Annex 32.3

Breach Design Report

(Black & Veatch)

Cherry Cobb Sands Compensation Site

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FIGURES

Figures 5, 9 and 19 have been produced using Ordnance Survey data.

1 INTRODUCTION

1.1 SITE DESCRIPTION

1.1.1 The Able Marine Energy Park (AMEP) on the south bank of the Humber is expected to require a compensatory managed realignment on the north bank of the Humber at Cherry Cobb Sands. The size of the required compensation is unknown but expected to range between 70 and 110 ha. Initial model tests examined a range of breaching scenarios for the 90 ha site shown in Figure 1, with subsequent tests considering a further range of breaching scenarios for the site of 110ha shown in Figure 2. The locations considered for the breaches are shown on Figure 3.

1.1.2 These preliminary tests were designed to identify the most favourable location, length and invert level of breaches required in the flood defences to inundate the proposed Compensation Site. The ground level within the Compensation Site is fairly uniform and close to 2.5 mAOD.

1.1.3 Subsequent model studies using a revised model (Annex 32.2) were used to test the 90 and 110 ha Compensation Sites that were later considered in greater detail. These model tests are described in Annexes 32.4 and 32.6.

1.1.4 The part of the Compensation Site identified in the site selection reports (Annex 30.1 Figure 8 and Annex 30.2 Figure 2) that has been chosen for the managed realignment is the southern part of the site. This is because the drainage creek running parallel to the flood defences increases in size to the south. In addition, at the southern end, the width of saltmarsh between the defence and the creek is fairly constant at around 80 m. In the northern part of the Compensation Site the level of the foreshore between the defence and the creek rises and the creek gradually becomes further offshore as a result of a minor change in the alignment of the flood defence.

1.1.5 The main difference between the 90 ha site modelled initially and the 110 ha site modelled subsequently (Figure 1 and Figure 2) is in the distance the site extends to the north. The southern parts of both sites are identical. For these modelling studies no large scale modification of ground level was considered. For some options with the 110 ha site, a raised track across the site was removed and in some options a section of the ground was lowered by up to 0.5 m to see if that reduced velocities within the site. If widespread modification of ground levels is required, further model tests will be required.

