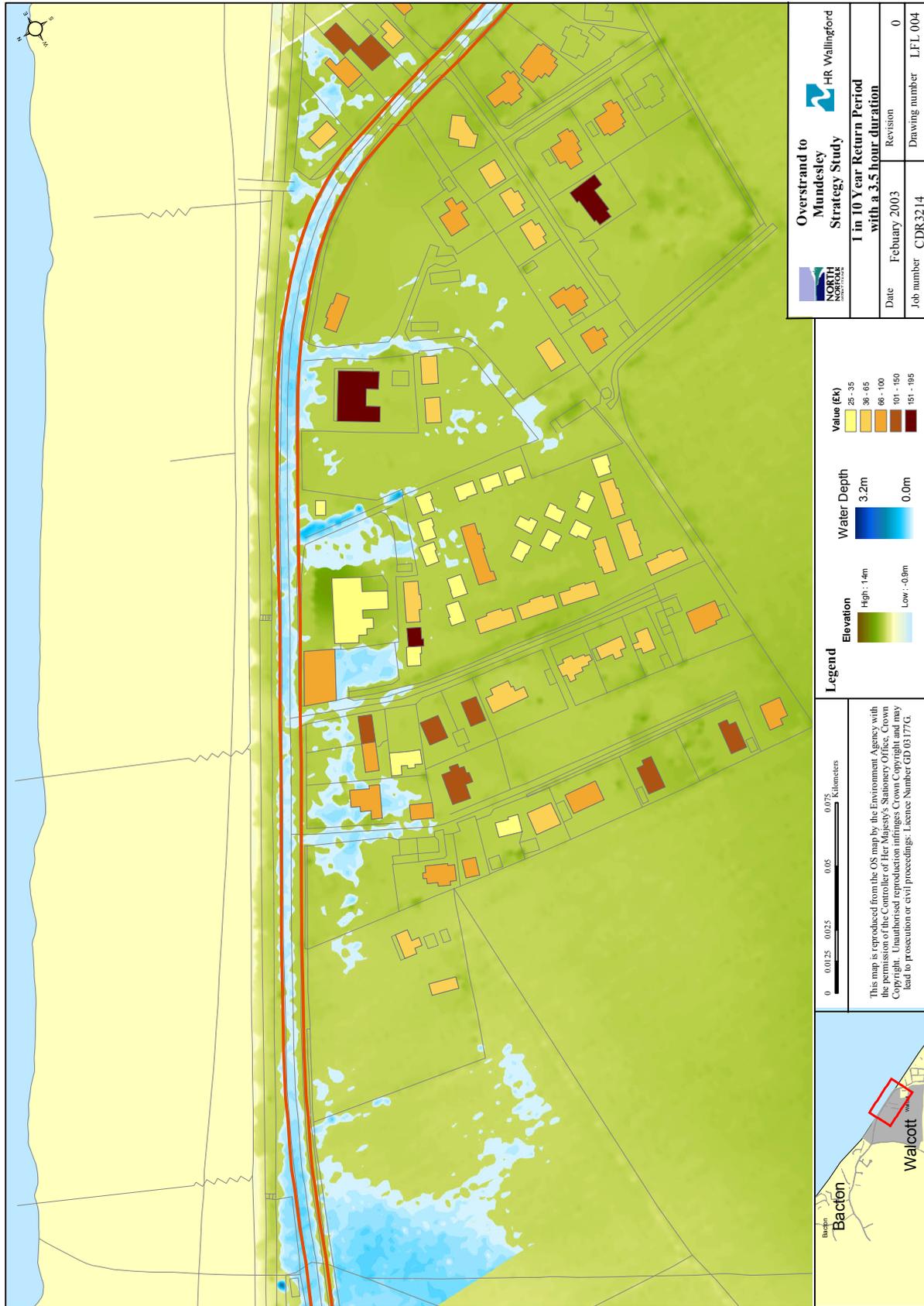


Figure 3.6 Flood extent and depth, 10-year return period event, maximum storm duration

