Envirommental Statement

Appendix D

Coastal Morphology Report

D

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EASTSIDE DEVELOPMENT, GIBRALTAR

EBG *Marine Engineering & Construction*

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Volume 3: Coastal morphology

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Volume 3: Coastal morphology

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ABSTRACT:

EBG is involved in the development of a new scheme on the east side of Gibraltar. To support the development phase of the scheme, WL | Delft Hydraulics was requested by EBG to execute various hydraulic studies to provide relevant input to the Environmental Impact Assessment process concerning the scheme.

The hydraulic studies carried out to support the Environmental Impact Assessment process address the following aspects:
1. Flow conditions
2. Normal wave conditions
3. Coastal morphology

This report presents the results of the above item 3: Coastal morphology

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Executive Summary

General

EBG is involved in the development of a new scheme on the east side of Gibraltar. To support the development phase of the scheme, WL | Delft Hydraulics was requested by EBG to execute various hydraulic studies to provide relevant input to the Environmental Impact Assessment process concerning the scheme. The project is referred to as “Eastside, Gibraltar”.

The objective of the study reported here is to consider the following coastal morphology issues related to the envisaged scheme:

- Coastal impact.
- Impact on the coastal profile.
- Estimate of annual sediment infill rates of borrow pits and other dredged areas.
- Guideline for beach nourishment works.
- Assessment of areas affected by dredging induced sediment plumes.

Figure 1.2 presents the layout considered in this study. It is proposed that fill material for the land reclamation will be dredged from the borrow areas as indicated in Figure 1.1. In addition the influence of the Both Worlds project at Sandy Bay was considered. Special attention was paid to transboundary effects (crossing the border of Gibraltar).

The approach and results of this coastal morphology study are presented in this report.

Study approach

The study has been carried out with support of different numerical models. For the prediction of the morphological impact of the planned works the shoreline model UNIBEST-CL+ was applied. The shoreline model is particularly suitable for the evaluation of large scale changes over long periods of time (years to decades). For details near structures (such as the planned scheme) the model DELFT3D provided additional information. With this model 2DH (2-Dimensional Horizontal) sediment transport patterns and seabed changes were computed. Cross-shore impacts in the shielded zone near the planned scheme and guidelines on beach nourishments were provided on the basis of expert judgement supported with a cross-shore equilibrium profile model (Dean, 1977).

For the prediction of the infill rates of dredged areas the model DELFT3D was applied. Sediment transport and (initial) erosion-accretion patterns were computed, on the basis of which the infill rates were estimated. Also the plume dispersion modelling was carried out with DELFT3D.

Results coastal impact- Eastside, Gibraltar

Shoreline analysis (satellite pictures) and numerical sediment transport modelling both indicate that the net longshore sand transport in the study area is negligible.

The impact of the scheme is predicted to be limited to Eastern Beach and Catalan Bay (see Figure 5.11). Due to a combination of reorientation of the shorelines and offshore sand loss in these areas, directly north of the planned development a maximum coastline displacement of 60 to 70 meter in seaward direction should be expected within the first years after construction, while for Catalan Bay a maximum seaward displacement of the coastline (immediately adjacent to the planned development) of 20 to 30 meter is expected during this period. Consequently erosion in the order of 15 to 25 meter is expected south of the central groyne at Eastern Beach and 5 to 10 meter erosion at the south of Catalan Bay. Due to offshore sand loss, the local (horizontal) erosion at Catalan Bay and at the southern section of Eastern Beach may increase an additional 20 meter after 25 years. The morphological impact on the second section of Eastern Beach (north of the central groyne) is much smaller and consists of a maximum accretion of at most some meters near the central groyne and a shoreline retreat of similar magnitude near the northern groyne. No significant impact is predicted north of the northern groyne of Eastern Beach or south of Catalan Bay.

Results coastal impact- In-combination effects
The “Both Worlds” project (see Figure 1.4), planned south of Sandy Bay, is expected not to affect Sandy Bay and/or the project area.

Results coastal impact- Transboundary effects
For the situation with the scheme only as well as for the combined scheme with the “Both Worlds” project, no cross-border coastal impact is predicted. However, it is recommended to implement regular monitoring north of the northern groyne of Eastern Beach. If any (temporary) adverse effects are found (although no adverse transboundary effects due to the planned development are expected), it could be considered (as an added contingency measure) to mitigate these effects by means of a small sand buffer north of the northern Eastern Beach groyne.

Results coastal profile slope
On the basis of the changes in wave climate it is predicted that the area in which the coastal profile slope is potentially influenced will be limited to Eastern Beach and Catalan Bay. The assessment for these beaches however indicated that even for these areas the changes to the coastal profile slope will be not significant.

No significant changes of the coastal profile slopes are predicted for the areas north of Eastern Beach and south of Catalan Bay.

Results infill rates dredged areas
The infill rates of the northern borrow area are predicted to be small. Estimated volumes of total deposited sediment in this borrow area after 1 year range from 50 to 500 m^3, which corresponds to an average (over the entire borrow pit area) sedimentation rate of about 1 mm/yr at most. However, the main deposition is expected to be close to the edges where gradients in the sediment transport are largest.

The infill rates in the southern borrow area are also predicted to be very small. It is estimated that the total amount of sedimentation that takes place ranges from 1,400 to 12,000 m^3 after 1 year (corresponding to 1 –8 mm/yr sedimentation).

Due to the low percentage of sand trapping by the borrow areas any adverse (erosion) effect on the surrounding seabed is predicted to be a very slow process. In addition, since the main part of these effects occur well below the closure depth, any effects of sand re-distribution around the borrow pits on the coast are expected to be very small.

Guidelines on beach nourishment

Given the impact of the scheme on the beaches of Eastern Beach and Catalan Bay some maintenance may be required. This beach nourishment is principally required at the stretches of Eastern Beach and Catalan Bay immediately adjacent to the planned development. Beach nourishment guidelines on period, location, beach slopes etc. are provided in this report.

No beach nourishment is expected to be required for other recreational beaches in the area.

Areas affected by dredging induced sediment plumes

Maximum suspended sediment concentrations of more than 512 mg/l[1] are expected within the Gibraltar Marine Nature Reserve, when the southern borrow area is used. If dredging takes place from the northern borrow area, maximum sediment concentrations of about 64 – 128 mg/l are expected across the Spanish border. The distance across which the sediment plume with concentrations above 64 mg/l extends into Spanish waters, is about 400 m during spring tide conditions and 200 m during neap tide.

From the modelling results it can also be concluded that least effects are expected during Scenario 1 (construction of trench and sea defences, see Figure 7.3a) compared to the other scenarios.

Sedimentation rates in the considered sensitive areas are estimated to be in the order of mm's – cm's only. Only for the scenarios with TSHD dredging, sediment deposition may be higher; during dredging at the northern borrow area, sediment deposition at wreck location W2 (see Figure 7.1) and across the Spanish border may be up to 0.1 m at water depths of 15 m. During dredging within the southern borrow area, the total volume of the sedimentation within the Gibraltar Marine Nature Reserve after 7 weeks of dredging is expected to be approximately 100,000 m^3, of which 80,000 m^3 is expected in an area of 1,000,000 m^2 (average sedimentation of approximately 0.1m). Most of this sedimentation will take place within the southern borrow area, which will have already been disturbed by the deepening.

[1] *In this report at several locations it is indicated that concentrations exceed 512 mg/l. This value is exceeded very locally around spill locations. Further quantification of such high local concentrations would require a different type of modelling than the applied midfield modelling.*

Executive Summary

Contents

List of Figures

List of Tables

List of Symbols

Symbol	**Units**	**Description**
H_s	m	significant wave height
T_p	s	peak period
S	m^3/yr	net longshore sediment transport
z	m	water depth
y	m	distance from the shoreline
$d_{l,t}$	m	closure depth over t years
w	m/s	sediment fall velocity
g	m/s^2	acceleration of gravity

1 Introduction

1.1 Background

EBG (Europese Bouw Groep) is involved in the development of a new scheme on the east side of Gibraltar. Figure 1.1 indicates the location of the project site. The scheme is planned at the existing Rubble Tip area between Eastern Beach to the north and Catalan Bay to the south. Directly north of Eastern Beach lies the Gibraltar airport runway and the Spanish border. Directly south of Catalan Bay the coastline is formed by rock outcrops. Sandy Bay is located further south (see Figure 1.2).

To support the schemes development phase, WL | Delft Hydraulics was requested by EBG to execute various hydraulic studies to provide relevant input to the Environmental Impact Assessment (EIA) process concerning the scheme. The project is referred to as "Eastside, Gibraltar".

EIA scenarios

In accordance with EIA legislation and guidance applied in Gibraltar, various impact scenarios have to be assessed. A brief description of these scenarios is given below.

<u>*Eastside Gibraltar Impacts*</u>
The development is subject to the EIA process, which requires the project proponent to provide environmental information including descriptions of the likely significant impacts of the proposals in terms of changes to the existing environmental conditions. The proposals include a high quality mixed use residential development on the east side of Gibraltar. Figure 1.2 shows the proposed scheme. The site of the proposed development will utilise the existing area of reclaimed land (presently a rubble tip) and will require a further eastward land reclamation of about 100m.

It is proposed that fill material for the land reclamation will be dredged from the borrow areas as indicated in Figure 1.1. If the material is dredged from one of the borrow areas the expected average deepening will be 0.4m (southern borrow area) or 0.9m (northern borrow area), see EBG (2007).

<u>*In-combination Effects*</u>
As part of the EIA the impacts of the scheme have to be considered in combination with another development envisaged at the east side. This development envisaged at the east side and considered in this scenario is the Both Worlds Project at Sandy Bay. This relatively small project is located at the southern end of Sandy Bay, see Figures 1.3 and 1.4. The development comprises a small land reclamation with 10-30m seaward extension over a shore parallel distance of about 50-60m. The land reclamation will be protected by a shore protection.