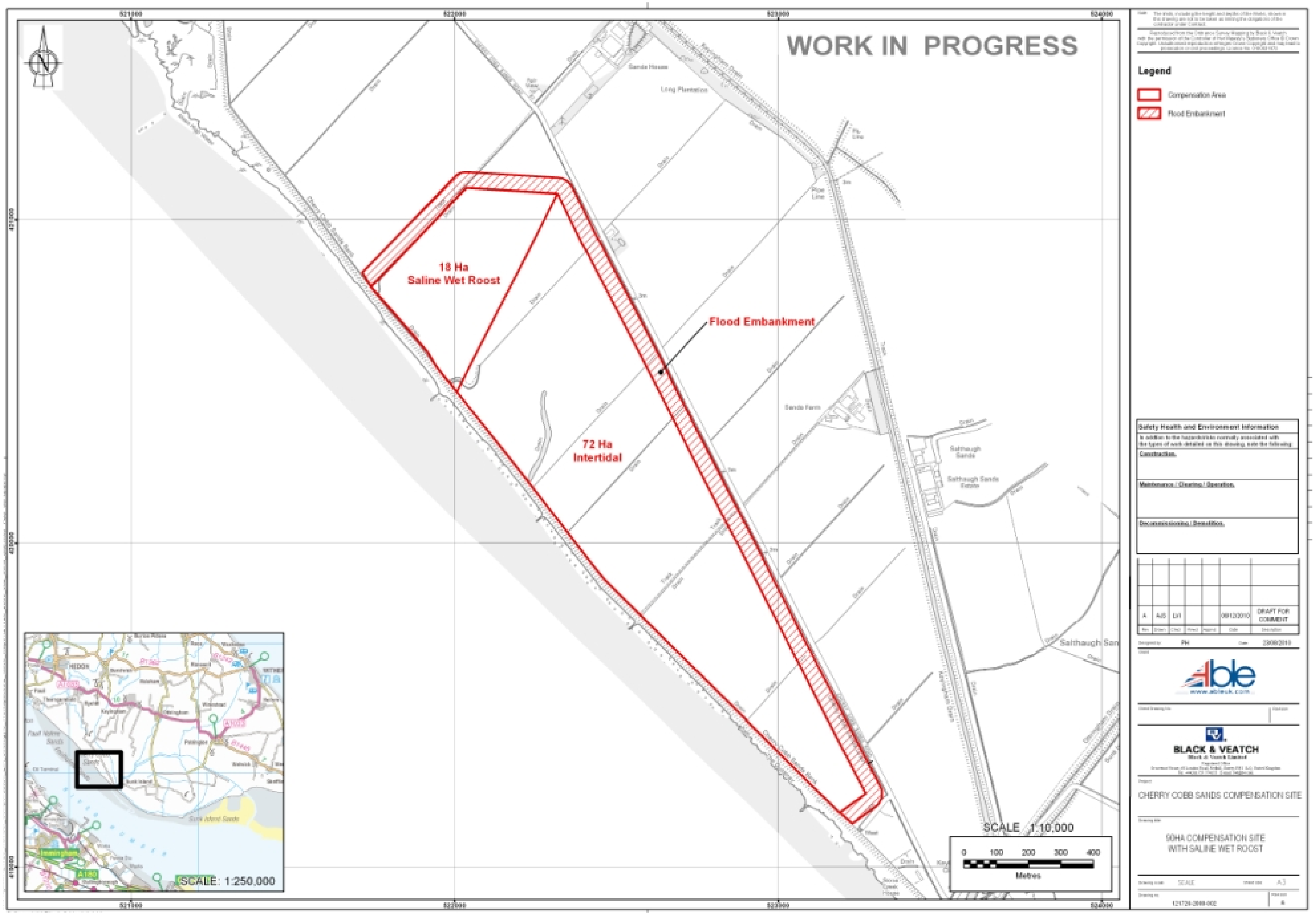


Figure 1 Original 90 ha Compensation Site arrangement

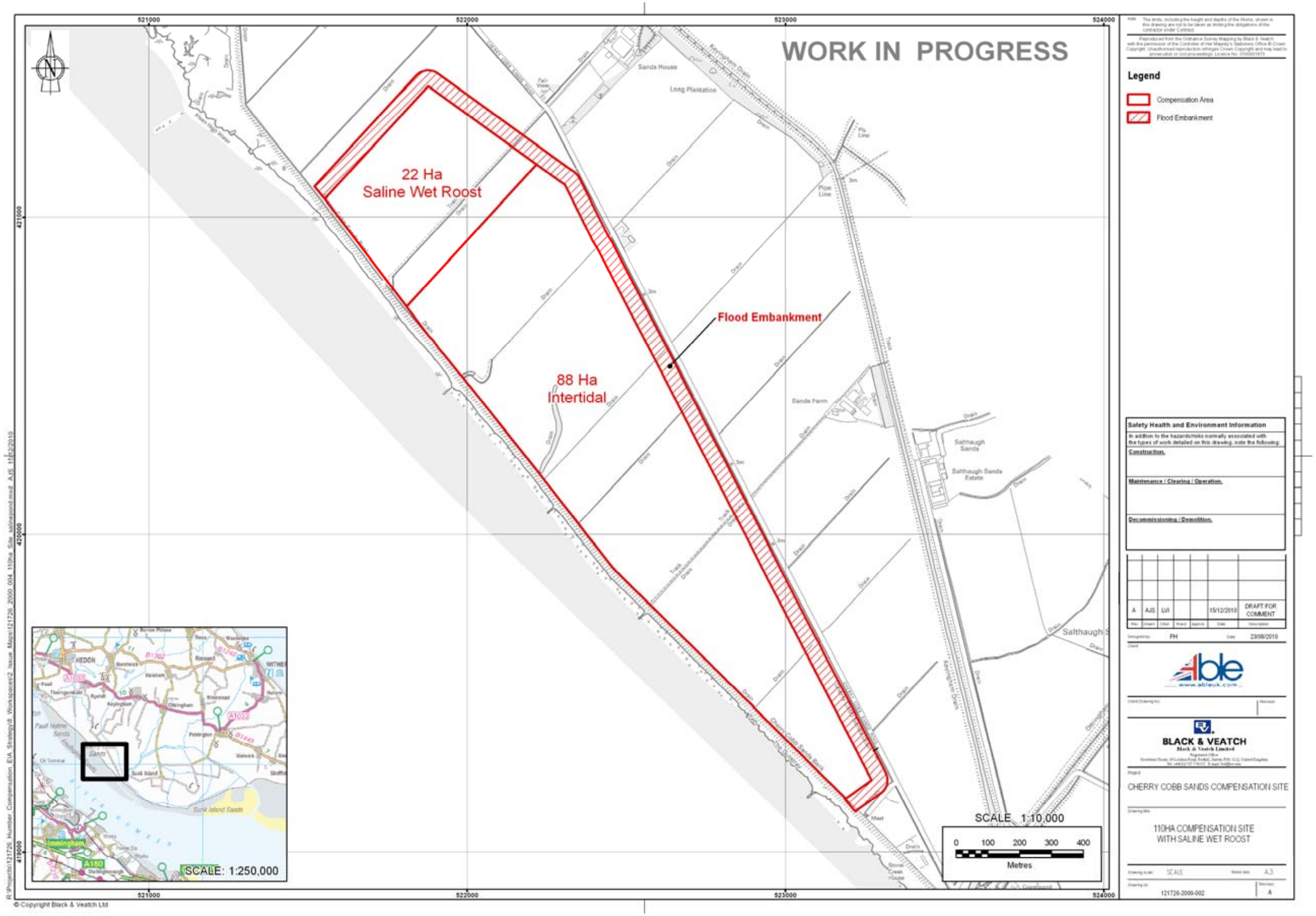


Figure 2 Original 110 ha Compensation Site Arrangement

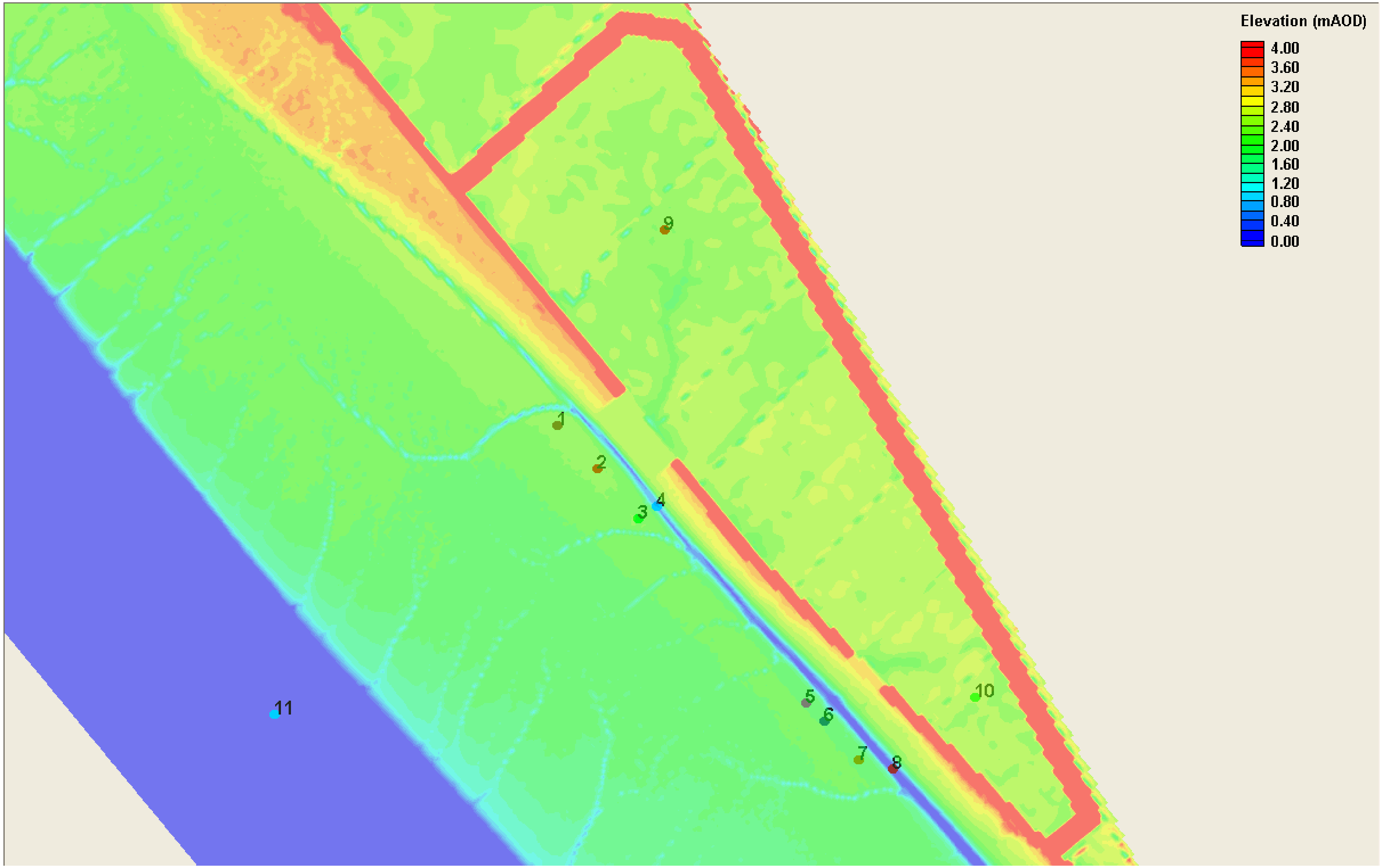


Figure 3 Breach locations and output points 1 to 11

1.2 *FORESHORE TOPOGRAPHY AND MODEL LIMITS*

1.2.1 The outflow from the Cherry Cobb Sands Compensation Site is expected to flow into a large creek named for this study as "Cherry Cobb Sands Creek" that runs parallel to and about 80 m seaward of the existing flood defences. The topography of the creek and foreshore is shown on Figure 4. Cherry Cobb Sands Creek drains to the south and picks up drainage flowing through the four sluices at Stone Creek and continues parallel to the coast for a further 2 to 2.5 km before turning seaward and entering the low water channel of the Humber. The southernmost part of the creek is believed to be fairly dynamic. Foul Holme Sands separates Cherry Cobb Sands Creek from the main Humber low water channel.

1.2.2 The boundaries of the model are shown on Figure 4. The set up of the hydraulic model and the derivation of its boundary conditions are described in Annex 32.2. As discussed in Annex 32.2, no calibration or verification of the hydraulic model was carried out for the phase of the study described in this Annex. This was because the more refined whole Humber estuary model described in Annex 8.1 was under development and would be used to drive an updated version of the Cherry Cobb Sands hydraulic model. This updated model was used for the tests of the 90 and 110 ha Compensation Sites that are reported in Annexes 32.4 and 32.6.

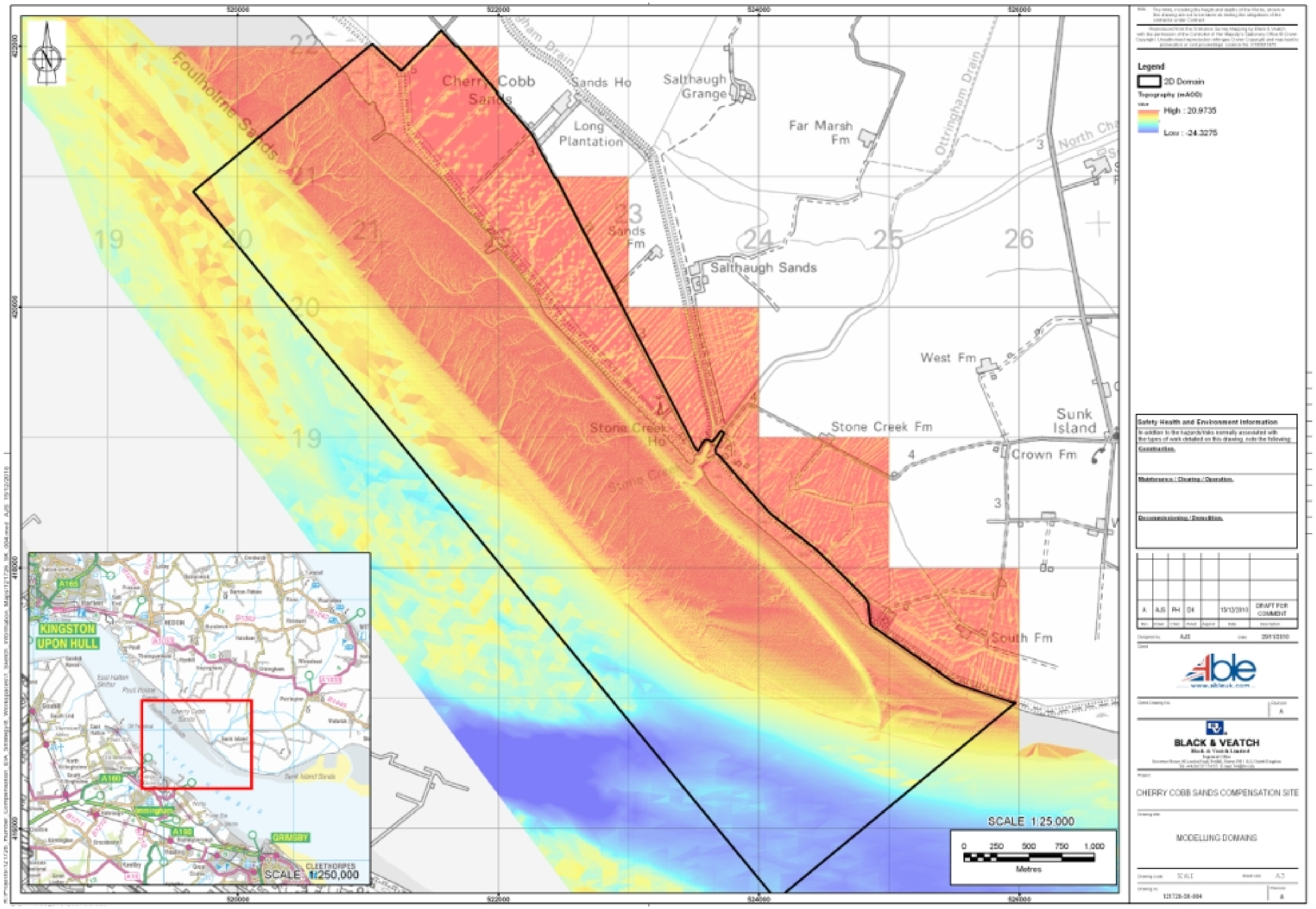


Figure 4 Foreshore topography and model boundaries near Cherry Cobb Sands

1.3 *TIDAL BOUNDARY CONDITIONS*

1.3.1 The original hydraulic model of the Humber estuary developed by JBA was tested against a tide with a high water level of 3.1 mAOD at Immingham (JBA 2011). High tide levels at the Compensation Site are similar to those at Immingham. A high tide level of 3.1 mAOD is a frequently occurring tide, but does not provide a severe test of the performance of the breach as the realignment site is only inundated to a depth of 0.6m. In order to provide a more severe test, the boundary conditions supplied were 'stretched' by multiplying all levels and flows by 1.25. This produced a tide with a high water level at Immingham of 3.85 mAOD. This stretched tide is suitable to compare options though it does not fully represent tidal conditions on a large tide. Observed tidal conditions are considered in Annexes 32.4 and 32.6

1.3.2 Inspection of the tide tables for Immingham (1996 and 2008-2011) showed that tides of 3.8 mAOD and higher are experienced several times each year. Their frequency of occurrence varies between 0.4% (2008) and 2.6% (2010). The highest predicted tides at Immingham have a high water level of 4.1 mAOD, but these do not occur every year. Thus the tide adopted for the design is typical of the largest tides that would be experienced each year at the compensation site, and might be experienced on average 1-2% of the time. As high water levels reduce, the frequency of occurrence increases as indicated on Table 1.