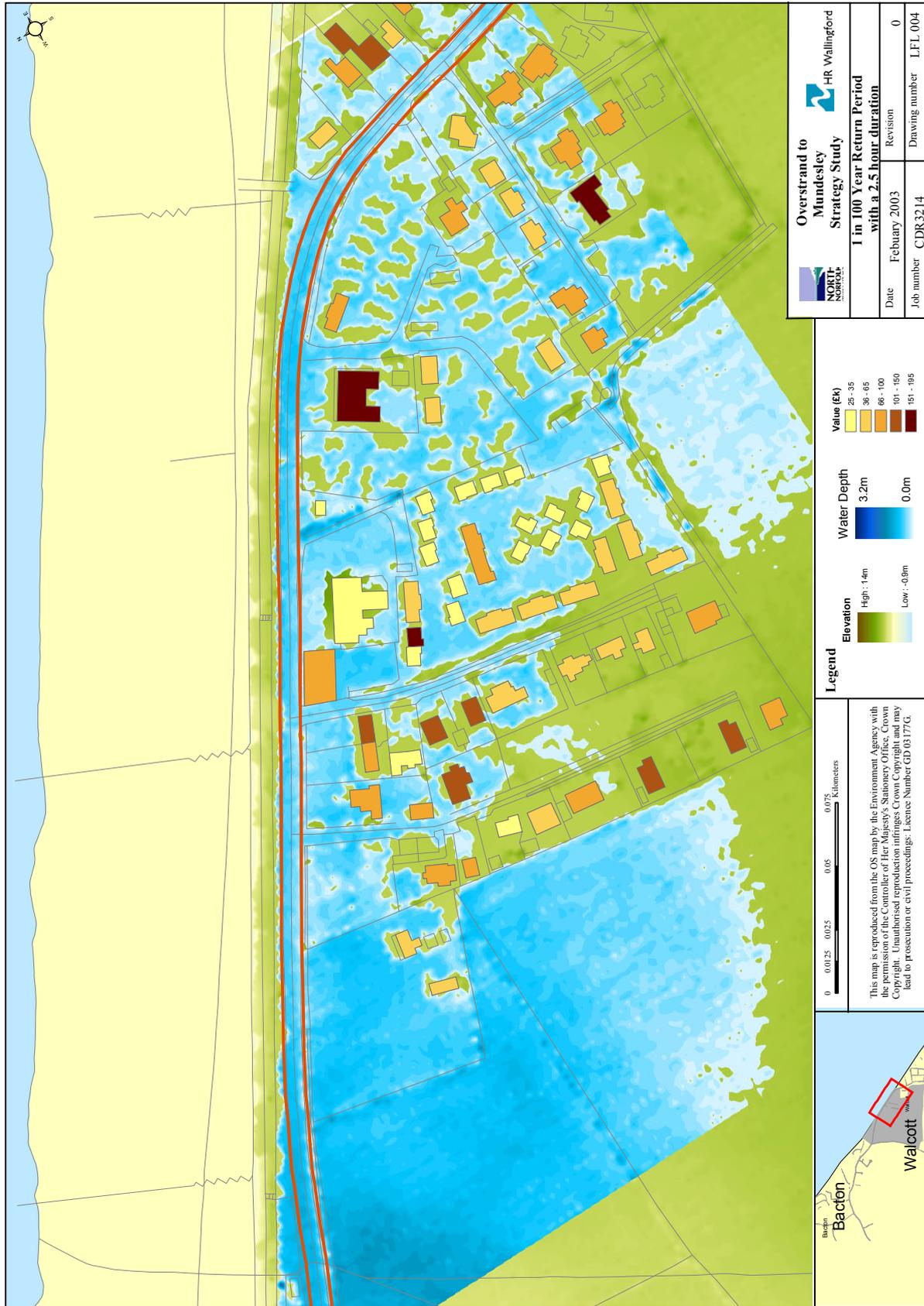


Figure 3.7 Flood extent and depth, 100-year return period event, minimum storm duration