Transboundary Effects
Given the close vicinity of the project location to the Spanish border, the transboundary effects of the above scenarios have to be considered in the EIA process. The location of the border, as derived from the Admiralty Charts, has been included in Figures 1.1 to 1.3, and delineates the border between Gibraltar and Spain's territorial waters for the purpose of assessing transboundary effects.

1.2 Scope of work

The scope of work for the hydraulic studies covers the following:

1. Flow conditions
The flow conditions at the project site have been determined by numerical flow modelling. For this purpose a flow model was prepared for the project area. Simulations were carried out with and without the scheme. This part of the study covers:

- Assessment of impact on tidal characteristics
- Determination of impact on storm surge behaviour
- Assessment of impact on current flow patterns
- Prediction of pollutant dispersion
- Assessment of beach cleansing and bathing water quality

2. Normal wave conditions
Wave conditions were studied to determine the normal wave conditions along the coast at the project site. Simulations were carried out with and without the scheme. This part of the study covers:

- Assessment of offshore and nearshore wave conditions
- Impact of the scheme on the annual nearshore wave climate.

3. Coastal morphology
Various coastal morphology aspects were determined using 2D and 1D morphological models. This part of the study includes:

- Assessment of coastal impact
- Determination of sediment infill rates of dredged areas
- Prediction of dredged plume dispersion
- Impact of the development on the cross-shore beach profiles
- Guidelines on beach maintenance work

The approach and results of the above three study items have been reported in separate volumes.

1.3 Aim of the present report

The objective of the study reported here is to consider the following coastal morphology issues related to the Environmental Impact Assessment (item 3 above):

- Coastal impact (alongshore and cross-shore), including recommendations for beach maintenance works (nourishments).
- Infill rates of borrow pits and dredged areas (e.g. navigation channel).
- Area affected by dredging induced sediment plumes.

1.4 Outline of the present document

The study approach is described in Chapter 2. Relevant environmental conditions are summarised in Chapter 3. In Chapter 4 a description of the coastal system is presented. The coastal impact of the planned works is predicted in Chapter 5. For the prediction of the infill rates of dredged areas reference is made to Chapter 6. The results of dredging plume modelling are described in Chapter 7. Conclusions and recommendations are presented in Chapter 8.

2 Study approach

The three study items described in Section 1.3 have been studied with support of different numerical models.

Coastal impact, shoreline changes
For the prediction of the morphological impact of the planned works the shoreline model UNIBEST-CL+ was used. In this model the cross-shore movement of a single line representing the coastline is computed from longshore gradients in the net longshore sediment transport. Longshore sediment transport is calculated from wave and flow data obtained from wave and flow modelling results presented in Volumes 1 and 2 (thus including longshore variation in wave conditions) and sediment transport formula included in the Unibest model. Seaward and landward movements are termed accretion and erosion respectively. Further details of the Unibest model are given in Appendix A. The shoreline model is particularly suitable for the evaluation of large scale changes over long periods of time (years to decades).

For details near structures (such as the planned scheme) the model DELFT3D provided additional information for the coastal impact assessment. In this model 2DH (2-Dimensional Horizontal) sediment transport patterns and seabed changes are computed. This model is less suitable for the evaluation of long periods of time.

By using these two models the large-scale impact due to interruption of the longshore transport and due to local shielding of the waves by the planned scheme was predicted in an alongshore direction.

Coastal impact, changes in cross-shore profiles
In addition, cross-shore impacts in the shielded zone near the proposed scheme were estimated using a simple cross-shore equilibrium profile model (Dean, 1977), results of previous studies carried out in the project area and expert interpretation.

Beach nourishment guidelines
Guidelines for beach maintenance works with minimum effect on the recreational use of the beaches were presented on the basis of a desk assessment.

Infill rates of borrow pits and dredged areas
For the prediction of the infill rates of dredged areas the model DELFT3D was applied. Sediment transport and (initial) erosion-accretion patterns were computed, on the basis of which the infill rates were estimated.

Plume dispersion
The dredging induced sediment plumes and deposition were also simulated using the DELFT3D model. In this assessment the combined effect of flows, waves and morphology were simulated to predict the dredging induced sediment plume behaviour.

3 Environmental conditions

3.1 Introduction

Relevant environmental and bathymetric data are summarized in this chapter. Coastline characteristics are described in Sections 3.2 to 3.4, and hydraulic conditions are discussed in Sections 3.5 and 3.6.

3.2 Description of the coastline

In this study not only the coastline of Gibraltar is considered, also the coast of Spain up to La Atunara harbour is considered since the impact on the Spanish coast should be taken into consideration in the study. In addition, the observed shoreline behaviour around La Atunara harbour may provide information for the shoreline model calibration.

The coastline at the east side of Gibraltar is relatively straight and only changes in orientation some kilometres in Spain (near La Atunara harbour). In this report the coastline orientation is expressed as the orientation of the seaward directed shore-normal relative to the North (measured clockwise). For the Gibraltar East coast this orientation is about 97ºN, north of La Atunara harbour it is 105ºN. The characteristic features (Figure 4.1) along the coast (from north to south) are:

- La Atunara harbour protruding approximately 300 m into the sea.
- Sandy beach between La Atunara harbour and the border Gibraltar-Spain.
- Sandy beach between the border and the Rubble Tip area (Eastern Beach).
- Two groynes along Eastern Beach protruding about 50 meters into the sea.
- Rubble Tip area protruding on average about 200 m into the sea.
- Sandy beach: Catalan Bay.
- Rocky coast between Catalan Bay and Sandy Bay with a moderately steep profile.
- Sandy beach: Sandy Bay.
- Rocky coast south of Sandy Bay with a steep profile.

3.3 Bathymetry and coastal profiles

3.3.1 Bathymetry

In this study the same bathymetry of the Strait of Gibraltar and the adjacent Mediterranean Sea (the Alboran Sea) was used as in the tidal flow and wave studies (WL | Delft Hydraulics 2007a and b). Water depths in the Alboran Sea range between several hundreds and two thousand meters, with an average depth of about 900 meters. The east coast of Gibraltar is rather steep with beach slopes of 1:15 to 1:30, slopes of 1:40 to 1:60 between OD and

OD -10 m and generally steeper slopes below this level, with a steepness of over 1:8 in deep water (deeper than OD -40 m).

3.3.2 Characteristic coastal profiles

Several bathymetric surveys have been carried out on the east Gibraltar coast. The first survey available for this study was from 1968. More recent bathymetric data were measured in 1996, 1999 and 2005, covering the nearshore area (shallower than OD -10 m) from Sandy Bay up to the Spanish border including part of the adjacent waters in Spain. Additional information was available from a survey carried out in 1999 (by Esgemar) for the surf zone, a land survey of the nearby airport and admiralty charts for the Spanish waters (Admiralty chart 144, 1448 and 142).

The bathymetrical surveys give a rather consistent picture of the profiles over time, as can be observed in Figures 3.1 to 3.3. These plots also show that morphological activity is small in the deeper parts of the profiles along the east Gibraltar coast. Coastal profile changes with time can be observed mainly in the zone between OD and OD -10 m, with the most dynamic part of the profile between OD and OD -6 m.

For the comparison of coastal profiles along the coast several cross-sections were selected, see Figure 3.4. Figure 3.5 shows an overview of the cross-shore profiles as established by interpolation of nearshore (Boskalis, 2006), surf zone (Esgemar, 1999) and land data. It can be concluded that the cross-shore profile of the east Gibraltar coast can be characterized with three representative profiles (viz. for Catalan Bay and Eastern Beach and for the Rubble Tip), as shown on Figure 3.6.

Cross-shore profiles north of Eastern Beach are relatively uniform up to the port of La Atunara, where there is a small change in profile (Figure 3.7). North of La Atunara harbour the cross-shore profile gradually changes to a more gentle slope (Figure 3.8). Two characteristic profiles can be distinguished for this part of the coast, as presented in Figure 3.9.

3.4 Sediment characteristics

Characteristics of the existing sediment on the beaches are described in the GoG Beaches Specification. In the GoG beaches specifications the grain diameters of the fractions on the beaches are reported as:

- D15 = 0.17 mm
- D50 = 0.28 mm
- D85 = 0.54 mm

Boskalis Westminster (2005) and Fugro (2000) describe the sediment at the beaches as fine to medium grained sand with a low content of silt. The sediment fall velocity of this sand is estimated at 0.035 m/s. In the Fugro (2000) study it is concluded that sand at offshore locations is poorly graded and ranges from loosely packed, in the top layer, to very dense, at about -2 meters below the top layer of the sand (Fugro, 2000). The sediment on the foreshore is of a similar size as the beach sediment, as illustrated in Figure 3.10.

3.5 Wind and wave climate

3.5.1 Wind

In WL | Delft Hydraulics 2007b the normal wind and wave conditions have been assessed. The main results are summarized below.

Two (opposite) wind directions are dominant in Gibraltar. Both western and eastern winds dominate the area. In the summer the prevailing wind is from the east, which is referred to as the 'Levanter'. During winter months the wind is prevailing from the west; This wind is locally known as the 'Poniente'. This wind can in winter occasionally be up to force 7 to 8, but has a more limited influence on the east Gibraltar coast, as it is on the lee side of the Gibraltar rock. Figure 3.11 presents a typical wind rose based on averaged data (HIRLAM dataset) from 1999 to 2005.

3.5.2 Waves

The yearly offshore climate in this region is characterized by about 50% waves from the west and about 25% from the east, and the rest by generally mixed seas with waves reaching Gibraltar from east and west simultaneously. This will generally lead to double peaked spectra with an eastern and a western component. Waves with peak periods longer than 10 seconds can come from both directions. In Figure 3.12 a spatial overview is given of the wave climate offshore.