1.3.3 The variability in the frequency of occurrence is a natural phenomenon as tide heights vary from year to year. The maximum frequency of tides above 2.5 and 3.4 mAOD in these five years occurred in 1996 and the minimum in 2008. This may reflect the effect of the lunar nodal tide that varies the main lunar component on an 18.6 year cycle. This cycle was at its peak in 1996 and a minimum in 2006. The occurrence of very high tides exceeding 3.8 mAOD follows a more complex pattern with the greatest frequency occurring in 2011 and the minimum in 2008.

Table 1 Frequency of occurrence of high tides at Immingham 1996, 2008-2011

Level mAOD	1996	2008	2009	2010	2011	Average (5 years)	Average (3 years) 1996, 2008, 2010
Percent >2.5	64.0	55.5	56.7	58.6	59.6	58.9	59.4
Percent ≥ 3.0	41.2	32.3		34.8			36.1
Percent ≥ 3.4	15.4	9.6	10.8	11.2	11.9	11.8	12.1
Percent ≥ 3.8	2.5	0.4	1.3	2.6	2.8	1.9	1.8
Percent ≥ 4.0	0.3	0.0		0.7			0.3

2 BREACH ARRANGEMENTS

2.1 LOCATIONS OF BREACHES

2.1.1 Breaches were tested primarily at both a northern site and a southern site as indicated on Figure 3, or at a combination of the two. Sites further south were not considered because of the small width of the realignment at its south end. Breaches further north than the northern breach site were not considered because of the increasing level of the foreshore between the creek and the flood defence and the increasing distance between these two features.

2.1.2 Breach lengths of 150 m to 300 m in length for the main breach, sometimes with a 100 m subsidiary breach, were tested. The invert level of these breaches varied between 3.0 mAOD and 2.0 mAOD. The average ground level in the site is 2.5 mAOD, while the site adjacent to the northern breach location was close to 2.0 mAOD. For all breaches, a swath of foreshore was cut out for the full width of the breach down to its invert level between the foreshore and the creek. This was done to allow good access through the saltmarsh between the estuary and the Compensation Site. The breaches that were tested are set out in Table 2. The arrangement with the lowered ground levels for Option H3 is shown in Figure 5.

Table 2 Breach lengths and locations used in model tests

Test	Northern breach		Southern breach		Comments
Area	Length m	Level mAOD	Length m	Level mAOD	Conditions inside site
AA-90	130	2.0	-	-	Existing
A-90	200	2.0	-	-	Existing
B-90	200	2.5	-	-	Existing
C-90	200	2.5	100	3.0	Existing
D-90	200	2.0	100	2.5	Existing
E-90	-	-	200	2.0	Existing
F1-110	-	-	200	2.0	Existing; same breach as E
F2-110	-	-	200	2.0	Track removed, foreshore channel at 45° to flood defence
F3-110*	-	-	200	2.0	Track removed with an area of 2 mAOD inside site
G-110*	100	2.5	200	2.0	Ground level in site as F3
H1-110*	-	-	250	2.0	Ground level in site as F3
I-110*	-	-	159	2.0	Ground level in site as F3
J-110*	100	2.5	150	2.0	Ground level in site as F3
K1-110*	-	-	300	2.0	Ground level in site as F3
K2-110*	-	-	300	2.0	Existing
H2-110*	-	-	250	2.0	Existing
H3-110*	-	-	250	2.0	Low ground extended north.
L-110*	-	-	250	2.0	As H2; breach moved 250m north

Note *Model tests F3-110 onwards included revised boundary conditions.

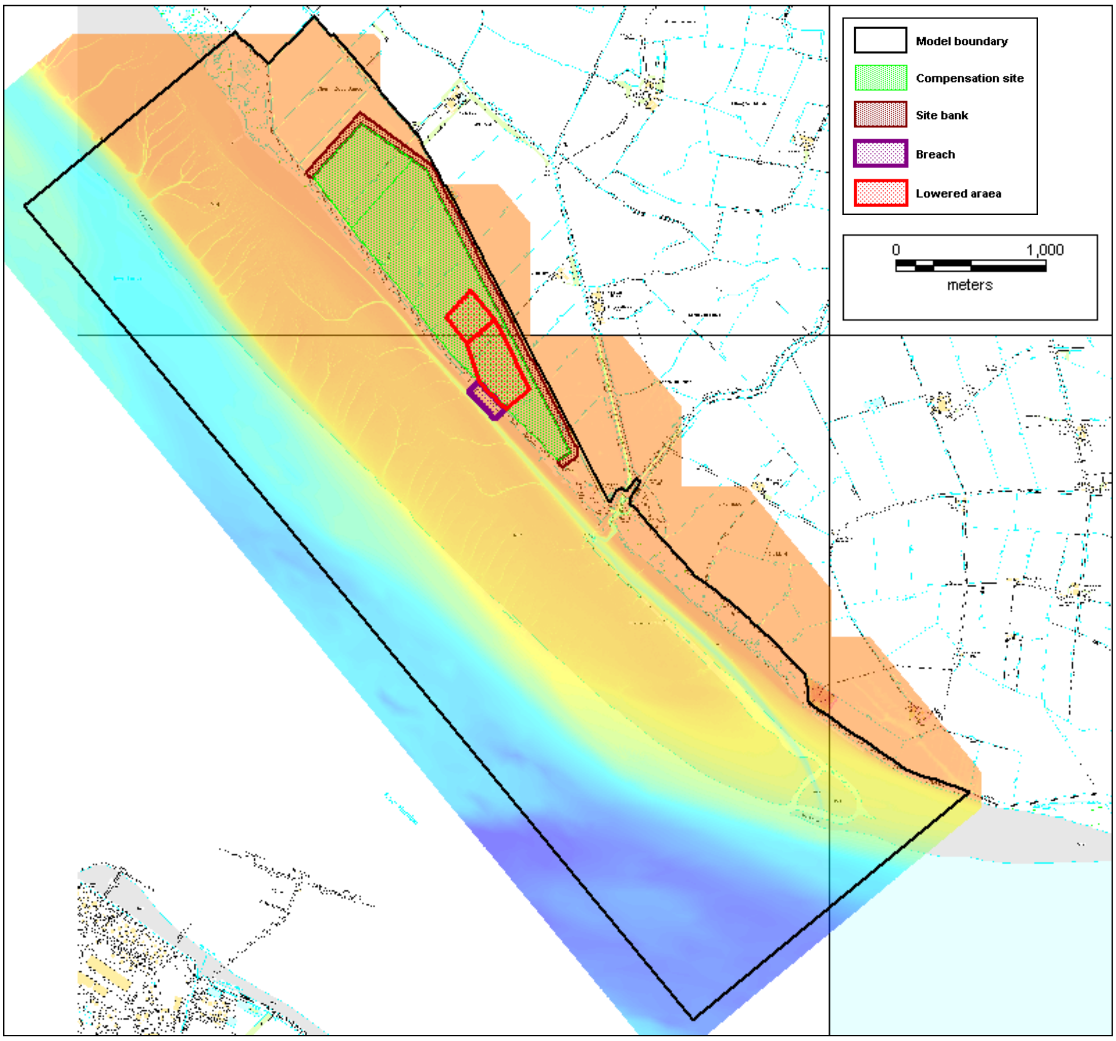


Figure 5 Layout of Option H3

2.2 ISSUES FOR CHOICE OF BREACHES

Loss of existing saltmarsh

2.2.1 The choice of breach arrangement is a compromise between various competing factors. One simple aspect is that as the length of breach increases, the area of saltmarsh in front of the breach that is destroyed to provide access to the realignment site will also increase. As the saltmarsh is around 80m wide in front of the existing Cherry Cobb defences, each 120 m of breach will lead to 9600 m^2 loss of saltmarsh. This is approximately 1 ha loss.

Enlargement of Cherry Cobb Sands Creek

2.2.2 The creek that runs in front of the flood defences is likely to take a proportion of the drainage of the managed realignment site. For instance, all drainage from the site will pass down this creek once the tide level in the estuary drops to expose the top of Foul Holme sand. This additional flow will lead to an enlargement of the creek.

Velocities over Foul Holme Sand

2.2.3 The presence of the Compensation Site at Cherry Cobb Sands will increase flows locally as the site is filled and emptied. Some of this flow will pass down the creek and lead to its enlargement. The remainder will enter or leave the site across Foul Holme Sand. These flows are limited to the time period when the sand bank is fully covered with water.

2.2.4 If velocities over Foul Holme Sand increase, there is the risk of erosion of the top of the sand bank. Initially scour of the top of the sandbank is likely to be slow, but if it starts to occur and a low way forms across the sand bank, the local increase in water depth will reduce the resistance of this flow path which will increase the flow and velocity in the low way and cause more rapid scour.

2.2.5 How a creek across Foul Holme Sand might develop is very difficult to predict and will depend on the balance between processes that maintain the existing sandbank in its present alignment and shape and the power of the drainage water from the managed realignment site to increase the size of any low way. The historic evolution of the Foul Holme foreshore including the presence across this foreshore of a creek is discussed in Annex 32.1, the report on foreshore evolution.

Velocities within the Compensation Site

2.2.6 Another aspect that was identified from early tests of the southern breach was the presence of high velocities for a short period within the Compensation Site just to the north of this breach as flows moved north to inundate the remainder of the site. The area of higher velocity ran alongside the realigned embankment a short distance north of the southern breach, if this was the longest or lowest breach. This might necessitate erosion protection to the toe of the embankment.