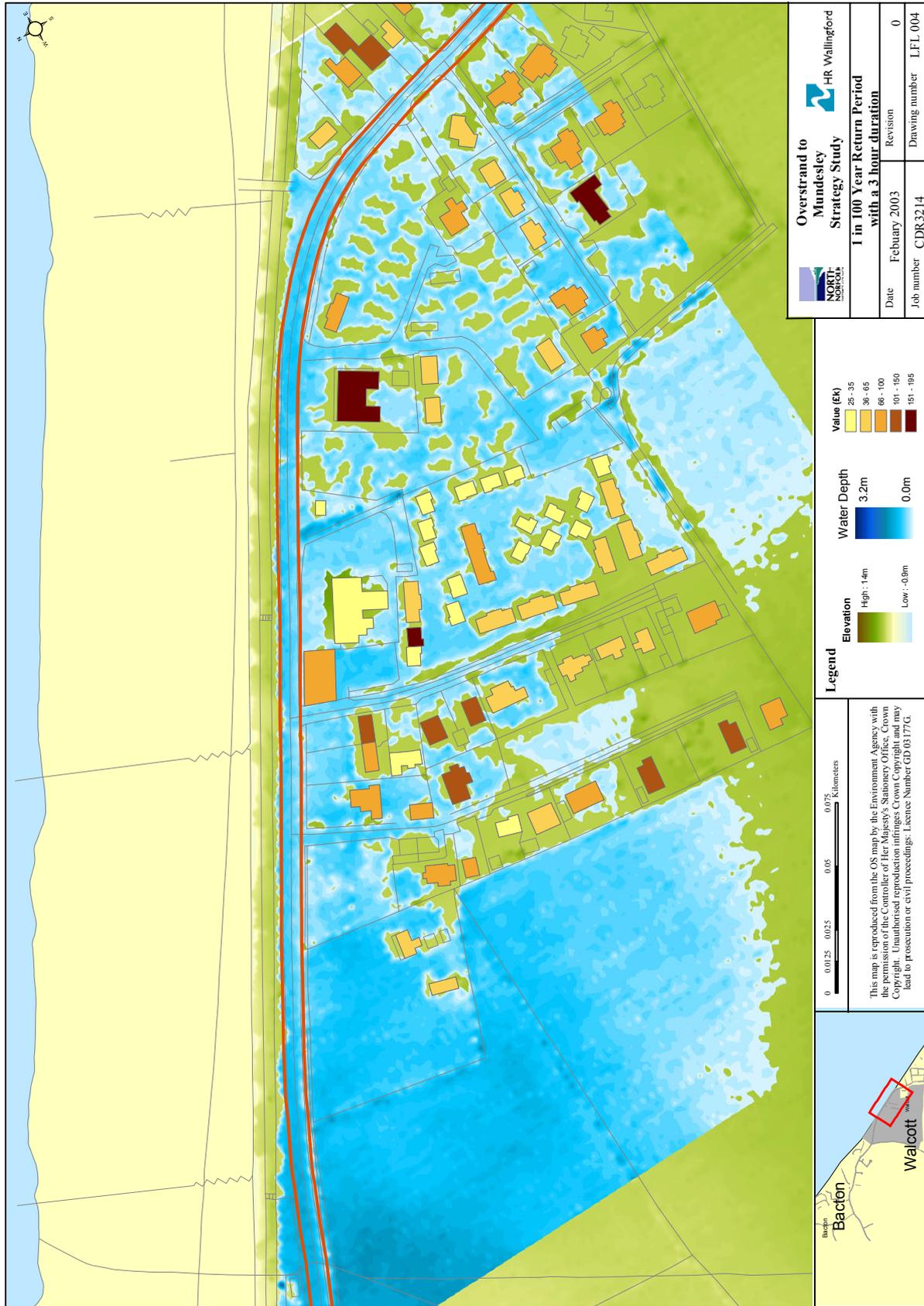


Figure 3.8 Flood extent and depth, 100-year return period event, mean storm duration



Figure 3.9 Flood extent and depth, 100-year return period event, maximum storm duration