In WL | Delft Hydraulics 2007b wave propagation computations have been carried out to derive the nearshore wave climate. The results have been summarised as typical nearshore wave roses (at a depth of ODN -6 m), as presented in Figures 3.13 and 3.14 for the present situation (without the scheme) and the proposed future situation respectively. From a comparison of the figures the effect of wave shielding just north and south of the scheme is very small.

3.6 Water levels and currents

Tidal flow in the area of the Strait of Gibraltar is driven by the tides coming from the Atlantic via the Gulf of Cadiz from the west, and the internally generated tides in the Mediterranean coming via the Alboran Sea. The combined effect of these tides results in a complicated tidal behaviour in the Strait. It is characterised by a large gradient in tidal amplitude: the tidal range reduces from about 2 m at Cadiz on the west side of the Strait to approximately 1 m at Gibraltar on the east side. The vertical datums at the east Gibraltar coast are indicated in Figure 3.15 and Table 3.1. This report uses Ordnance Datum for the presentation of bathymetric and water level related data.

A detailed flow model has been set up for the region in WL | Delft Hydraulics (2007a). This study characterises the tidal current pattern in the Strait of Gibraltar as mainly barotropic (currents run parallel with the direction of the water level gradient). The tidal streams can reach speeds up to 1 m/s along the axis of the strait. Because at the east side of Gibraltar the tidal flow is no longer confined by the dimensions of the Strait, less pronounced tidal streams can be expected there. Maximum tidal currents nearshore along the east Gibraltar coast (between OD -6 m and OD -8 m) range from 0.3 m/s at spring tide to 0.1 m/s at neap tide.

	Level relative to CD (m)	Level relative to OD (m)
HAT	1.1	1.0
MHWS	1.0	0.9
MHWN	0.7	0.6
MSL	0.5	0.4
MLWN	0.3	0.2
Alicante Datum	0.2	0.2
MLWS	0.1	0.0
Ordnance Datum	0.1	0.0
CD	0.0	-0.1
LAT	-0.1	-0.2

Table 3.1 Tidal levels at the project site

4 The coastal system

4.1 Introduction

A good understanding of the coastal system and its governing processes is required to make predictions of the impact of the planned development on the coast. A disturbance of the system (caused by the scheme) can be quantified if the sediment balance is known. Historical data from charts and satellite images were used to gain insight into the direction and magnitude of the net longshore transport in the area. The data were used to calibrate the numerical shoreline model.

In Section 4.2 the main relevant coastal physical processes are briefly described. In Section 4.3 the results of the shoreline analysis, based on satellite pictures, are described. On the basis of this analysis conclusions on shoreline behaviour and the net sediment transport are drawn, which form the basis for calibration of the shoreline model. In Section 4.4 the setup, calibration and the results of shoreline modelling for the existing coastline (without the planned works) are described.

4.2 Processes

Coastal processes can be categorized as longshore and cross-shore processes:

- Longshore processes, causing redistribution of sediment along the coast. In general, interaction between different coastal sections may occur through longshore transport. This transport is generated by waves, but is in some cases affected by tidal currents or directly by wind. Alongshore gradients in this transport may cause local erosion or accumulation of sediment. In this study such longshore transport gradients have been identified on the basis of satellite images of the coast and numerical models.
- Cross-shore processes, resulting in supply or loss of sediment normal to the coast. A potential sediment source can be onshore directed sediment transport by waves. Sediment loss can be the result of wind (moving sand inland), wave-related offshore flow (undertow, rip currents) or sea level rise (reshaping the cross-shore profile).

4.3 Shoreline analysis

Satellite pictures showing historical coastlines were acquired for characteristic moments in time (Table 4.1). These satellite images give an overview of coastline behaviour over a period of 14 years (1991 - 2005). Special attention was given to the effect of construction of the port of La Atunara (between 1991 and 1992). Since this port interrupts the longshore transport along the coast, the shoreline behaviour around it provides information on the direction and rate of the net longshore transport. The pictures are presented in Figure 4.1.

Date		Water level [m, MSL]	Accuracy [m]	Source
09-11-1991	11:36 GMT	0.01	±20	Spot
24-10-1992	11:06 GMT	0.4	±15	Spot
14-01-2001	11:03 GMT	-0.33	±8	Ikonos
28-05-2005	11:22 GMT	-0.35	±5	Ikonos

Table 4.1 Satellite images, dates with corresponding tidal water levels

The visible coastlines are related to certain tidal water levels (Table 4.1). These water levels were estimated with a tidal hindcast model on the basis of the date and time of the picture. Since the water level to some extent determines the location of the waterline, all the coastlines were transformed to a coastline at MSL. On the basis of the beach slope (estimated from Section 3.3.1) the horizontal correction was calculated (Table 4.2). For a good interpretation of these coastlines the accuracy of the images needs to be taken into account, especially for the older images. The different satellite images have different accuracies, as indicated in Table 4.1.

Year		**1991**	**1992**	**2001**	**2005**
Tidal waterlevel (m, MSL)		0	0.4	-0.33	-0.35
Beach	slope	horizontal shift (m) (+ = seaward shift)			
Catalan Bay	1:15	0	6	-5.0	-5.3
Eastern beach	1:20	0	8	-6.6	-7.0
La Atunara (south)	1:30	0	12	-10	-11
La Atunara (north)	1:30	0	12	-10	-10.5

Table 4.2 Applied horizontal corrections of the shoreline

In the following the derived shorelines (including above mentioned corrections) are discussed.

Note that it has been assumed that in the considered period (November 1991 to May 2005) the shoreline position was not affected by any beach nourishment or dredging activities (no data on such activities were available for this study). In other words, it was assumed that the shoreline changes are due to natural processes.

4.3.1 East Gibraltar

Rubble Tip area

On the satellite images presented in Figure 4.2 it can be seen that the Rubble Tip, at the east coast of Gibraltar, was extended in the period between 1991 and 2001. This does not appear to have had a significant impact on the adjacent beaches (Catalan Bay and Eastern Beach) as discussed below.

Catalan Bay

Figure 4.3 shows the four shorelines for Catalan Bay. Slight coastal changes can be distinguished from the 2001 satellite image at Catalan Bay. For the years 1991, 1992 and 2005 the shorelines are very similar. In 2001 an overall reorientation of the coast can be observed. This is expected to be a temporary reorientation which may have been caused by the conditions immediately prior to the 2001 picture (e.g. a long period of persistent waves from relatively southerly directions). The coastal orientation can adjust relatively fast to temporal changes and seasonal fluctuations in wave conditions in small bays, such as Catalan Bay. The shoreline position of 2005 indicates that the re-orientation in 2001 was temporary. Figure 4.4 shows estimated accuracy ranges around the coastlines. This figure illustrates that no long-term sedimentation or erosion can be distinguished, when accuracy constraints are taken into account (for example: the range around the 2005 coastline lies well within the range around the 1991 coastline. In fact, all ranges have some overlap).

Figure 4.5 shows the envelope around all coastlines presented in Figure 4.3. From this presentation at each location along the considered coastal stretch the most landward and seaward position is presented. This gives an impression of the effects of temporal fluctuations of the shoreline. On the basis of the images this can be estimated to be less than 20 m.

Eastern Beach

Figure 4.6 shows the four shorelines for Eastern Beach. In a similar way to that discussed above for Catalan Bay, the beach orientation fluctuates along Eastern Beach, but no long-term changes in the coastline can be distinguished. The differences between the coastlines of 1991 and 2005, for example, are very small. It is therefore expected that changes in beach width are caused by temporal changes in wind and wave climate. Figure 4.7 gives the bandwidth of accuracy for the determination of the coastline from the satellite images. All accuracy ranges have some overlap.

The envelope wherein coastal fluctuations (i.e. due to reorientation) can be determined from the images is displayed in Figure 4.8. From this figure the order of magnitude of such fluctuations is estimated to be less than 20 m.

La Atunara (Spain)

La Atunara harbour was constructed in 1992, see Figure 4.9. Even though this harbour forms an interruption of the longshore transport in the area, shoreline changes with time around La Atunara harbour are small. Reference is made to Figures 4.10 and 4.11. The shorelines in 2001 and 2005 are almost identical. Local to the south of the harbour the shoreline seems to have moved landward in the period between 1991 and 2005. In this area in particular the shoreline position in the 1992 image is located relatively seaward (also compared to the 1991 shoreline). This might partially be due to disturbances during port construction (it is not known where the material dredged from the port basin between 1991 and 1992 was dumped. However in 1992 there seem to be two “bumps” in the shoreline close to the port, which may indicate that some material has been dumped there). A slight tendency towards erosion south of Atunara harbour can be identified between 1991 and

2001. This may be a leeside effect of interruption of a south-going transport by the harbour breakwaters. However, the occurrence of a net south-going transport is not clearly confirmed by accretion north of the harbour. Taking the inaccuracy ranges into account (Figure 4.11) overall accretion north of the port in the period 1992-2005 is negligible (observed change is mainly due to some re-distribution of sand).

It should be noted that in the area of La Atunara harbour relatively large corrections of the shorelines have been applied to account for water level differences, based on estimated relatively gentle beach slopes near the water line, see Table 4.2. In this correction the 1992 shoreline has been shifted 12 m seaward and the 2001 and 2005 lines approximately 10 m landward. Part of the presented landward movement (Figures 4.10 and 4.11) of the shoreline in the period 1992-2005 is due to these corrections. Over-estimation of the corrections (for example if the actual beach slope would have been somewhat steeper than assumed at the time the picture was made) would lead to over-estimation of the shoreline displacement here (for example, if slopes are 1:15 instead of 1:30, corrections should be half the applied values). Furthermore, it has been assumed that there has been no human interference with the beaches.