3 MODEL RESULTS

3.1.1 Review of model results was a mixture of visual comparison of the flow field associated with different arrangements and review of the time history of velocities near critical points in the foreshore and within the site. The locations where velocities and levels are calculated are shown on Figure 3 and Figure 6.

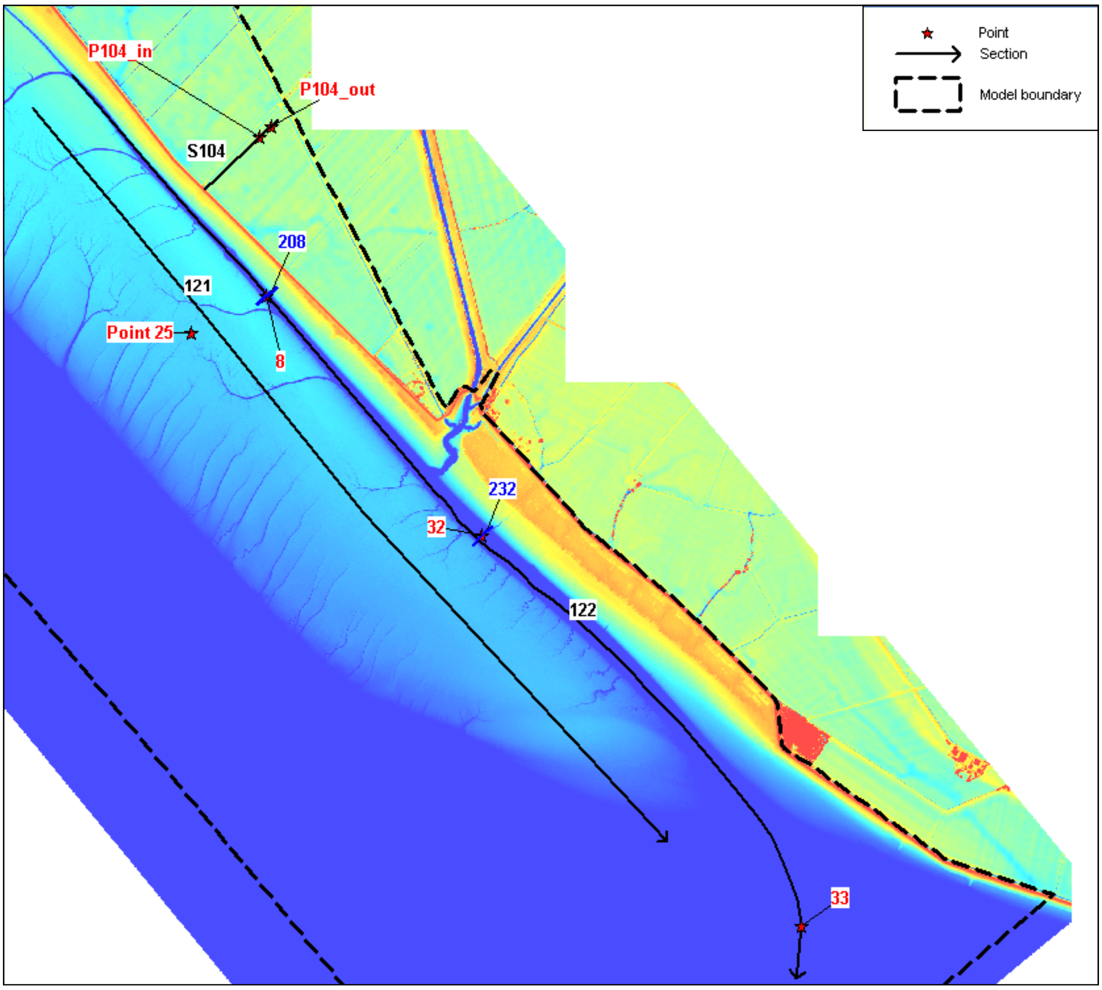


Figure 6 Model output Points and long sections

3.2 MODEL TIDE LEVELS

3.2.1 The model tests considered a high spring tide. High and low tide levels on this tide are listed in Table 3. Point 11 shows water levels within the Humber. Points 8, 32 and 33 report levels in Cherry Cobb Sands Creek, upstream and downstream of the Stone Creek drainage outfall and close to the Humber confluence respectively, while Points 9 and 104 are in the northern and southern parts of the Compensation Site respectively.

3.2.2 The tide levels with and without the scheme are very similar in the estuary as Table 3 indicates. High tide levels with option H3 shown on Figure 5, which was considered one of the preferred options at the conclusion of this phase of study, are reduced by 0.01or 0.02 m in the Humber and near Stone Creek, but with no change near the confluence of Cherry Cobb Sands Creek with the Humber. Low tide levels are not changed in the Humber, but are raised close to the breach in Cherry Cobb Sands Creek (Point 8) by 0.22 m and near the confluence with the Humber (Point 33) by 0.15 m, in both cases due to the drainage from the Compensation Site. At Point 32 there is no change in the low water level suggesting that there is a different control on low tide levels at this site. The small changes identified of up to 0.02 m may well be associated with modelling effects and may not represent changes that would occur in practice. The changes in excess of 0.1 m are likely to represent changes that would occur in practice.

3.2.3 The predicted tidal levels at each site are shown in Figure 7 for the model with the Compensation Site in operation. These water level profiles are indistinguishable from those in the baseline case except for Points 8 and 33 where low water levels are lower as indicated in Table 3.

Table 3 Model tide levels

	High water levels mAOD		**Low water levels mAOD**	
	Baseline	**Option H3**	**Baseline**	**Option H3**
Humber Estuary Pt 11	3.92	3.91	-3.08	-3.08
Cherry Cobb Sands Creek north of Stone Creek Pt 8	3.93	3.91	-0.07	0.15
Cherry Cobb Sands Creek south of Stone Creek Pt 32	3.80	3.79	-0.59	-0.59
Cherry Cobb Sands Creek near Humber Pt 33	3.65	3.65	-1.97	-1.82
Inside Site Pt 104_in	-	3.87	-	2.07
North end of site Pt 9	-	3.96	-	2.52

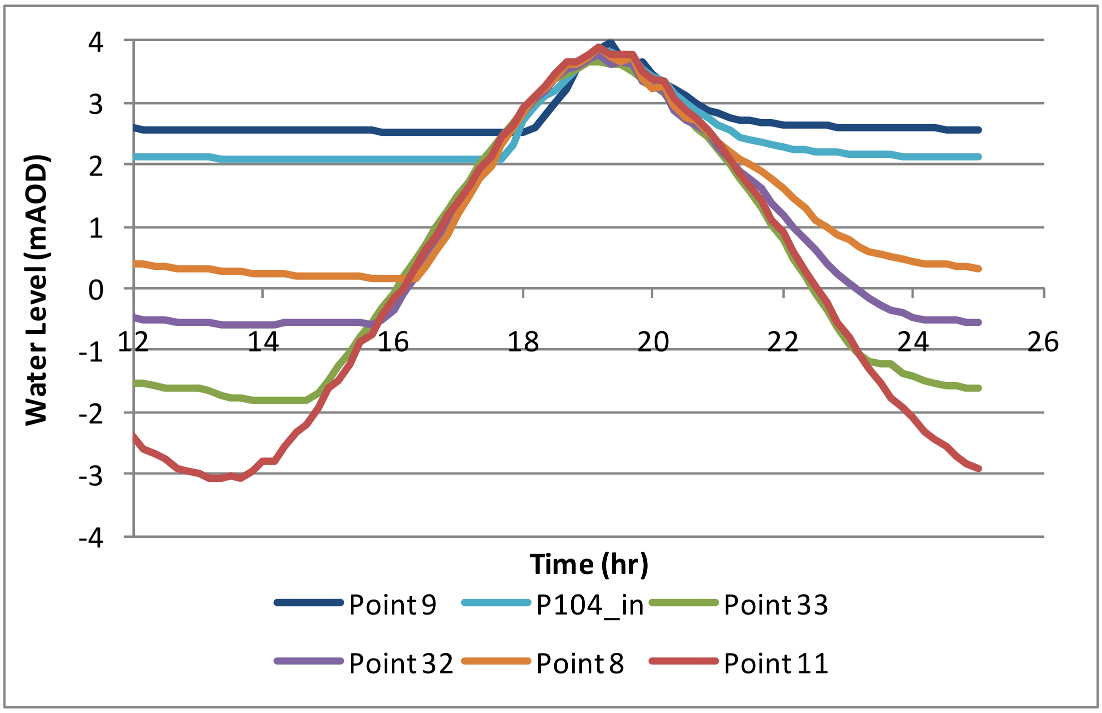


Figure 7 Model tide levels

3.3 *SALTMARSH LOSS AND BREACH VELOCITY*

3.3.1 Assessment of the potential loss of saltmarsh was considered proportional to the length of breach, whether at the north of the south. On this measure, for the tests of Table 2, Option AA with one 130 m breach caused the least loss of saltmarsh estimated at 1.04 ha while Options C, D, G and K with a total breach length of 300 m might cause the greatest saltmarsh loss estimated at 2.4 ha. Option H with a 250 m breach is estimated to cause a loss of 2.0 ha of saltmarsh.

3.3.2 Comparison of average peak maximum and peak maximum velocity in the southern breach as a function of breach length is illustrated in Figure 8. This shows a reduction in peak velocity as the breach length increases from 200 m (Option F3) to 250 m (Option H1), but little further reduction as the length increases to 300 m (Option K1). This led to the conclusion that for a single breach 250 m is probably the optimum length to minimise loss of saltmarsh, without causing high velocities in the breach that might lead to further scouring.

3.3.3 The areas of high velocity in and near to the southern breach for Option H3 are shown in Figure 9. This shows high velocity especially near the northern end of the breach and where the water draining from the Compensation Site enters Cherry Cobb Sands Creek. These latter areas of high velocity will cause local erosion which will form a creek connecting Cherry Cobb Sands Creek to the breach site.

3.3.4 A variety of options involving two breaches have been tested as illustrated in Table 2. The combined length of the two breaches was either 250 m (Option J) or 300 m (Options C, D and G), so offer no reduction in breach length compared to the single breach options.