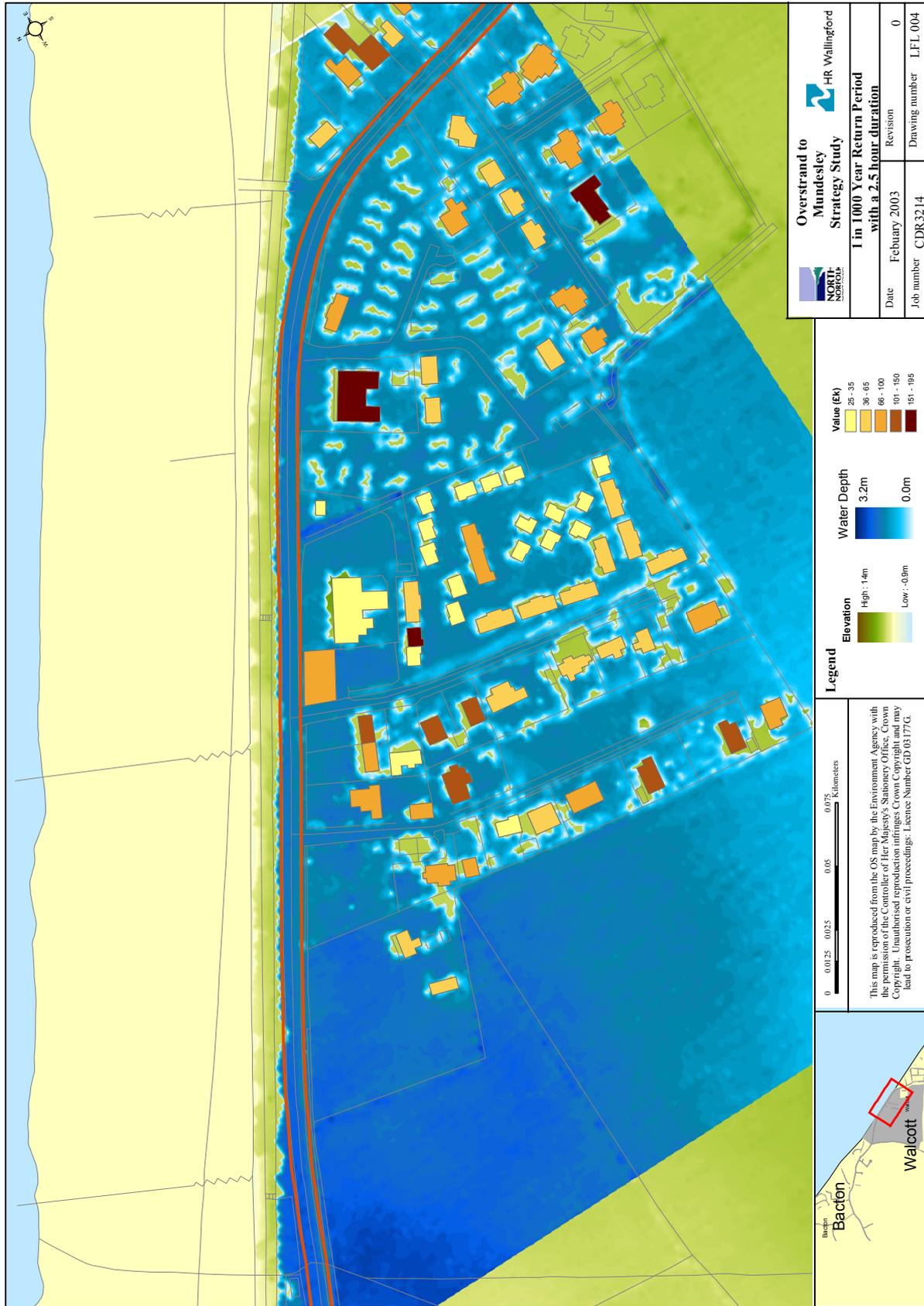


Figure 3.10 Flood extent and depth, 1000-year return period event, minimum storm duration



Figure 3.11 Flood extent and depth, 1000-year return period event, mean storm duration



Figure 3.12 Flood extent and depth, 1000-year return period event, maximum storm duration

4. IMPACT OF CLIMATE CHANGE ON FLOOD DAMAGE

As discussed in the accompanying interim report on hydrodynamics, the effects of climate change on wave and water level conditions in the study area have been assessed in accordance with the recommendations of MAFF (1999). Thus, future mean sea level rise of 6mm/yr has been assumed in the study area, causing a corresponding increase in wave height at the toe of the seawall resulting from the increased water levels. These increased wave heights and water levels will necessarily generate increased overtopping and higher flood damages than reported in Sections 2 and 3.

To assess the potential magnitude of such climate change impact, the following steps were taken:

1. Revised mean discharge rates for the 1, 10, 100, and 1000-year return period events were calculated for future years up to year 100, at intervals of 10 years. This utilised the procedure for calculation of overtopping rates discussed in Section 2.2, where overtopping calculations for the present day, i.e. year 0, are discussed.
2. Flood damage estimates for these future years were made through a comparison of flood damage and mean discharge rate for the year 0 calculation. For the 1, 10, and 100-year events, future year damages were calculated by fitting a cubic spline to the year 0 results. However, a 3rd order polynomial curve was fitted to the year 0 results to estimate damages for the 1000-year events.
3. Finally, EAD in each year was calculated using the procedure outlined in Section 3.2.