Considering the negligible sand accumulation north of La Atunara harbour in a period of 14 years and the above discussion on the observations south of the harbour, the net longshore transport near La Atunara harbour is concluded to be very small. If there is any noticeable net transport in this area it tends to be southward directed. North of La Atunara the beach becomes wider and beach slopes are gentler. It is our impression that on larger time scales the Gibraltar coast has been fed with sand from the north, however no historical evidence for this has been found. The bay shapes of the coast further north of La Atunara confirm that (at least in the past) the net transport in the area north of La Atunara was directed southwards. It cannot be firmly distinguished from the satellite images whether this net southward transport is still present today. As it was indistinguishable in the most recent period (2001-2005) it was concluded that the shoreline behaviour around La Atunara harbour confirms that the net transport in the study area is negligible.

The conclusion of a negligible net transport along the considered coast of Gibraltar was also drawn in WL | Delft Hydraulics (2000) on the basis of considerations related to the two groynes at Eastern Beach combined with longshore transport computations.

4.4 The coastline model

The results of above analysis are supported by shoreline modelling with the model UNIBEST-CL+ and a 2D-model analysis with Delft3D. These models each offer different approaches to the schematization of the morphological effects. In this section the shoreline model UNIBEST-CL+ is described. For a description of Delft3D reference is made to Chapter 6.

4.4.1 Model setup UNIBEST-CL+

The model setup for UNIBEST constitutes of the following aspects:

- Coastline schematization.

- Boundary conditions.
- Model parameters.
- Sediment transport computations.
- Calibration of transport in the shoreline model.

These aspects are described in more detail below.

Coastline schematization

The coastline has been set up in real world coordinates (European Datum UTM-30N), for which data was used from maps (Admirality Chart 144) and satellite images. For locations where the coastline changes abruptly the 1D-approach requires some smoothing, which was applied for the rocky parts of the coast south of Catalan Bay.

In the model the Rubble Tip area is protected by a revetment and groynes at both sides. In this way sedimentation/erosion is set to zero and longshore transport is limited along the Rubble Tip. At La Atunara a similar setup is chosen, thus simulating the interruption of the longshore transport by the harbour breakwaters at that location. Figure 4.14 shows the coastline schematization as well as the characteristics and location of the groynes. The cross-shore profiles (Section 3.3) are defined at 14 locations (Figure 4.12).

Boundary conditions

For the assessment of the offshore and nearshore wave climate reference is made to WL | Delft Hydraulics (2007b). This climate is characterised by 66 wave conditions, see Table 4.3. For each of these conditions wave propagation towards the coast has been calculated with the model Delft3D-WAVE (SWAN). Characteristic results of the wave modelling showing the transition from offshore to nearshore wave roses are presented in Figures 3.12 (offshore roses) and 3.13 (nearshore roses). The computed nearshore wave climates at OD -8 m were used as a boundary condition at 14 locations (Figure 4.12) in the UNIBEST-CL+ model.

In addition, local wave conditions near large structures were derived from the Delft3D-WAVE model. With these local conditions, wave sheltering around large structures (such as the planned development at the Rubble Tip) is accounted for. Effects of changes in the nearshore roses due to the planned development are illustrated in Figure 3.14 (compared with Figure 3.13, which shows the situation without the scheme). In the UNIBEST-CL+ model these local conditions around the scheme were defined at OD -4 m and OD -6 m. To include diffraction around small structures the wave heights of the local conditions were adjusted according to formulations for diffraction around structures by Kamphuis (1992), which are based on data presented by Goda (1985). For more information reference is made to Appendix A.

Model parameters

Model parameters are parameters used to define the driving forces (hydrodynamic forces) and transport characteristics. These parameters can be divided into two categories:

- Sand transport formula.
- Surfzone characteristics.

The longshore sand transport formula used in this study was the Bijker formula (Bijker, 1971). The median sediment diameter was set at D_{50} = 0.28 mm, with a fall velocity of 0.035 m/s. In UNIBEST-CL+ the surf zone characteristics of the waves are computed with a wave propagation model. This model includes the basic equations for wave propagation and decay, as well as the cross-shore water level set-up. The surf zone characteristics are characterized by a wave breaking coefficient (γ, estimated at 0.8), which depends on wave steepness (relation between wave height and wave length). With this coefficient the depth at which a wave breaks can be calculated. A typical bottom roughness of 0.1 m was used to calculate the effect of bottom friction on the waves.

Wave height	Period	Direction	Duration
0.29 m	8.6 s	68.0 °N	4.4 days
0.3 m	6.2 s	63.4 °N	19.5 days
0.33 m	2.6 s	93.8 °N	8.6 days
0.71 m	6.9 s	107.7 °N	22.4 days
0.6 m	6.9 s	104.0 °N	34.5 days
0.4 m	2.6 s	87.9 °N	20.2 days
0.16 m	6.2 s	81.8 °N	4.7 days
0.16 m	10.6 s	111.2 °N	4.1 days
0.11 m	1.7 s	81.8 °N	7.4 days
0.21 m	13.1 s	129.4 °N	3.8 days
0.17 m	9.5 s	152.7 °N	4.7 days
0.17 m	9.5 s	155.8 °N	33.5 days
0.17 m	9.5 s	152.7 °N	21.0 days
0.16 m	11.8 s	125.6 °N	34.7 days
0.17 m	10.6 s	142.7 °N	19.3 days
0.17 m	11.8 s	107.2 °N	4.0 days
0.76 m	6.9 s	77.0 °N	0.08 days
0.86 m	10.6 s	75.7 °N	0.39 days
1.16 m	6.2 s	97.2 °N	3.8 days
1.08 m	6.2 s	106.7 °N	18.6 days
1.02 m	6.9 s	110.0 °N	15.2 days
0.86 m	7.7 s	112.3 °N	2.6 days
0.76 m	7.7 s	95.7 °N	0.66 days
0.24 m	10.6 s	150.7 °N	0.50 days
0.46 m	3.3 s	111.1 °N	0.47 days
0.32 m	10.6 s	160.6 °N	0.50 days
0.25 m	8.6 s	171.1 °N	1.6 days
0.24 m	8.6 s	167.8 °N	10.2 days
0.25 m	8.6 s	167.6 °N	6.7 days
0.28 m	9.5 s	170.7 °N	4.6 days
0.27 m	8.6 s	209.2 °N	3.3 days
0.26 m	1.4 s	280.1 °N	0.78 days
1.31 m	9.5 s	93.5 °N	0.27 days
1.39 m	6.2 s	96.2 °N	10.4 days
1.43 m	6.2 s	99.7 °N	6.3 days
1.03 m	4.0 s	102.9 °N	0.16 days
1.27 m	4.5 s	89.8 °N	0.04 days

0.38 m	9.5 s	124.5 ºN	0.04 days
0.41 m	5.0 s	136.1 ºN	0.04 days
0.48 m	10.6 s	146.5 ºN	0.04 days
0.59 m	3.6 s	166.3 ºN	0.04 days
0.21 m	2.1 s	184.6 ºN	0.04 days
0.47 m	9.5 s	157.9 ºN	3.0 days
0.35 m	9.5 s	168.6 ºN	6.4 days
0.32 m	8.6 s	197.6 ºN	1.2 days
0.26 m	1.4 s	283.8 ºN	0.23 days
1.99 m	13.1 s	93.2 ºN	0.08 days
1.86 m	6.9 s	96.8 ºN	3.5 days
1.85 m	6.9 s	100.2 ºN	2.6 days
0.63 m	9.5 s	159.0 ºN	0.47 days
0.38 m	2.9 s	188.0 ºN	0.08 days
2.28 m	7.7 s	97.0 ºN	2.4 days
2.25 m	6.9 s	99.4 ºN	1.1 days
1.57 m	10.6 s	97.7 ºN	0.08 days
0.71 m	10.6 s	150.8 ºN	0.16 days
2.73 m	11.8 s	95.1 ºN	0.04 days
2.71 m	7.7 s	98.2 ºN	1.1 days
2.62 m	7.7 s	100.1 ºN	0.43 days
2.26 m	10.6 s	111.2 ºN	0.04 days
3.12 m	8.6 s	97.2 ºN	0.54 days
2.78 m	7.7 s	98.4 ºN	0.04 days
0.52 m	9.5 s	151.8 ºN	0.04 days
3.57 m	8.6 s	96.1 ºN	0.54 days
3.73 m	8.6 s	97.7 ºN	0.27 days
4.12 m	9.5 s	98.2 ºN	0.19 days
4.62 m	9.5 s	97.9 ºN	0.04 days

Table 4.3 Offshore wave climate schematisation

Calibration

In order to obtain accurate predictions, the model was calibrated. In this process the model was adjusted to available information from external sources (e.g. satellite images) and some scatter in the model was filtered out.

For the calibration it was considered that within the accuracy of the satellite images no clear pattern of accretion or erosion could be concluded for the area around the ‘Rubble Tip’ as well as around La Atunara harbour. Since these features form obstructions for the net longshore transport, on the basis of the observations the best estimate of the net transport is that this is negligible. This will form the basic scenario for which the shoreline computations have been carried out.

The uncalibrated net longshore-transport pattern consisted of a small alternating north- and southward transport. Computed equilibrium orientations deviated in the order of one degree or less from the actual coastline orientation, which due to some scatter resulted in this alternating pattern of small transports. Therefore the equilibrium angles were smoothed alongshore and adjusted by 0.7 degrees to the north. With this slight calibration a more or less stable coast was simulated in the present situation (without the scheme), which corresponded with the observations. After this calibration the net longshore transport in the model varied between 0 and 1,500 m^3/yr only. Small gradients in the transport smoothed out in the initial running stage of the model, without significant shoreline changes.

The above implies that the prediction for the shoreline behaviour without the scheme (the baseline prediction) is a stable coastline (see also Section 5.3).