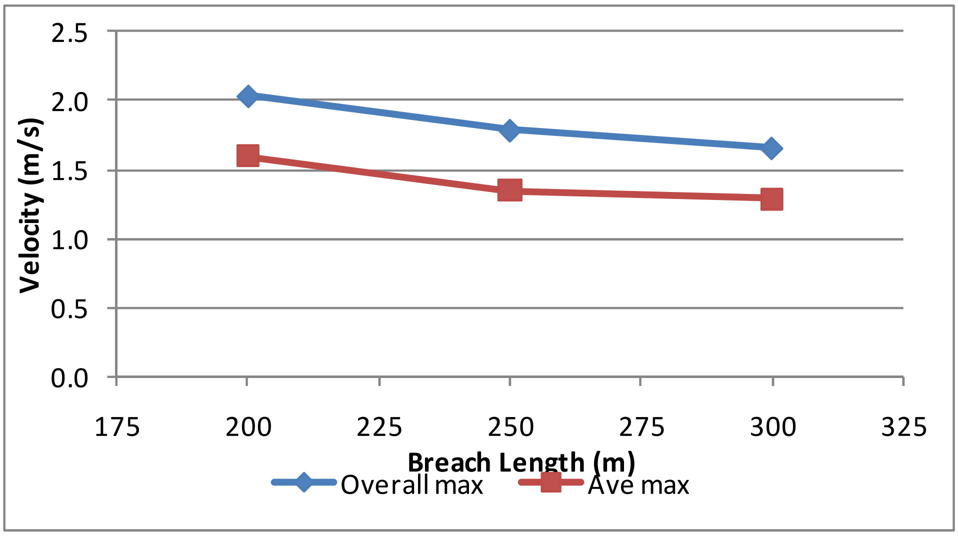


Figure 8 Effect of breach length on maximum breach velocity

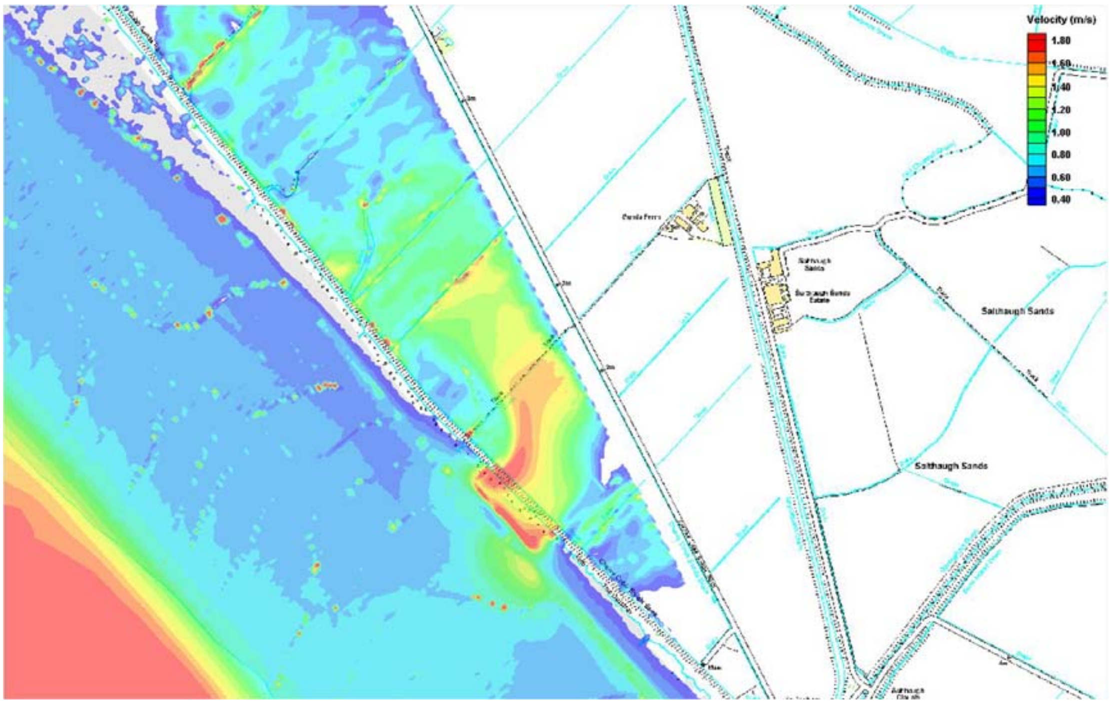


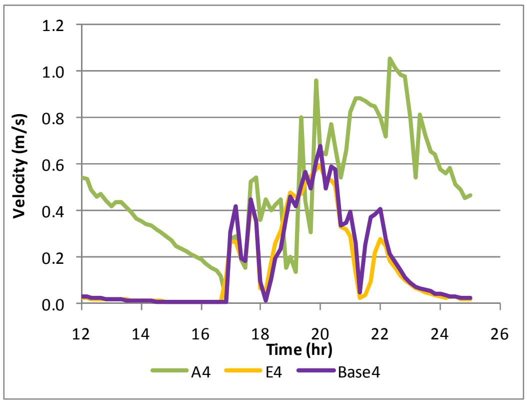
Figure 9   Maximum velocities within the Compensation Site (Option H3)

3.4   ENLARGEMENT OF CHERRY COBB SANDS CREEK

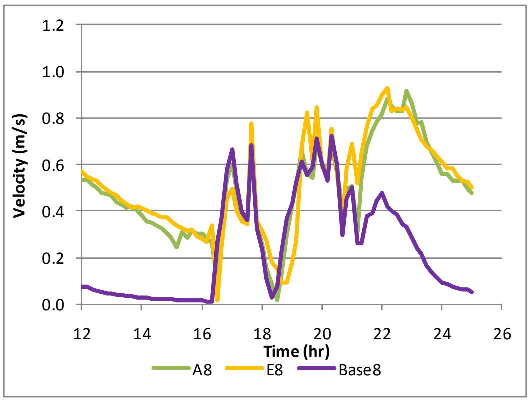
3.4.1 Assessment of flows in the creek was made by considering Point 4 (see Figure 3) a short distance downstream of the northern breach, Point 8 (see Figure 3 and Figure 6) a short distance downstream of the southern breach and Point 32 (see Figure 6) downstream of the Stone Creek drainage outfalls. Figure 10a shows that flows at Point 4 in the creek downstream of the northern breach are only affected by a northern breach (Option A). As expected, a southern breach (Option E) has little effect in the creek at Point 4 which is north and so upstream of the breach. The effects of the drainage flows from the Compensation Site are particularly evident between hours 21 and 25 on Figure 10 while the tide is ebbing. The Compensation Site only has a small effect on flows during the flood tide (hours 17 to 19).

3.4.2 Flows in Cherry Cobb Sands Creek at Point 8 downstream of the southern breach are shown in Figure 10b. These velocities are similar for schemes with a northern breach (Option A) or a southern breach (Option E). The flows at Point 8 are, however, sensitive to the size of the Compensation Site as indicated on Figure 11a, which compares flows with a 90 ha site (Option E) and for a 110 ha site (Option F1); in both cases with a 200 m southern breach. The 22% larger 110 ha site leads to an 11% increase in the average velocity at Point 8 and an increase in maximum velocity from 0.94 m/s to 1.06 m/s as shown on Figure 11a.

3.4.3 The effects on velocity in Cherry Cobb Sands Creek at Point 8 of changes in breach length between 150 m and 250 m are shown on Figure 11b for the 110 ha Compensation Site. These changes in breach length do not affect average velocity, though it is noteworthy that the maximum velocity is 9% higher with the shortest breach.

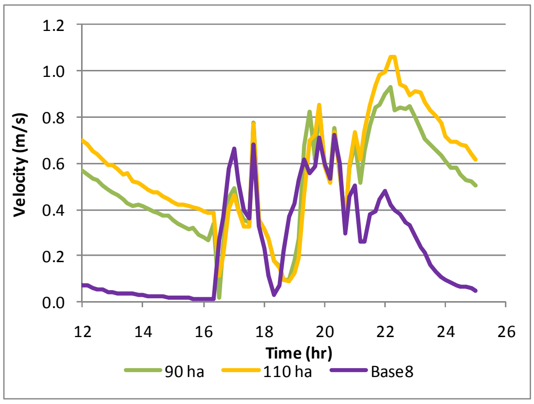


10a) Point 4 (north breach)

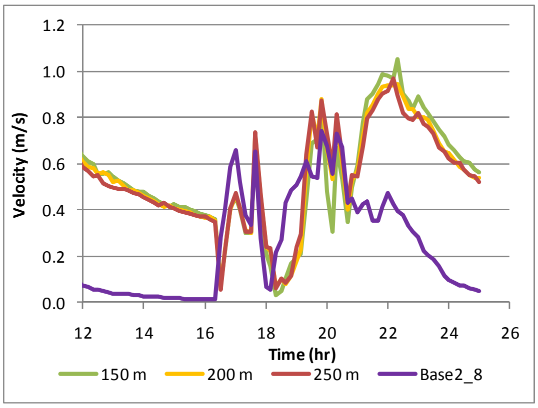


10b) Point 8 (south breach)

Figure 10 Comparison of velocities in Cherry Cobb Sands Creek north of Stone Creek



11a) Effect of site size on velocity



11b) Effect of breach length on velocity

Figure 11 Effect of site size and breach length on flows in Cherry Cobb Sands Creek

3.4.4 Profile 122 along Cherry Cobb Sands Creek (see Figure 6) is used to show the maximum velocity in this creek on Figure 12 for the existing situation and with Option H3 (Table 2), a 110 ha Compensation Site including a 250 m long southern breach. The breach is 1000 m upstream of Stone Creek and its location is shown on Figure 12a. The level of the bed of Cherry Cobb Sands Creek as reported by the 2007 LiDAR is shown on Figure 12 to fall along the length of the creek. Upstream of Stone Creek it rises from -1 mAOD to +1 mAOD, while downstream it falls from -1 mAOD to -2.5 mAOD where it joins the main low water area of the Humber.