Plotting flood damage versus the base 10 logarithm of mean discharge rate in years 0 to 100, Figure 4.1 illustrates the flood damage estimation described in step 2 above. In the figure, four points for each year (0 to 100) represent the flood damage for the four return period events (1, 10, 100, and 1000). Note that mean discharge for the year 100 event (solid blue squares) is typically close to but less than mean discharge for the next higher return period event from year 0 (black, unfilled circles). For example, mean discharge for year 100, return period 10, is less than that for year 0, return period 100.

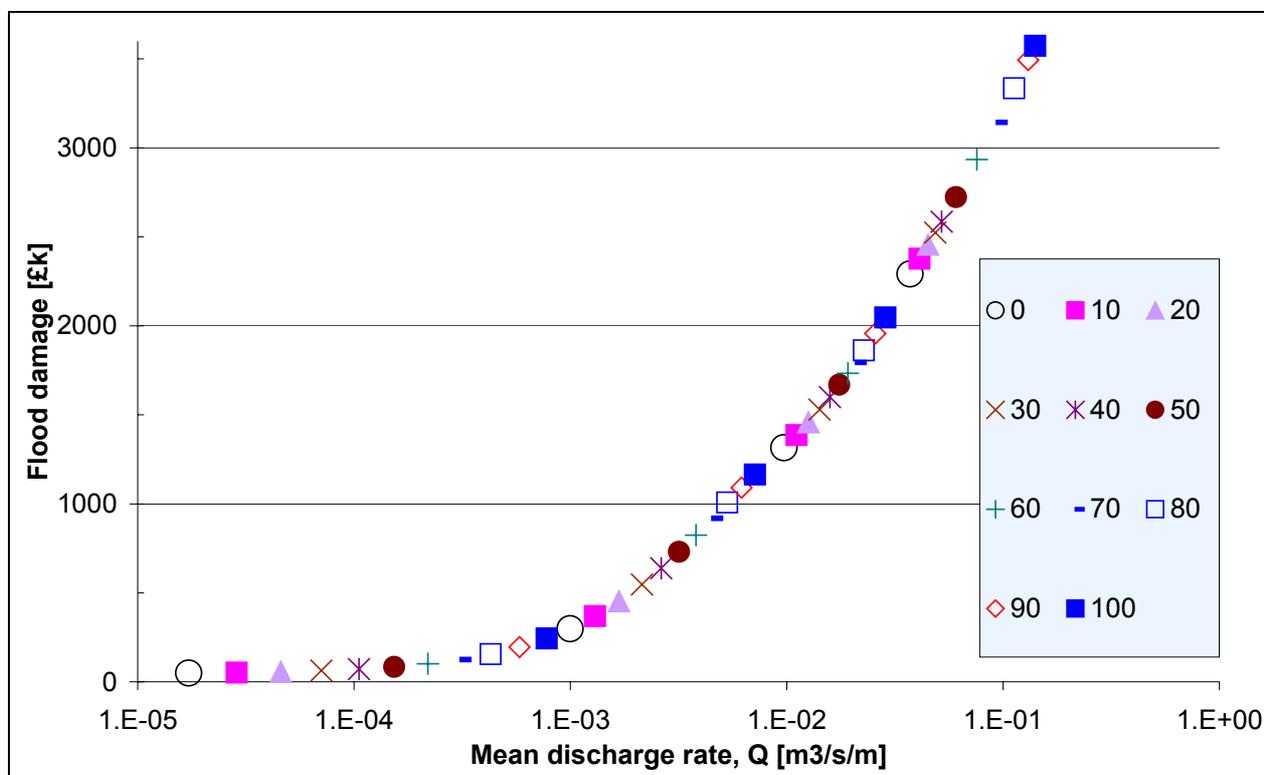


Figure 4.1 Flood damage versus mean discharge rate for 1, 10, 100, and 1000-year return period events in years 0 to 100

Representing the losses due to coastal flooding of residential properties in the study area, the calculated flood damage estimates and projected EAD for years 0 to 100 are listed in Table 4.1. As discussed previously in Section 3, the results in Table 4.1 have been calculated using the mean storm duration (3 hours) and the best estimate damage values from Table 3.1.