Sediment transport computations

Sediment transport generated by waves alone and for the combined effect of waves and tidal currents was computed. In the nearshore zone of the east Gibraltar coast (at a depth of about OD -8 m) tidal current velocities range from 0.3 m/s at spring tide to 0.1 m/s at neap tide, as illustrated in Figure 4.13. This figure is based on results of the flow study (WL | Delft Hydraulics, 2007a). Vertical and horizontal tides are asymmetric. Flow velocities are mainly northward during the 5 hours around high water, and southward the remaining time. Figure 4.13 was used to create 14 characteristic conditions for the tidal currents (and corresponding water level) at the input locations (rays in Figure 4.12). This was achieved by dividing the water level and current data into segments, for each of these segments a duration and representative water level and current was calculated. So each segment has characteristic water level and current conditions, which are presented in Table 4.4.

Waterlevel	Tidal current	Duration
-0.18 m	0.25 m/s	4.8 %
-0.05 m	0.18 m/s	8.7 %
0.08 m	0.11 m/s	7.6 %
0.18 m	0 m/s	13.4 %
0.21 m	-0.11 m/s	8.0 %
0.29 m	-0.19 m/s	4.7 %
0.34 m	-0.26 m/s	3.4 %
-0.33 m	0.26 m/s	6.3 %
-0.23 m	0.19 m/s	7.7 %
-0.16 m	0.11 m/s	7.0 %
-0.11 m	0 m/s	12.2 %
-0.01 m	-0.11 m/s	7.3 %
0.1 m	-0.18 m/s	5.9 %
0.22 m	-0.24 m/s	3.0 %

Table 4.4 Characteristic tidal conditions

The sediment transport computations showed that the effect of the tidal currents on the net transport was very small to negligible, compared to the effect of the waves. However, for all shoreline computations the transport computed with combined waves and tidal currents was applied.

An overview of the shoreline model, with some typical locations, is presented in Figure 4.14. Along the east Gibraltar coast 'equilibrium orientations'[2] were computed, which deviated in the order of one degree or less from the actual coast orientation. The 'equilibrium orientation' is the coastal orientation for which the net longshore transport is zero. On the basis of the computations it was concluded that the actual coastal orientation is very close to the equilibrium orientation and ranges between 97 and 99 degrees from Sandy Bay up to La Atunara harbour, and about 105 degrees north of La Atunara harbour (Figure 4.15).

With good data the shoreline model is capable of computing equilibrium orientations with an accuracy of some degrees. The deviation of 1 degree from the actual coastline orientation therefore indicates that the model suggests that the net transport along this coast is very close to zero.

[2] *Equilibrium orientations are calculated with Unibest on the basis of the provided input (waves, flows, sediment characteristics, beach profile etc.). In Unibest the net sediment transport is not computed for the present shoreline orientation only, but for a range of coastal orientations. On the basis of these calculations the model derives for which shoreline orientation the net transport is zero (this is the equilibrium orientation).*

5 Coastal impact

5.1 Objective

In this chapter predictions of the coastal impact are presented. The study approach for the impact predictions is discussed in Section 5.2. A baseline prediction is made of the future shoreline behaviour for the present situation, without the planned development and without the ”Both Worlds” project. This baseline prediction is presented in Section 5.3. It forms the reference for the coastal impact predictions.

In Section 5.4 the coastal impact of the planned development (Eastside, Gibraltar, see layout in Figure 5.1) is presented. In-combination effects for the planned development and the “Both Worlds” project at Sandy Bay are discussed in Section 5.5. Transboundary effects are discussed in Section 5.6. All impacts were assessed relative to the baseline prediction (described in Section 5.3).

5.2 Approach

For the impact prediction, longshore and cross-shore effects were considered separately. Alongshore re-distribution was computed with the shoreline model described in Chapter 4. Permanent offshore sand loss due to rip-currents and sand loss to the steep foreshore (and borrow pit) were estimated and included in the shoreline model as sediment sinks, in order to assess the combined effect of alongshore and cross-shore processes.

In this chapter the results of the shoreline modelling are discussed. With a shoreline model *horizontal* erosion or accretion is computed (respectively a landward or seaward displacement of the coastline).

5.3 Present situation (baseline)

As discussed in Chapter 4 the net longshore transport along Eastern Beach and Catalan Bay is negligible and the shoreline is stable, also near interruptions of the sandy beaches (e.g. short groynes). This implies that the baseline prediction is a stable coastline.

It is noted that the shoreline model prediction is based on longshore transport gradients. An additional cross-shore effect on the shoreline is caused by sea level rise, which is illustrated schematically in Figure 5.2. As a result of sea level rise a small (horizontal) erosive tendency of the coast can be expected in the range of 0.1 to 0.3 m/yr (for past rates of sea level rise). If a future scenario of sea level rise of 0.5 m/century is applied this (horizontal) erosion rate may increase to a range of 0.2 to 0.5 m/yr. Though these shoreline migration rates are small, on the longer term (decades) they may noticeably affect the overall shoreline position. This is a natural effect which is not affected by the planned development. This effect is not included in the shoreline modelling and should be superimposed on any long-term coastline change predicted with the model.

5.4 Eastside, Gibraltar (scheme) Impacts

5.4.1 Alongshore re-distribution of sand

The predicted impact of the planned development on the coastline due to alongshore sand re-distribution is presented in the Figures 5.3 to 5.5. The zero chainage along the coast in Figure 5.5 is formed by the northern boundary of the shoreline model (approximately 1.4 km north of Atunara Port). The results of the runs for the autonomous situation are marked as ‘AUT’ while the results of the runs with the planned development (longshore effects only) are marked as ‘ED’ (Eastside, Gibraltar).

The impact on Eastern Beach and Catalan Bay is considerable, while effects further away from the scheme are very limited. Accretion is predicted against the development at Eastern Beach and Catalan Bay, establishing a new ‘equilibrium’ shoreline orientation within some (approximately 1 or 2) years. Due to this effect the shielded areas between the northern and southern extensions of the development and the coastline are expected to fill up partially with sediment. The maximum computed seaward displacement of the coastline adjacent to the development is 60 to 70 meters for Eastern Beach (with respect to the initial coastline) and 20 to 30 meters for Catalan Bay. This sediment in the accreting zone originates from sections of Eastern Beach and Catalan Bay at some distance from the development, inducing an erosion of 15 to 25 meters just south of the central groyne at Eastern Beach and about 5 to 10 meters at the southern end of Catalan Bay. The morphological impact on the second section of Eastern Beach (north of the central groyne) is much smaller and consists of a maximum accretion of at most some meters near the central groyne and a shoreline retreat of similar magnitude near the northern groyne. No significant impact is predicted north of the northern groyne of Eastern Beach or south of Catalan Bay.

Note that all the above shoreline displacements should be interpreted with an accuracy margin, which in the areas of maximum erosion and accretion at Catalan Bay and Eastern Beach may be up to 10 to 15 m (in the areas with smaller displacements accuracy margins are smaller). In addition, cross-shore effects should be super-imposed. Reference is made to Sections 5.4.2 and 5.4.3.

Near the border of Spain the coastal impact is predicted to be within the natural variations of the coast and therefore not noticeable (over a period of 25 years).

Along the rocky coast south of Catalan Bay the effect of the planned development on the nearshore wave climate is insignificant, and the predicted effect on the net longshore transport negligible. No noticeable impacts on the seabed are predicted in this area (which is confirmed by Delft3D computations presented in Chapter 6). Effects of the planned development will not reach up to Sandy Bay.

For the situation with the planned development alone, predicted impacts due to alongshore sand re-distribution can be summarised as follows:

- For Catalan Bay considerable re-reorientation of the shoreline is predicted, resulting in considerable erosion of the southern part of this beach.

- For Eastern Beach a considerable re-orientation is predicted mainly for the part between the planned development and the central groyne, resulting in considerable erosion south of this groyne. Accretion is expected in the south of Eastern Beach (adjacent to the planned development).
- Between the central and the northern groyne impacts are small to moderate.
- No significant impact is predicted north of the northern groyne.
- Near the border of Spain the coastal impact is predicted to be negligible.
- No effects are predicted for Sandy Bay and negligible effects along the rocky coast between Catalan Bay and Sandy Bay.

It is emphasised that in this section only the effects of alongshore re-distribution of sand were discussed. Locally, additional erosion will occur due to cross-shore effects, as will be discussed further in the Sections 5.4.2 and 5.4.3.

5.4.2 Cross shore effects

General

In the previous section changes in the net longshore transport along the coast and resulting alongshore re-distribution of sand due to the planned development were predicted. In addition, in the area directly affected by the development there may be cross-shore effects affecting the shoreline. The most relevant issues in cross-shore direction are:

- The average profile slope. If due to locally changed conditions the shape of the average coastal profile changes significantly, this may affect the position of the shoreline and thus the beach widths adjacent to the planned development.
- Cross-shore sand loss. The planned development may induce new cross-shore transport mechanisms close to the planned development, resulting in permanent offshore sand loss,
- Beach width fluctuations. In the shielded zone the wave heights reaching the beach will change, thus beach width fluctuations may be affected. Particularly the beach width reduction during storm conditions will be considered.

Above cross-shore phenomena potentially only affect short alongshore coastal stretches close to the planned development, viz. zones of a few hundred meters in which the nearshore waves are directly affected by the planned development, so they remain limited to Catalan Bay and Eastern Beach.

Predictions of coastal slopes are generally based on empirical methods. With process-based cross-shore numerical models order of magnitudes of cross-shore sediment transport can be estimated to some extent, but due to the many (subtle) processes in cross-shore direction such models are not yet capable of an accurate reproduction of long-term average profile slopes (unless extensive data are available for detailed calibration of the model). These restrictions of cross-shore models apply even more for coastal sections near structures since in such areas additional processes such as partial wave shielding, wave reflection and forced rip currents are complicating factors.