3.4.5 The effect of the breach, shown on Figure 12a is to increase average maximum velocities in Cherry Cobb Sands Creek between the breach and Stone Creek from 0.67 m/s to 0.97 m/s, a 44 % increase. There is little change to maximum velocities in Cherry Cobb Sands Creek upstream of the breach location. Downstream of Stone Creek, the Compensation Site increases average maximum velocities 9 % from 0.95 m/s to 1.04 m/s as Figure 12b shows. For about 1200 m downstream of Stone Creek there is negligible change in maximum velocity, but further downstream the Compensation Site does cause some increase in velocity as the creek approaches the low water in the Humber.

3.4.6 The velocity and tide level variation during a high spring tide along Cherry Cobb Sands Creek is illustrated for Point 8 upstream of Stone Creek and Point 32 downstream for Option H3 in Figure 13. The water level variation shows that at Point 8, low water levels are close to 0.1 mAOD, while 50 m upstream of Point 32, low water level is around -0.9 mAOD, a 1m fall in low water level over approximately 1200 m in an area where bed levels drop 1.5 m (Figure 12).

3.4.7 The velocities in the base case drop to zero during the low tide period, indicating that there is probably very little flow at low tide in Cherry Cobb Sands Creek. The undulating bed level of the creek is likely to lead to retention of water at a level 0.1 – 0.2 m lower than those shown in Figure 13

3.4.8 During the flood tide, velocities in Cherry Cobb Sands Creek are generally little affected by the presence of the Compensation Site especially downstream of Stone Creek at Point 32 (Figure 13b). On the ebb tide, the initial period of high velocity is also relatively little affected by the presence of the Compensation Site especially at Point 32. At Point 8 there is a small increase in the peak velocity on both the flood and early ebb tides (Figure 13b).

3.4.9 The major effect of the Compensation Site at both Point 8 and Point 32 is found during the latter part of the ebb tide when there is a period of high velocity while the water held on the site drains out down Cherry Cobb Sands Creek. This occurs once water levels have dropped below +2 mAOD so there is no outflow across Foul Holme Sand at the site of the breach just to the north of Point 8. The effect is greatest at Point 8 and reduces in the larger and deeper section of creek downstream of the Stone Creek outfalls at Point 32. The profile of maximum velocity across the creek at these two points is shown on Figure 14. These profiles are taken looking south, so Foul Holme Sand is on the right and the existing flood defence embankment is beyond the left hand side of each plot.

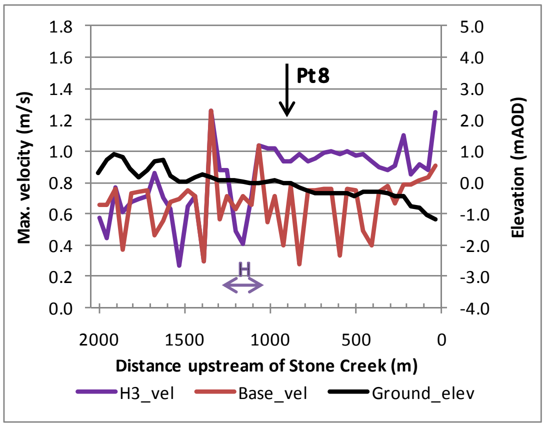
3.4.10 A velocity and water level profile at Point 33 (see Figure 6) on Figure 15 shows that close to the confluence of Cherry Cobb Sands Creek with the Humber low water area, velocities are higher than further upstream in the creek but the changes associated with the Compensation Site are relatively small. Nevertheless, these high velocities could be expected to transport large quantities of sediment highlighting the dynamic nature of the downstream portion of the creek and the likelihood that this will continue with the introduction of the Compensation Site.

3.4.11 The conclusions from these tests are that:

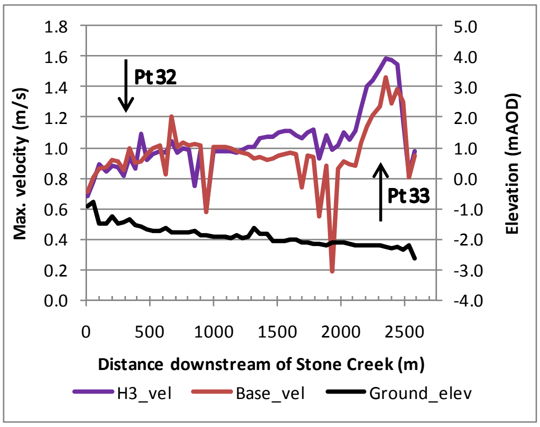
- The Compensation Site will considerably increase velocities and the duration of high velocities in Cherry Cobb Sands Creek upstream of the outfall with Stone Creek. This is likely to lead to increased erosion in this part of the creek. The extent of this area of increased erosion is likely to extend from the breach to the Stone Creek confluence so that the closer the breach is to Stone Creek, the shorter the length of creek that is likely to experience erosion.

- For about 1 km downstream of the Stone Creek confluence, Cherry Cobb Sands Creek is not predicted to experience an increase in maximum velocity, but the duration of high velocities is predicted to increase. This is likely to lead to some erosion of this part of the creek.

- The changes in velocity at the downstream end of Cherry Cobb Sands Creek are not large, but as velocities in the shallow flows that occur at low tide are quite high, this part of the creek is likely to remain dynamic.

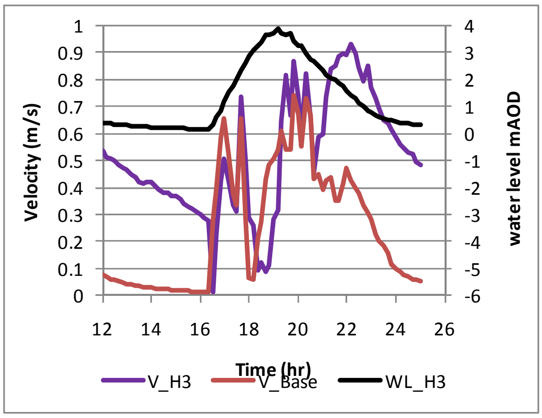


12a) Profile 122 upstream of Stone Creek

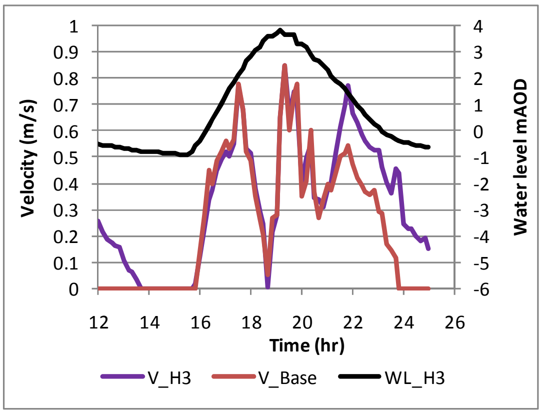


12b) Profile 122 downstream of Stone creek

Figure 12 Maximum velocity profiles along Cherry Cobb Sands Creek

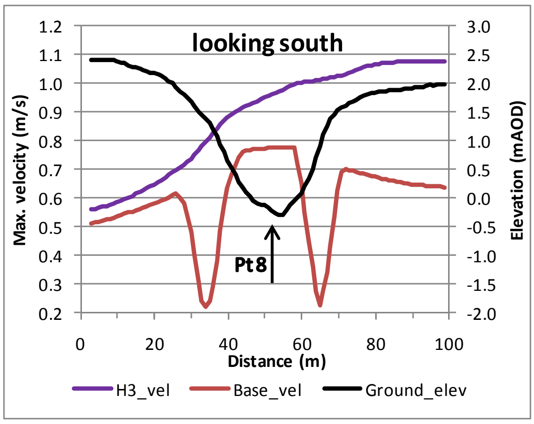


13a) Water level and velocity at Point 8

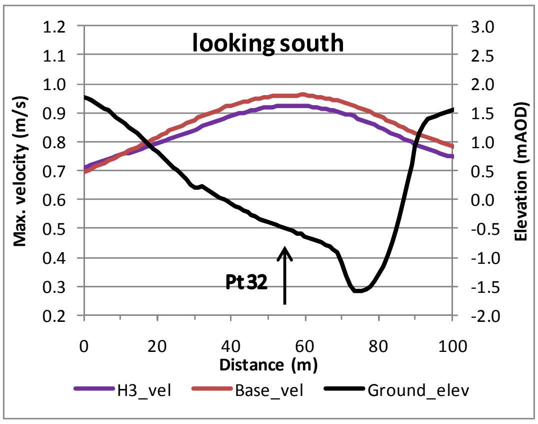


13b) Water level and velocity at Point 32

Figure 13 Tidal velocity variations in Cherry Cobb Sands Creek



14a) Maximum velocities at Point 8



14b) Maximum velocities at Point 32

Figure 14 Maximum velocities at cross sections in Cherry Cobb Sands Creek

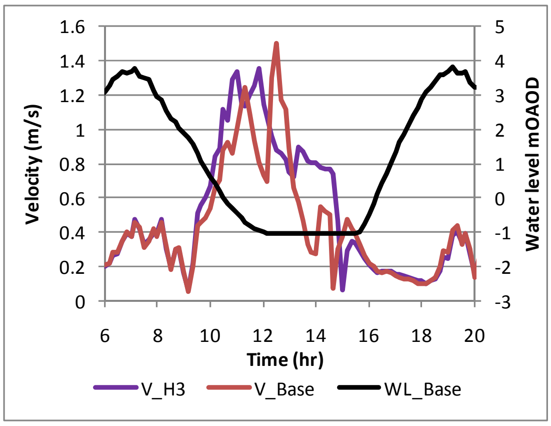


Figure 15 Velocity and level profile at Point 33

3.5 *VELOCITIES OVER FOUL HOLME SAND*

3.5.1 Around high tide, Foul Holme Sand is covered by tidal waters, so some of the water flooding onto and draining out of the Compensation Site could flow across this bank increasing the risk of erosion of the top of the bank. Consideration of the flows across Foul Holme Sand has played an important role in determining a reasonable location and level for breaches that allow tidal water to inundate the Compensation Site. Velocities across Foul Holme Sand are illustrated in Figure 9 for Option H3.