Table 4.1 Mean discharge rate, estimated flood damages, and projected EAD to year 100 for present-day seawall geometry

Year	0		10		20		30	
Return Period (yr)	Q (m3/s/m)	Damages (£k)						
1	1.7E-05	49	2.9E-05	52	4.6E-05	57	7.1E-05	63
10	1.0E-03	298	1.3E-03	369	1.7E-03	452	2.1E-03	547
100	9.7E-03	1316	1.1E-02	1386	1.3E-02	1457	1.4E-02	1529
1000	3.7E-02	2292	4.1E-02	2378	4.5E-02	2455	4.9E-02	2524
EAD (£k):		85		95		107		122

Year	40		50		60		70	
Return Period (yr)	Q (m3/s/m)	Damages (£k)						
1	1.1E-04	72	1.5E-04	85	2.2E-04	102	3.1E-04	125
10	2.6E-03	638	3.2E-03	731	3.8E-03	825	4.5E-03	919
100	1.6E-02	1600	1.7E-02	1669	1.9E-02	1734	2.1E-02	1794
1000	5.2E-02	2586	6.0E-02	2723	7.6E-02	2935	9.3E-02	3143
EAD (£k):		139		160		185		215

Year	80		90		100	
Return Period (yr)	Q (m3/s/m)	Damages (£k)	Q (m3/s/m)	Damages (£k)	Q (m3/s/m)	Damages (£k)
1	4.3E-04	156	5.8E-04	195	7.8E-04	245
10	5.3E-03	1008	6.2E-03	1091	7.1E-03	1165
100	2.3E-02	1862	2.6E-02	1958	2.9E-02	2048
1000	1.1E-01	3336	1.3E-01	3493	1.4E-01	3574
EAD (£k):		251		295		347

5. COASTAL FLOODING IN THE DO NOTHING SCENARIO

To assess the benefits of intervention options on the coastline that may mitigate the effects of flooding, it is necessary to understand the losses associated with coastal flooding during the Do Nothing scenario. This scenario is further described in the accompanying report entitled “Do Nothing” – Erosion Probability and Erosion Losses report.

Based on the results of the defence condition survey, the seawall at Walcott has been assessed as in fair condition, with residual life of between 5 to 10 years (see the accompanying ‘Defence condition survey’ report). Furthermore, based on the results of shoreline evolution analysis discussed in the ‘Appraisal of the Do Nothing coastal defence option’ report, the shoreline at Walcott is predicted to be receding at a rate of the order of 0.5m/year. Therefore, to assess future coastal flooding in the Do Nothing scenario, revised overtopping calculations have been conducted for all years following year 10. These revised calculations have been made under the following assumptions:

- With failure of the seawall at year 10, the crest level of the structure effectively drops to the level of the hinterland (implemented in overtopping calculations by removing the wave return wall); and
- Reducing beach levels by 0.5m to account for shoreline recession (implemented in overtopping calculations by reducing the level of the toe of the wall by 0.5m).

Thus, Table 5.1 indicates the alterations to seawall dimensions used for revised overtopping calculations for the Do Nothing scenario, covering year 10 and all subsequent periods (altered from the present day dimensions given in Table 2.2).

Table 5.1 Altered seawall dimensions for Do Nothing scenario overtopping calculations, years 10+

Variable	Value	Units
Seawall crest level	5.85	mOD
Return wall height	0	m
Seawall toe level	1.95	mOD
Seawall slope	1 in 1.43	
Foreshore slope	1 in 80	

In addition to the above changes to seawall geometry at year 10, the effects of climate change on overtopping rate in future years must be considered as discussed in Section 4. With such allowances for the Do Nothing scenario, the losses due to coastal flooding of residential properties in the study area, the calculated flood damage estimates, and projected EAD for years 0 to 100 are listed in Table 5.2. These values may be used to estimate the benefits of continued provision of sea defence along this frontage. As discussed previously in Section 3, the results in Table 5.2 have been calculated using the mean storm duration (3 hours) and the best estimate damage values from Table 3.1. Note that flood damages in a single event have been capped at a maximum value of £5,025k, which is equivalent to the write-off value of the property in the flood plain. This is pertinent only to the highest damage events listed in Table 5.2 (i.e. 100-year return period events in year 60 and beyond).