Along the coast close to the planned development the nearshore wave climate changes to some extent, as waves from certain directions are shielded by the planned development breakwaters. Just north of the planned development waves from south-easterly directions are shielded, just south of the planned development waves from north-easterly directions are shielded. Due to directional spreading and diffraction part of the energy of these waves still enters the shielded zone, though with a lower wave height and a modified direction. The wave shielding results in a potential decrease of wave energy in the affected zone.

However, a complicating factor is that part of the incoming waves reflect against the sea defence of the planned development, thus increasing the wave energy near the planned development and might compensate part of the wave energy reduction caused by shielding. For example, the beach north of the planned development will be more strongly attacked by waves from north-easterly directions since in addition to the directly incoming waves simultaneously reflected waves can reach the beach, as illustrated in Figure 5.6. Cross-patterns of waves may occur in these areas. In this case however, the effects on morphology are expected to be very limited, because of the rounded shape of the planned development and the moderate reflection coefficient.

An additional complicating aspect is the occurrence of forced rip currents along the northern and southern extensions of the planned development. Rip currents are local offshore directed currents usually caused by non-uniform wave heights along a coast or forced by local bathymetrical features (bars and troughs). Along coasts with relatively shore-normal wave incidence - such as the considered study area - their occurrence is usually more frequent than along coasts with obliquely incoming waves. Locations immediately adjacent to the sea defence of a land reclamation are particularly sensitive for offshore directed rip currents. Any alongshore current generated by the waves (and tide) is forced in an offshore direction by the sea defence of the land reclamation. These forced rip currents may transport some sediment in an offshore direction. If transported to deeper water this sand may be permanently lost. Furthermore the planned development does not block the complete active zone (part of the coastal profile wherein sediment is transported). Sand may therefore be transported along the planned development. Some of this sand will settle at the seaward side of the planned development, resulting in some additional sand loss, which is attributed to the offshore losses due to rip currents.

The Delft3D computations indicate the occurrence of both natural rip currents along this coast (due to relatively shore-normal wave incidence) and forced rip currents along the planned development. Reference is made to Figure 5.7 (note: the grid of the numerical model is more dense than suggested by the density of vectors. Only in every 8th grid point the computed vector was presented in the plots). The figures show that particularly during almost shore-normal incidence of relatively high waves considerable offshore directed currents occur along the planned development which can transport some sand out of the active zone to the deeper region (say water depth larger than 10 m). It is noted that under these conditions (almost normal wave attack) rip currents also occur in the model along the natural coast which reaches less far offshore than the rip currents near the planned development. It should be noted that the conditions for which these strong rips occur have only a limited percentage of occurrence per year (less than 5% of the time, in both the current and the future situation). Under more oblique wave attack (and lower waves) the rip currents are not pronounced, as illustrated in Figure 5.8. (Note: the rip currents generated along the planned development are predicted not to significantly adversely affect swimming conditions at the adjacent beaches, considering the low percentage of occurrence and the relatively large wave heights for which they occur. Under the relevant conditions it can be expected that no swimming takes place. In general, swimming close to structures (also near to the planned development) should be avoided.

The above described combination of effects complicates an accurate estimate of the profile slopes and sand loss near to the planned development. In the following sections our best judgement is presented, based on a combination of quantitative estimates of some effects and more qualitative considerations related to others.

Impact on profile slope

The present coastal profiles in the study area are steep, particularly in the deeper regions (below approximately OD -10 m). The upper part of the profile, down to approximately OD -8 m can be described well with a slightly calibrated (5 to 15% steepened) standard Bruun-Dean profile for sediment with D_{50} = 0.3 mm, as illustrated in Figure 5.9.

The standard Bruun-Dean-profile (Dean,1977) is described by:

$$z = A \cdot y^{0.67}$$

with:

z = water depth
y = cross-shore distance relative to water line

A is a coefficient which is determined by the sediment size. For D_{50}= 0.3 mm (approximately the sediment in the area) A = 0.12, for the calibrated (15% steepened) profile A_{cal} = 0.14.

Below OD -8 m the actual profile deviates largely from the equilibrium sandy profile that would be expected on the basis of the Bruun-Dean profile. It is likely that at larger depths a transition from a sandy (upper part) to a rocky (lower part) profile is found.

The shape of the equilibrium cross-shore profile is a function of the wave conditions and sediment characteristics. Based on (more or less) theoretical and empirical considerations it was found that the beach slope is related to the dimensionless fall velocity parameter H/(wT) (Wiegel, 1964; Dalrymple and Thompson, 1976; Gourlay, 1980), in which H = the incoming wave height, T = the corresponding wave period and w = sediment fall velocity.
The effect of wave shielding on the profile slope was estimated on the basis of this parameter H/(wT). A lower value of the parameter H/(wT) tends to lead to steeper slopes.

The above-mentioned Bruun-Dean-profile was modified to account for a modified H/(wT) parameter in the shielded zone. However, the Bruun-Dean profile cannot account for changes in the wave height H and period T. Theoretically a reduction of the ratio H/T with a factor α should have a similar effect on the profile slope as an increase of the sediment fall velocity with a factor $1/\alpha$. On the basis of the results of the wave computations with and without the planned development (see Figures 3.13 and 3.14) in the shielded zone close to the structures the reduction of the H/(wT) parameter after construction of the planned development was estimated. Along the northern part of Catalan Bay the average (weighed over all conditions) value of H/wT is estimated to reduce with a factor 0.7 to 0.8. The effect can be simulated by applying the Bruun-Dean profile for D_{50} = 0.32 to 0.35 mm (instead of D_{50} = 0.3 mm). This results in a Bruun-Dean profile with a standard value of A = 0.13 to 0.14 (for D_{50} = 0.32 to 0.35 mm) instead of A = 0.12 (for D_{50} = 0.3 mm), in other words in a potential profile steepening due to wave shielding of about 10 to 20%.

However, the above estimate does not take into account two counteracting effects, viz. wave reflection against the southern sea defence and the potential for the development of rip currents close to the planned development. Waves from south-westerly directions reflect against the southern sea defence and locally increase the wave energy at the beach, see Figure 5.6. This effect (partly) compensates the above effect of shielding of waves from north-westerly directions. The effect is difficult to quantify. However, in some cases it was found that due to this effect the profile close to the structure becomes somewhat gentler than the exposed profile instead of steeper. Another potential effect which tends to create gentler profile slopes is the occurrence of rip currents (Figure 5.7). At the location of the planned development, (wave-induced) longshore currents in the nearshore zone may be forced in an offshore direction by the structures of the planned development.

On the basis of above considerations it is recommended not to anticipate on a significant profile steepening in the shielded zone close to the sea defence of the planned development, but to anticipate a profile slope similar to the present one.

The closure depth of the profile is not expected to change significantly. The closure depth is defined as the depth immediately seaward of the surf zone, where the wave forces can no longer produce a measurable change in bed elevation and thus in depth (van Rijn, 1998). Near the closure depth (and deeper) cross-shore interactions are weak. On the exposed beach this closure depth can be estimated on the basis of Hallermeier (1978):

$$d_{l,t} = 2.28 H_{s,t}^2 - 68.5\left(\frac{H_{s,t}^2}{gT_t^2}\right)$$

with

- $d_{l,t}$ = the predicted closure depth over t years
- $H_{s,t}$ = the non-breaking significant wave height that is exceeded 12 hours per t years
- T_t = the corresponding wave period
- g = the acceleration due to gravity

On the basis of the wave conditions presented in WL | Delft Hydraulics (2007b) the short term (years) closure depth is estimated at OD -7 to -8 m. The long term (decades) closure depth is estimated at OD -10 to -11 m. The depth of closure is related to maximum wave heights reaching the beach. It is estimated that the depth of closure will not change, as the most seaward end of the planned development extends only to OD -6m.

In summary, no significant change of the beach slopes is expected in the shielded zones around the planned development (Catalan Bay and Eastern Beach only) (for the case of the planned development alone).

Impact on offshore sand loss

Permanent cross-shore sand loss was defined here as sand transported offshore beyond the OD -10 m depth contour. This contour is located near the long-term closure depth. Sand transported offshore beyond this depth contour disappears to the relatively steep deep water foreshore from which it is expected not to be transported back onshore.

The main mechanisms for permanent offshore loss are:

- Occurrence of strong and long rip currents along the planned development during severe wave conditions.
- Shift of the toe of the sandy profile to the steep foreshore in the predicted accreting areas close to the planned development.

Offshore dredging in the borrow areas may potentially affect offshore sand loss. This is discussed separately in Chapter 6.

Offshore loss is expected to occur due to the strong rip currents developing along the planned development under certain conditions. As illustrated in Figure 5.7 strong rip currents with a significant offshore reach should be expected during periods with high waves approaching the shore almost perpendicularly. On the basis of sediment transport computations carried out with the Delft3D model (Chapter 6) the permanent sand loss due to the rip currents along the northern tip of the planned development is estimated at 100 to 1,000 m^3/yr. Along the southern tip only a small net loss due to rip currents is indicated by the model. However, we consider the occurrence of rip currents sensitive for small variations in bathymetry and conditions. Therefore, in our interpretation of the results we have assumed that sand loss due to rip currents south of the planned development will also be 100 to 1,000 m^3/yr. This loss occurs during severe wave action and though it is generated very locally along the planned development, due to alongshore sand re-distribution by the waves it can be expected to adversely affect the total sediment balance of Catalan Bay and Eastern Beach.

The deeper depth contours (beyond OD -6m) are expected to be hardly influenced by the seaward displacement of the coastline near to the planned development, due to the enclosure of the accreted areas of the beaches by the northern and southern outcrops of the scheme. Additional permanent loss due to a seaward shift of the active coastal profile (at the OD -10 m contour) (illustrated in Figure 5.10) is expected to be very limited.