3.5.2 Model velocities have been recorded at a number of Points on Foul Holme Sand and supplemented by a long section showing the distribution of maximum velocity over the sandbank. Points 1, 2 and 3 (see Figure 3) have been used to consider changes to predicted velocities on Foul Holme Sand near the northern breach, while Points 5, 6, 7 (see Figure 3) and 25 (see Figure 6) have been used to consider predicted velocities near the southern breach location. Long section 121 (see Figure 6) plots maximum velocity along the crest of Foul Holme Sand parallel to Cherry Cobb Sands Creek to show effects of the different breach locations on velocities across this bank.

3.5.3 A comparison of the maximum velocity profile along the top of Foul Holme Sand in Figure 16 compares maximum velocities for Option A with a northern breach and Option E for the same 90 ha site with a similar breach but in a different location. The comparison shows that both breach locations cause an increase in maximum velocity over Foul Holme Sand. A maximum velocity of 0.91 m/s is predicted for Option A with the northern breach but only 0.73 m/s for Option E with the southern breach. These compare with a predicted maximum velocity of 0.60 - 0.65 m/s in this section in the baseline case.

3.5.4 If there is a southern breach as in Option E, there is a small reduction in maximum velocity in the vicinity of the northern breach. This is probably a result of the restriction to flow along the northern part of the crest of Foul Holme Sand caused by the outflow from the Compensation Site.

3.5.5 A further test of the effect of the position of the breach on velocities over Foul Holme Sand was carried out by comparing velocities for the 250 m breach of Option H3 with the similar length breach 250 m further north with Option L. The results presented in Figure 17 show that even this change in breach location leads to an increase in predicted maximum velocity.

3.5.6 The velocity profile at Point 25 on the top of Foul Holme Sand in Figure 18 shows that the velocity is little affected by the presence of the breach for most of the time that this site is flooded. However, there is an increase in velocity of around 0.1 m/s for about 1.5 hrs on the ebb tide which raises the maximum velocity to about 0.8 m/s. Point 25 is closer to Cherry Cobb Sands Creek than Section 121 and the velocities at this point are somewhat higher than on the section shown in Figure 17.

3.5.7 The conclusion has been drawn, therefore, that to minimise the increase in maximum velocity and hence the potential for erosion of Foul Holme Sand the breach for the Compensation Site should be located as far south as possible.

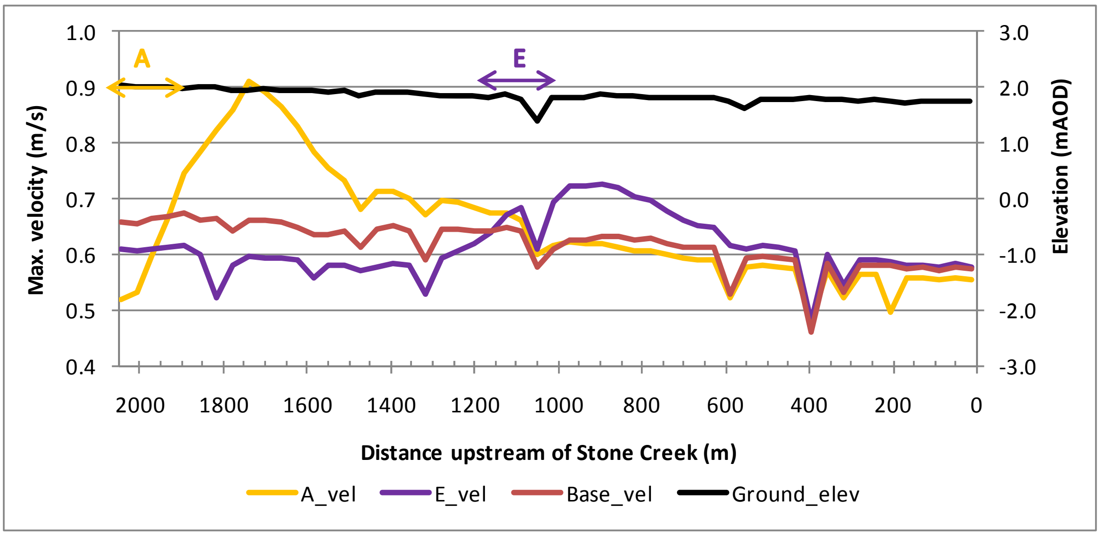


Figure 16 Maximum velocities over Foul Holme Sand for northern and southern breaches on Section 121

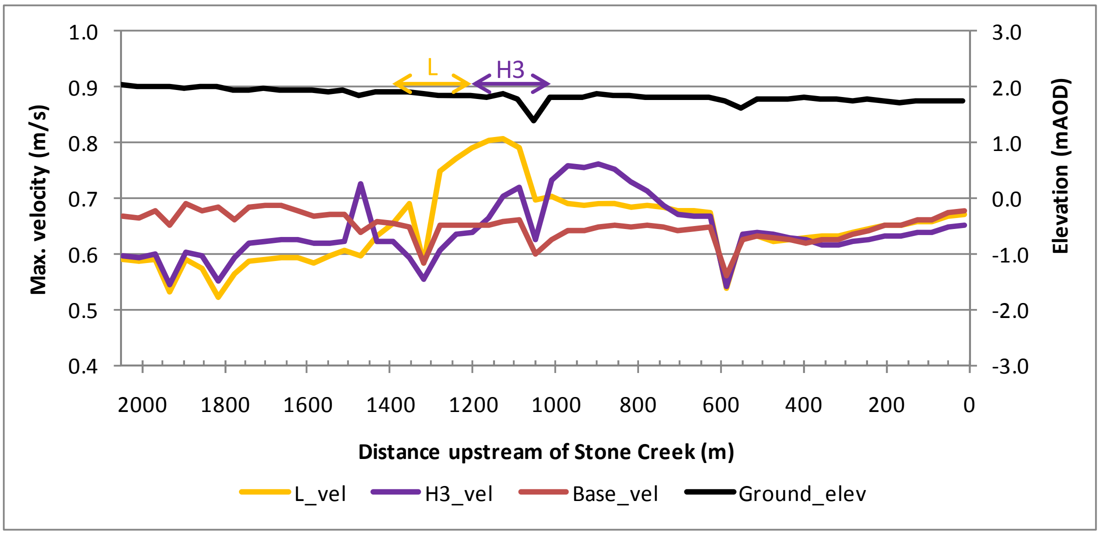


Figure 17 Maximum velocities over Foul Holme Sand for southern breach on Section 121

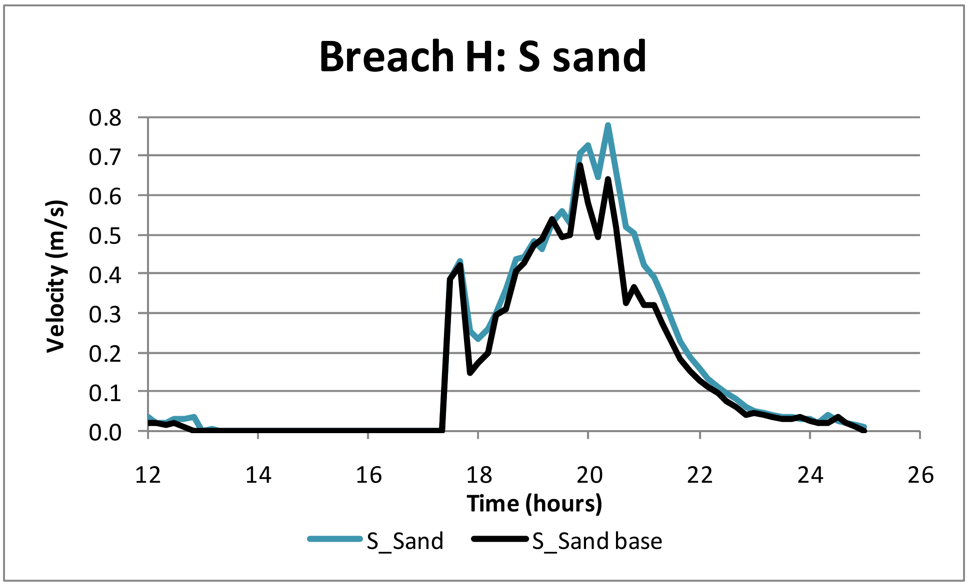


Figure 18 Velocities across Foul Holme Sand at Point 25 for Option H

3.6 VELOCITIES WITHIN THE COMPENSATION SITE

3.6.1 An area of high velocities was identified within the Compensation Site to the north of the southern breach as water flowed through a relatively narrow section of the site to reach the northern part of the site. A plot of the maximum velocities within and outside the Compensation Site is shown in Figure 19. This shows a zone of high velocity as expected in the area of the breach and an extensive area of similarly high velocities within the site to the north of the breach.

3.6.2 If the ground level in the area of high velocity in the Compensation Site is lowered by 0.5 m, but leaving a 50 m wide strip alongside the realigned defence where level is unchanged for Option H3 (Figure 5), the area of high velocity is reduced as illustrated on Figure 9. A high velocity area remains near the northern end of the breach and in an adjacent section of the Compensation Site.

3.6.3 Section 104 (see Figure 6), which is taken across the Compensation Site and includes the area of reduced level, is shown in Figure 20. The velocities in this area are illustrated at two points along this transect on Figure 21. Point P104-in (see Figure 6) located within the area of lowered ground (Figure 21a) shows that within the area of reduced ground level there is little change in the velocity profile with peak velocities on the flood tide being close to 1.5 m/s for both ground levels. Outside the area of lowered ground close to the flood defence at Point P104-out (see Figure 6) there is a predicted reduction in peak flood velocity from 1.3 to 1.1 m/s (Figure 21b) as a result of the lowered ground level. At both sites, peak velocities on the ebb tide (after hour 20) are in the range 0.6 to 0.7 m/s.