The Present Value of the total flood damages over 100 years has been calculated as £4,176k (see Table 5.3). The expected annual damages are capped in year nine as the cumulative non-discounted losses reach the asset write off value of £5,025k, therefore, all expected annual damages in subsequent years are ignored. The PV damages per year are shown for the 100-year strategy in Table 5.3.

Table 5.2 Mean discharge rate, estimated flood damages, and projected EAD to year 100 for the Do Nothing scenario

Year	0		10		20		30	
Return Period (yr)	Q (m ³ /s/m)	Damages (£k)						
1	1.7E-05	49	5.2E-03	992	6.5E-03	1115	8.0E-03	1223
10	1.0E-03	298	3.5E-02	2232	4.0E-02	2357	4.6E-02	2471
100	9.7E-03	1316	1.3E-01	3514	1.5E-01	3618	1.6E-01	3720
1000	3.7E-02	2292	3.5E-01	4638	3.7E-01	4730	4.0E-01	4821
EAD (£k):		85	1130		1253		1362	

Year	40		50		60		70	
Return Period (yr)	Q (m ³ /s/m)	Damages (£k)						
1	9.8E-03	1322	1.2E-02	1425	1.4E-02	1535	1.7E-02	1650
10	5.2E-02	2584	5.9E-02	2697	6.6E-02	2808	7.4E-02	2919
100	1.8E-01	3822	1.9E-01	3922	2.1E-01	4022	2.3E-01	4121
1000	4.3E-01	4912	4.6E-01	5002	4.9E-01	5025	5.3E-01	5025
EAD (£k):		1462	1566		1675		1790	

Year	80		90		100	
Return Period (yr)	Q (m ³ /s/m)	Damages (£k)	Q (m ³ /s/m)	Damages (£k)	Q (m ³ /s/m)	Damages (£k)
1	2.0E-02	1769	2.4E-02	1890	2.7E-02	2014
10	8.3E-02	3028	9.3E-02	3137	1.0E-01	3245
100	2.5E-01	4219	2.7E-01	4316	2.9E-01	4412
1000	5.6E-01	5025	6.0E-01	5025	6.4E-01	5025
EAD (£k):		1907	2027		2149	