On the basis of the above considerations it is concluded that the total offshore loss is determined almost completely by the rip currents, and losses are estimated at roughly 100 to 1,000 m^3/yr at each side of the planned development. It is noted that the contribution of cross-shore transport to the sediment balance generally includes considerable uncertainty, as also indicated in the considerations above. For this specific situation it can be concluded that – given the very small net longshore transport in this area – the offshore loss is a significant contributor which should be taken into account in the shoreline prediction and beach maintenance estimates. Reference is made to Section 5.4.3.

In summary, it is predicted that the planned development will cause some permanent offshore sand loss in an area close to the planned development (northern part of Catalan

Bay, southern part of Eastern Beach). This will be included in the shoreline predictions in Section 5.4.3.

Impact on profile fluctuations

During storm conditions the coastal profile reshapes to a storm profile. Sand is transported from the dry beach in an offshore direction. In most situations this sand remains in the active zone where it can be transported back to the shore during milder conditions (unless locally there are other processes which result in permanent offshore loss, as discussed above in the sub-section 'impact on offshore sand loss').

In WL | Delft Hydraulics (2007b) extreme conditions were assessed. The extreme wave conditions were assessed as being more or less normal to the coast. In the nearshore area (say up to depth contour OD -10 m) the 1/1 yr storm is predicted to approach from a direction of about 96°N, and the 1/100 yr storm from a direction of about 99 °N. It should however be realised that these directions represent a class-average and that the actual direction of wave incidence may deviate some degrees from this average. This implies that the computed extreme wave height can still reach most of the beach in the shielded zones of the development (just north of the planned development as well as just south of the planned development) almost undisturbed. Only for a very small part of the shielded zones (very close to the development) the wave conditions are expected to be slightly smaller than those in the present situation. Shoreline retreat due to profile deformation during a storm is expected to be similar for the adjacent beaches and only slightly smaller for the 'shielded' locations very close to the development. If it is assumed that for some storms the waves will mainly attack the beach north of the planned development and for some storms mainly the beach south of the planned development the main effect is that in the area close to the planned development the large shoreline retreats corresponding with severe conditions will occur less frequently.

In summary, in the shielding zones only a slight reduction of cross-shore beach fluctuations should be expected (directly adjacent to the planned development). For the major part of the beaches of Catalan Bay and Eastern Beach the effect on cross-shore fluctuations will be nil.

5.4.3 Combined alongshore and cross-shore effects

In order to include the effect of the estimated permanent offshore sand loss, sediment sinks were included in the shoreline model just north and south of the planned development. The sinks were set at a sand loss of 1,000 m^3/yr each. This is considered a conservative estimate, since north as well as south of the planned development offshore losses of 100 to 1,000 m^3/yr were estimated (see Section 5.4.2).

The results are presented in Figure 5.11 for the case with only the planned development. The run with rip currents (offshore losses) is marked as 'EDrc' (Eastside Gibraltar with rip currents) while the results of the run without the offshore sand loss (as discussed in Section 5.4.1) are marked as 'ED' (Eastside Gibraltar). Of the latter run only the state after 25 years (blue line) is visible, since after 5 and 10 years the shoreline position is equal.

For the runs with the offshore sand loss a constant overall retreat of the equilibrium-shaped beach is predicted (Figure 5.11). It can be observed that at Catalan Bay due to the modelled offshore loss the maximum erosion increases by an additional 20 m after 25 years, from approximately 5-10 m to about 25 m. A similar increase of erosion with 20 m in 25 years can be observed to the north of the planned development. It can be observed here that at Eastern Beach the adverse effect of the offshore loss is limited mainly to the area between the planned development and the central groyne. North of the northern groyne (i.e. closer to the border) no significant effects of the permanent offshore loss are expected (small deviations should be interpreted as scatter). It is noted that all presented results of shoreline changes should be interpreted within a range of several meters, particularly the effects on the shoreline caused by offshore losses.

In summary, with the planned development at the ‘Rubble Tip’ in place, the adverse effects are expected to be caused partly by re-orientation of the shoreline close to the planned development. In the first years after construction this effect is dominant. Over the longer term the effect of offshore loss becomes dominant, see Figure 5.11. South of the planned development the affected stretch is limited by the length of Catalan Bay. North of the planned development the effects are contained mainly in the area south of the central groyne, and to a small extent between the central and northern groyne. No significant impact of the planned development is predicted north of the Eastern Beach northern groyne or south of Catalan Bay.

5.4.4 Migitating measures

The impacts of the planned development can be mitigated by means of regular nourishment of the eroding spots indicated in Figure 5.11. To minimise offshore sand loss, it would be preferable to carry out relatively small-scale nourishment operations.

However, from an operational point of view and in order to avoid local rapid shoreline retreat rates due to re-orientation of the shorelines in the first years after construction of the planned development, an alternative approach would be to create the equilibrium shape of the beach in one initial sand nourishment operation immediately after construction of the planned development, as illustrated in Figure 5.12. In this way the erosion further along the beach due to the alongshore re-distribution of sand can be prevented. The total required volume of sand for this initial nourishment is about 30,000 to 60,000 m^3 north of the planned development and 20,000 to 30,000 m^3 south of the planned development. A potential disadvantage would be that the shoreline is shifted forward at an earlier stage, and as a result cross-shore losses will be relatively large. In the overall seaward shifted state of the beaches as indicated in Figure 5.12 the offshore losses should be expected to be somewhat larger, since a larger overall seaward shift of the coastal profile tends to result in a larger offshore sand loss, as discussed in Section 5.4.2. (Note: in Figure 5.12 only the most adversely affected areas were mitigated). South of the northern groyne of Eastern Beach also some slight erosion is predicted (Figure 5.3), which can be mitigated with a very small nourishment volume).

As a result of the ongoing offshore loss some regular re-nourishment will be required. It is also recommended that monitoring will be implemented and some maintenance be anticipated for the beach north of the northern groyne. The shoreline model has suggested that cross-border effects are predicted to be negligible. However, it is possible that small changes in shoreline trends could be masked by inaccuracies in the shorelines derived from the satellite pictures. Therefore, although there is no evidence to support the supposition, it is possible that a very small northwards transport (some thousands of m^3/yr at most) could be present, but which was not detected on the basis of the shoreline analysis. Even if such a northerly transport were to exist the impact of the planned development on this area would be minimal. Given the above, it is recommended to monitor the beach just south of the border after construction of the planned development. A low-maintenance solution which would provide a contingency measure to avoid any possible cross-border impact might involve the placement of a sand buffer north of the northern groyne at Eastern Beach. If a buffer were to be placed the behaviour of the buffer should be monitored and some re-nourishment (on average some thousands of m^3/year at most) should be anticipated.

Guidelines for beach nourishment

Since the beaches will be used for recreational purposes the following recommendations are given for nourishment operations.

Preferably the beaches should have their maximum width prior to the main tourist season (spring and summer). Therefore the preferred time for beach maintenance would be some time before the spring season. In that way maximum use is made of the placed sand. In addition, the wide beaches will remain for a longer period of time due to the relatively mild wave conditions during spring and summer. It is recommended that some wave action be allowed on the beach before the high season.

For recreational purposes the profile should preferably not be steeper than 1:10 above the water line. If the sand is placed under an arbitrary slope on the beach, scarps (almost vertical steps in the profile) are likely to develop. Between the waterline and OD -1.5 m the nourished profile should preferably not be steeper than 1:30. In this way an area of 40 to 50 m wide is created in which bathers can have contact with the seabed. An arbitrarily placed slope will generally be steep and it may take time before the waves have reshaped the beach to meet above criteria. On a steep beach, waves will break close to the shore. These locally breaking waves may induce local strong currents close to the shoreline and bathers can be picked up by the breaking waves. On a steep slope bathers will lose contact with the seabed within a horizontal distance of several meters. The combination of breaking waves (uncontrolled swimming) and a strong seaward increase of water depth (steep slope) should be avoided.

On the basis of the above it is recommended that the sand should preferably not just be dumped on the beach, but if possible some sand should be placed below MSL in order to speed up the process of coastal profile development towards above slopes.

Since the beaches may be used not only for recreational purposes (but for example also for protection of the hinterland) the volume of re-nourishment sand may also depend on the expected sand loss during the more severe (autumn and winter) seasons. A monitoring plan should be set-up in which criteria should be defined for re-nourishment. Criteria can best be defined in the design stage of the beaches. It may be considered to create stockpiles of dredged material at the back of the beach or at an inland location, in order to minimise the disturbance caused by dredging activities year-on-year and to enable re-nourishment to be done by land plant in some years.

5.5 In-combination effects

5.5.1 Introduction

In this study "in-combination effects" are the effects of other developments than the planned development at the 'Rubble Tip'. In this section the coastal impact of the planned development in combination with the "Both Worlds" project (planned south of Sandy Bay, see Figure 1.3) is discussed. At present, no other future developments are known.

5.5.2 Both Worlds Project

A small extension of the rocky outcrop just south of Sandy Bay is planned ("Both Worlds" Project, see Figure 1.4). The impact of this project is evaluated on the basis of expert judgement.

The extension is very small and does not create a noticeable wave shielding effect on Sandy Bay beach or any other beach. It also does not form a noticeable interruption of current and sediment transport patterns. No significant change of beach slopes due to the "Both Worlds" project is therefore expected.

The effect of this extension on Sandy Bay or on any other beach is predicted to be negligible. Predictions made for the planned development at the 'Rubble Tip' are not affected by the "Both Worlds" project. Additional effects to the ones described in the previous Section, due to alongshore sand re-distribution for the combination of the planned development and the "Both Worlds" project are expected to be negligible. As a result mitigating measures for the "in-combination effects" are similar to those discussed in Section 5.4.4.

5.6 Transboundary effects

As discussed in Sections 5.4 and 5.5, for the situation with the planned development alone as well as for the planned development in combination with the "Both Worlds" project, the coastal impact near the border of Spain is predicted to be negligible (over a period of 25 years).