3.6.4 The high velocities inside the breach will require erosion protection for the realigned flood defences. Providing an area of lowered ground level is expected to reduce velocities close to the flood defence and so reduce the pressure on these defences.

3.6.5 The high velocities over the area inside the breach on the flood tide lead to a risk of erosion in this area. The risk will depend on velocities experienced during lower tides.

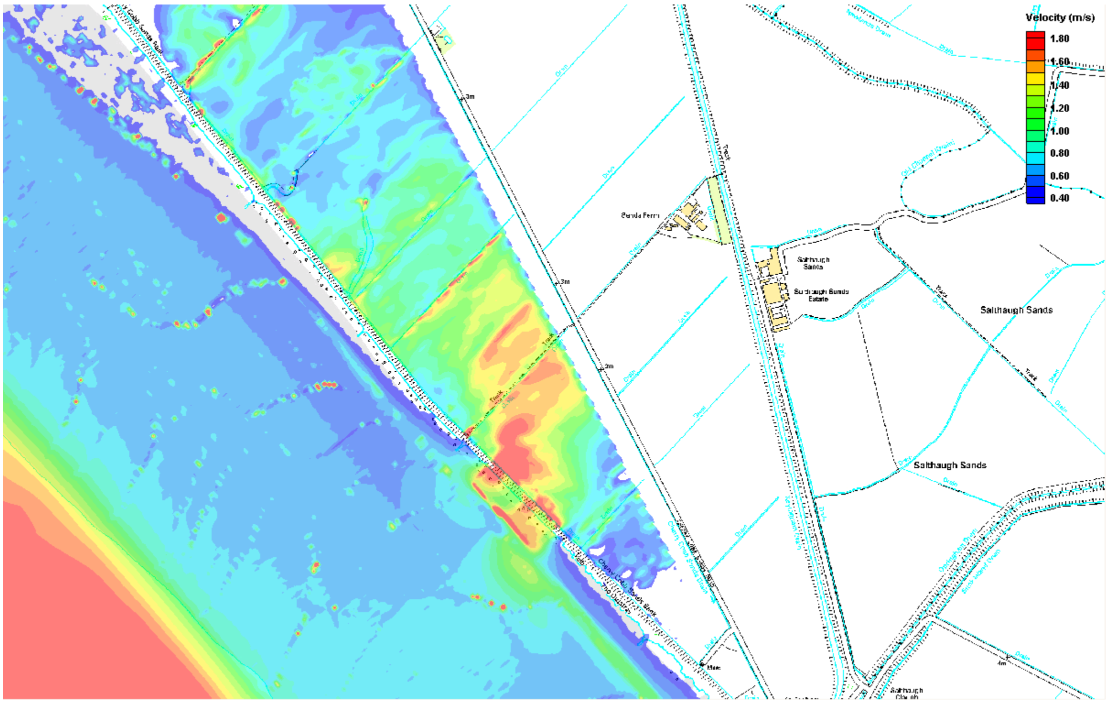


Figure 19 Maximum velocities within the Compensation Site (Option H2)

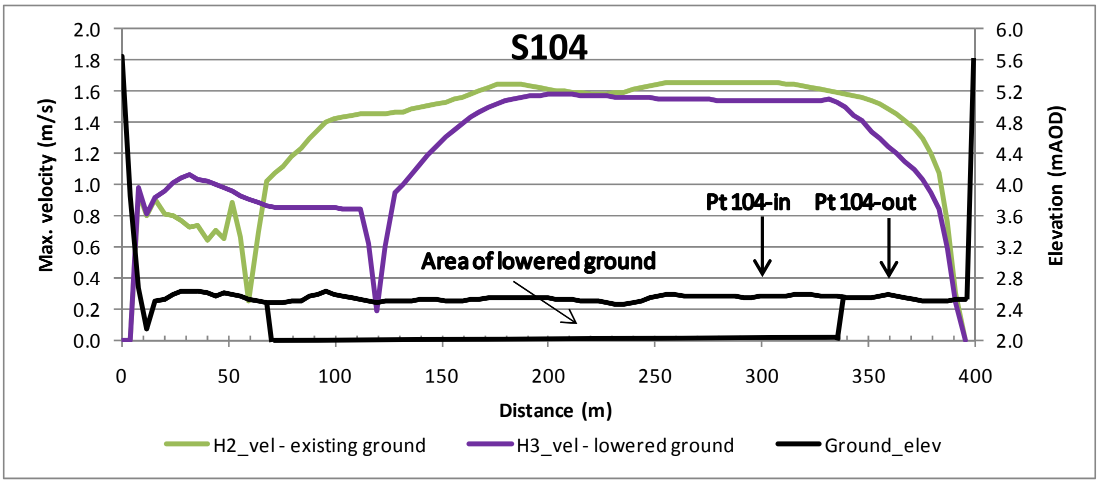
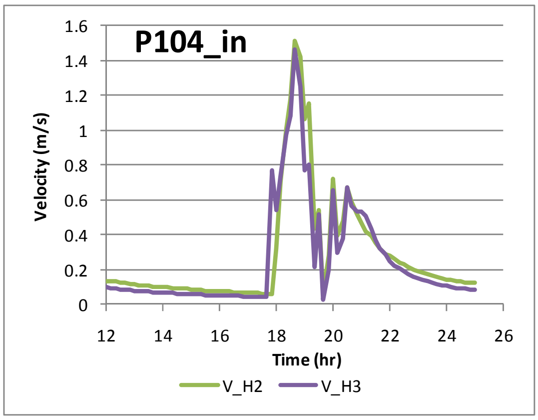
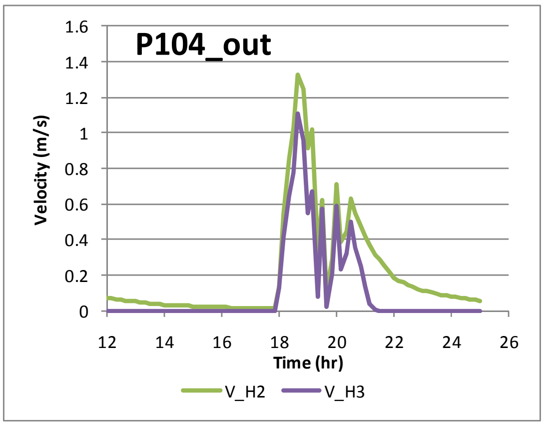


Figure 20 Maximum velocity on transect 104 across Compensation Site



21a) Velocities at Point 104-in



21b) Velocities at Point 104-out

Figure 21 Velocities within the Compensation Site for Options H2 and H3

4 *FINDINGS*

4.1.1 The model to design the breach of the Compensation Site was driven by flows and levels calculated in a preliminary version of the Humber model. This version of the Humber model used a simplified intertidal bathymetry and was only calibrated against a low spring tide.

4.1.2 The detailed model used for breach design utilised 2007 LiDAR to represent the intertidal bathymetry and a 2010 topographic survey of site ground levels. The available model boundary conditions were stretched to represent a high spring tide with a high tide level of 3.9 m AOD in the Humber close to Immingham. Despite these assumptions and simplifications, the model findings are unlikely to be seriously affected as velocities in the area of the Compensation Site model are primarily driven by the local bathymetry and the shape of the imposed tide curve, both of which are reasonably accurate.

4.1.3 The design tests indicate an area of increased velocities over Foul Holme Sand on the early ebb tide as a result of drainage of the Compensation Site through the breach. This is only likely to last for a short period until the falling tide exposes the top of the sandbank, after which all flows will be directed down Cherry Cobb Sands Creek.

4.1.4 Location of a breach as far south as practicable limits the increase in peak velocity over Foul Holme Sand and so is likely to minimise the risk of erosion of this intertidal bank. The use of the southern breach site is therefore recommended.

4.1.5 Flows on the ebb tide in Cherry Cobb Sands Creek are predicted to increase as a result of drainage from the Compensation Site. The increase is predicted to be most marked in the section of creek between the breach site and the Stone Creek drainage outfall. South (downstream on the ebb tide) of the Stone Creek outfall maximum velocities are not predicted to increase, but the duration of high velocities is likely to increase as a result of the Compensation Site drainage. At the downstream end of the creek, velocities are likely to remain high, as they are at present, so the existing dynamic nature of the channel is likely to continue.

4.1.6 The larger flows in Cherry Cobb Sands Creek are likely to lead to erosion increasing the width and depth of this creek, especially between the breach site and the Stone Creek outfall. South of this outfall, the creek is larger, so there is likely to be rather less change than upstream.

4.1.7 Increasing the breach length will reduce velocities through the breach, though this reduction becomes less marked as the breach lengthens. A longer breach is predicted to lead to greater loss of saltmarsh in front of the breach. Considering the disadvantage of losing this natural resource, there seems little benefit in increasing the breach length beyond 250 m which should limit the loss of fronting saltmarsh to about 2 ha.

4.1.8 One consequence of a southern breach is that velocities inside the Compensation Site on the flood tide are predicted to be fairly high as the incoming tide floods north to flood the remainder of the site. Local reduction of ground levels near the breach within the site has benefit in reducing velocities along the realigned flood defence but this reduction is unlikely to be sufficient to avoid the need for erosion protection to this defence. The protection already included to protect the flood defence embankment against wave attack will also be suitable to protect the toe of the embankment against erosion by tidal currents, provided the toe details are suitable.

5 RECOMMENDATION

5.1.1 A 250 m long breach with an invert level of 2 mAOD is recommended situated towards the southern end of the Compensation Site. Removal of some of the saltmarsh fronting the breach site down to 2 mAOD is recommended, with the expectation that all the saltmarsh fronting the breach site will be eroded away fairly rapidly, leading to a direct loss of about 2 ha of saltmarsh.