Year	0	1	2	3	4	5	6	7	8	9
Discount rate	0	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Annual Discount	1	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035
Product of discounts	1	0.966	0.934	0.902	0.871	0.842	0.814	0.786	0.759	0.734
EAD (£k)	85									1130
EAD Linear (£k)	85	201.11	317.22	433.33	549.44	665.56	781.67	897.78	1013.89	1130.00
PV (£k)	85	194	296	391	479	560	636	706	770	59
Year	10	11	12	13	14	15	16	17	18	19
Discount rate	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Annual Discount	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035
Product of discounts	0.709	0.685	0.662	0.639	0.618	0.597	0.577	0.557	0.538	0.520
EAD (£k)										1253
EAD Linear (£k)	1142.30	1154.6	1166.9	1179.2	1191.5	1203.8	1216.1	1228.4	1240.7	1253
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	20	21	22	23	24	25	26	27	28	29
Discount rate	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Annual Discount	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035	1.035
Product of discounts	0.503	0.486	0.469	0.453	0.438	0.423	0.409	0.395	0.382	0.369
EAD (£k)										1362
EAD Linear (£k)	1263.9	1274.8	1285.7	1296.6	1307.5	1318.4	1329.3	1340.2	1351.1	1362
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	30	31	32	33	34	35	36	37	38	39
Discount rate	3.5	3	3	3	3	3	3	3	3	3
Annual Discount	1.035	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Product of discounts	0.356	0.346	0.336	0.326	0.317	0.307	0.298	0.290	0.281	0.273
EAD (£k)										1462
EAD Linear (£k)	1372	1382	1392	1402	1412	1422	1432	1442	1452	1462
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	40	41	42	43	44	45	46	47	48	49
Discount rate	3	3	3	3	3	3	3	3	3	3
Annual Discount	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Product of discounts	0.265	0.257	0.250	0.243	0.236	0.229	0.222	0.216	0.209	0.203
EAD (£k)										1566
EAD Linear (£k)	1472.4	1482.8	1493.2	1503.6	1514	1524.4	1534.8	1545.2	1555.6	1566
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	50	51	52	53	54	55	56	57	58	59
Discount rate	3	3	3	3	3	3	3	3	3	3
Annual Discount	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Product of discounts	0.197	0.192	0.186	0.181	0.175	0.170	0.165	0.160	0.156	0.151
EAD (£k)										1675
EAD Linear (£k)	1576.9	1587.80	1598.70	1609.60	1620.50	1631.40	1642.30	1653.20	1664.10	1675.00
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	60	61	62	63	64	65	66	67	68	69
Discount rate	3	3	3	3	3	3	3	3	3	3
Annual Discount	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
Product of discounts	0.147	0.143	0.138	0.134	0.130	0.127	0.123	0.119	0.116	0.112
EAD (£k)										1790
EAD Linear (£k)	1686.50	1698	1709.5	1721	1732.5	1744	1755.5	1767	1778.5	1790
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	70	71	72	73	74	75	76	77	78	79
Discount rate	3	3	3	3	3	3	2.5	2.5	2.5	2.5
Annual Discount	1.03	1.03	1.03	1.03	1.03	1.03	1.025	1.025	1.025	1.025
Product of discounts	0.109	0.106	0.103	0.100	0.097	0.094	0.092	0.090	0.087	0.085
EAD (£k)										1907
EAD Linear (£k)	1801.7	1813.4	1825.1	1836.8	1848.5	1860.2	1871.9	1883.6	1895.3	1907
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	80	81	82	83	84	85	86	87	88	89
Discount rate	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Annual Discount	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025
Product of discounts	0.083	0.081	0.079	0.077	0.075	0.074	0.072	0.070	0.068	0.067
EAD (£k)										2027
EAD Linear (£k)	1919	1931	1943	1955	1967	1979	1991	2003	2015	2027
PV (£k)	0	0	0	0	0	0	0	0	0	0
Year	90	91	92	93	94	95	96	97	98	99
Discount rate	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Annual Discount	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025	1.025
Product of discounts	0.065	0.063	0.062	0.060	0.059	0.057	0.056	0.055	0.053	0.052
EAD (£k)										2149
EAD Linear (£k)	2039.2	2051.4	2063.6	2075.8	2088	2100.2	2112.4	2124.6	2136.8	2149
PV (£k)	0	0	0	0	0	0	0	0	0	0
Discount Sum over 50 years	24.495									
Discount Sum over 100 years	29.813									
Damages Sum over 50 years								4,176		
Damages Sum over 100 years								4,176		

Table 5.3 Present Value of flood losses for the ‘Do Nothing’ scenario over the next 100 years

6. REFERENCES

- Bates PD and De Roo APJ 2000 A simple raster-based model for flood inundation simulation. *Journal of Hydrology*, **236** 54-77.
- Besley P 1999 Overtopping of Seawalls Design and Assessment Manual. Environment Agency, R&D Technical Report W178.
- FHRC 2003 The benefits of flood and coastal defence: techniques and data for 2003. Penning-Rowsell E, Johnson C, Tunstall S, Tapsell S, Morris J, Chatterton J, Coker A, Green C. Flood Hazard Research Centre, Middlesex University.
- Goda Y 1985 Random seas and design of maritime structures. University of Tokyo Press.
- Herbert DM 1996 The overtopping of seawalls: A comparison between prototype and physical model data. HR Report No. TR22. HR Wallingford, Howbery Park, Wallingford, Oxon, OX10 8BA.
- Horritt MS and Bates PD 2001 Effects of spatial resolution on a raster based model of flood flow, *Journal of Hydrology* **253** 239-249.
- HR Wallingford 2002 Overstrand to Walcott Strategy Study: Hydrodynamics, Part II: Technical support information. HR Report No. EX 4692. HR Wallingford, Howbery Park, Wallingford, Oxon, OX10 8BA.
- Jensen OJ and Sorensen T 1979 Overspilling/overtopping of rubble-mound breakwaters. Results of studies, useful design procedures. *Coastal Engng.*, **3**. pp 51-65.
- MAFF 1999 Flood and coastal defence project appraisal guidance: FCDPAG3: Economic appraisal. MAFF Publication PB 4650.
- Simm JD 1991 Manual on the use of rock in coastal and shoreline engineering. CIRIA special publication 83, CUR Report 154.