Regular monitoring is recommended for the beaches north of the northern groyne of Eastern Beach. If any (temporary) adverse effects are found (although no adverse transboundary effects are expected), it could be considered (as an added contingency measure) to mitigate these effects by means of a small sand buffer north of the northern Eastern Beach groyne.

6 Small-scale morphological impacts and infill rates

6.1 Objectives

This chapter describes the aspects of the coastal morphology study that have been studied with support of the Delft3D modelling suite. The study objectives for this particular part of the coastal morphology study are:

- to predict changes in yearly transport patterns due to the planned development.
- to assess the annual infill rates at the borrow pits.
- to predict small-scale morphological changes due to the planned development (i.e. potential scour areas).

Section 6.2 of this chapter describes the study approach and the setup of the morphological model. In Sections 6.3 – 6.6, the results are presented.

6.2 Approach

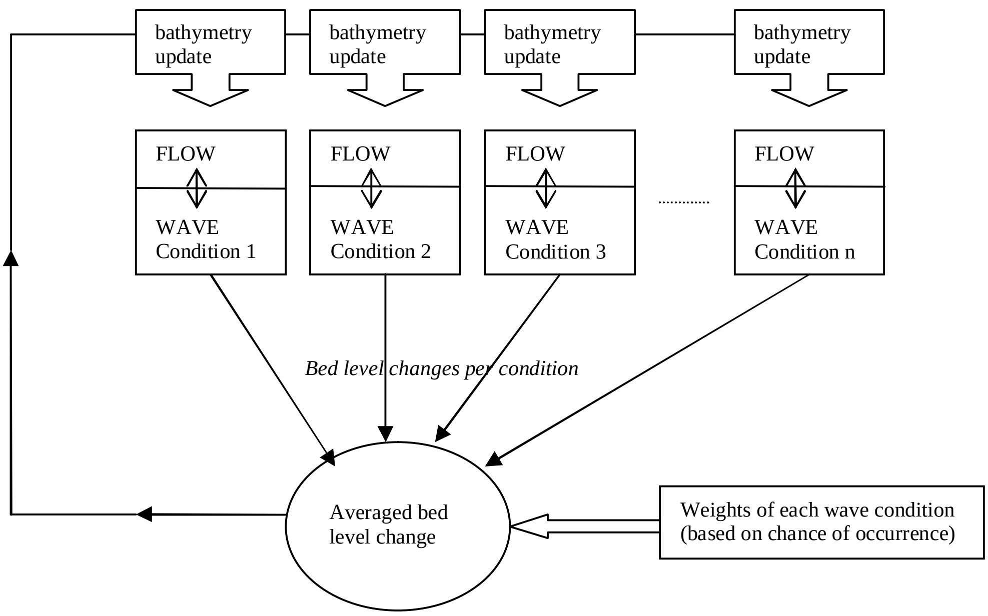
6.2.1 Introduction

This study is carried out using the Delft3D modelling suite, including Delft3D-FLOW, acting in conjunction with Delft3D-Online Wave and Delft3D-Online Morphology. To calculate wave-current interactions, the Delft3D-Online Wave enables a direct coupling with the short-waves model Delft3D-WAVE, which uses the third generation SWAN-model. The Online Morphology module calculates the sediment transport and the morphological updating within the flow simulation. In this way the morphological modelling incorporates the effect of tide and waves simultaneously.

The Delft3D-Online Wave functionality requires the setup of a flow and a wave model. Input for this study has been derived from the calibrated Delft3D-FLOW model, set up in Volume 1 (flow conditions) of the overall study, and the calibrated Delft3D-WAVE model described in Volume 2 (normal wave conditions).

The morphological model simulates one morphological year, taking into account the spring-neap tidal cycle and the yearly wave climate, derived in Volume 2 of the overall study.

For this study, the option of parallel simulations of all schematised wave conditions within Delft3D has been used. In this way, the model incorporates the impacts of all wave conditions simultaneously. This means that for each wave condition separate morphological simulations have been carried out simultaneously. During each time step a resulting bed level change was determined based on the morphological changes of each wave condition and its percentage of occurrence. The resulting updated bathymetry was used for the next computational time step for each separate simulation. This parallel process is schematized in following diagram.



In order to achieve acceptable computational times, the 66 wave conditions used for the normal wave study (volume 2) were reduced to a set of 11 conditions, representing the whole yearly wave climate. The reduction of wave conditions was carried out as follows: the effects of all waves from a given direction sector were aggregated and all those waves were then replaced with a single wave having the same longshore transport capability, the same direction and the same aggregated frequency of occurrence. In this way the number of waves was reduced from 66 to 11 (see Table 6.1).

Furthermore, one of the complications inherent in carrying out morphological projections on the basis of hydrodynamic flows is that morphological developments take place on a time scale several times longer than typical flow changes (for example, tidal flows change significantly in a period of hours, whereas the morphology of a coastline will usually take weeks, months, or years to change significantly). A well established approach to this problem is to use a “morphological time scale factor”, by which a reduction of computational time is achieved. Whereby the speed of the changes in the morphology is scaled up to a rate that it begins to have a significant impact on the hydrodynamic flows.

The implementation of the morphological time scale factor is achieved by simply multiplying the erosion and deposition fluxes from the bed to the flow and vice-versa by the factor, at each computational time-step. This allows accelerated bed-level changes to be incorporated dynamically into the hydrodynamic flow calculations. For this study, a factor of 175 was used, so that the hydrodynamic simulation period was reduced to 365 / 175 = 50.5 hrs. For the tidal conditions, a 50.5 hrs period was selected out of the entire 15 days spring-neap tidal cycle, which is representative for morphological changes over the entire spring-neap cycle (the morphological tide).

6.2.2 Model setup

6.2.2.1 Computational grids and bathymetry

Figures 6.1a and 6.1b show the computational grids used for the flow model. The original flow grid used for the flow study in this project has an average resolution of 30x30 m^2 at the borrow areas, which is sufficient for the purpose of determining infill rates. For the assessment of small-scale morphological impacts around the future planned development, the grid resolution of the original grid is inadequate. Also north of the planned development, in the direction of the harbour of La Atunara, the grid is rather coarse. Therefore, it was decided to locally refine the computational grid, using the option of domain decomposition (DD) within Delft3D. This method allows an increase of the model resolution in a limited area around the area of interest, without the need to refine the entire model. The refinement factor was 3. The resolution around the planned development is now approximately 8 x 8 m^2. Further north, the resolution decreases to 10 x 50 m^2 near the edge of the refined domain. The model bathymetry is based on the bathymetry as applied in the flow study (in Volume 1), see Figure 6.2.

The calibrated wave model used for the wave modelling tasks of the Eastside, Gibraltar study (described in Volume 2), consists of an overall grid and two nested finer grids. The boundary conditions were already defined for the overall grid during the wave modelling tasks (Volume 2), so this grid was also used for this study. Besides this overall grid a finer grid was necessary for more resolution in the areas of interest. The wave model should fully cover the domains used in the flow model, so that everywhere in the flow domain wave information was available. Since this was not the case with the nested fine wave grids used in the wave study (Volume 2) a new fine wave grid was set up. The fine grid has a resolution of 40 x 40 m^2 near the proposed development. Figure 6.3 shows this computational wave grid.

6.2.2.2 Boundary conditions

The hydrodynamics of the original calibrated flow model was forced by a 15-days spring-neap tide boundary condition. For the morphological modelling, a morphological representative tide was chosen (see Figure 6.4). The “morphological tide” is a tidal cycle that has a similar effect on the morphology as the total spring-neap tidal cycle. The schematisation technique is discussed in WL | Delft Hydraulics, 1989. The morphological tide is a tide with a high water level which is the average of all high water levels within the 15-days cycle, increased by 10%.

The 11 wave conditions used (see Section 6.2.1) are presented in Table 6.1 (for a location nearby the rubble tip at a depth of OD -8m). The probability of occurrence is also provided in the table (in days per year).

Condition	Sig. wave height [m]	Peak period [s]	Direction [degrees]	Duration [days]
1	0.33	5.9	75.0	53
2	1.50	10.9	91.4	0.39
3	1.13	6.1	96.0	23
4	2.08	7.5	96.3	5.9
5	3.36	8.6	95.7	1.4
6	3.73	9.5	96.8	0.23
7	0.91	6.4	105.2	62
8	1.29	7.0	100.8	2.6
9	0.48	7.2	110.2	87
10	1.03	6.7	110.4	15
11	0.23	9.3	162.5	93

Table 6.1: Selection of wave conditions for morphological simulation

6.2.2.3 Sediment transport formula

Several transport formulae can be used for the computation of sediment transport within Delft3D-Online-Morphology. For this study the Bijker-formula is used, which is in accordance with the UNIBEST-model (Chapter 5). The various input parameter for the Bijker-formula were also the same as used for the UNIBEST study. An important input parameter is the 90% sediment grain size diameter (D90), which was set at 0.56 mm (see Section 3.4).

6.2.3 Scenarios

The morphological model is used to simulate two scenarios. The first scenario is the present situation, without the proposed development. For the second scenario, the proposed development layout was included in the model. Furthermore, in the borrow areas the seabed was deepened by 0.9 meter within the northern borrow area and by 0.4 meter within the southern borrow area (see EBG 2007b), to assess the infill rates of these areas. Figure 6.5 shows the layout for the second scenario. It is noted that both borrow areas were included in the model at the same time, while in reality only one of the areas will be used. Since it can be assumed that the borrow areas do not affect each other this approach is justified.

6.3 Present situation (sediment transport)

For the present situation, only sediment transport results are presented. Based on the average sediment transport for each wave condition and the weights of all wave conditions, the average annual sediment transport was calculated. Figures 6.6a presents the average annual sediment transport for the present situation (Note: transport vectors have been presented in every fourth model grid point only).

As discussed in Chapter 4, for this particular area, the wave directions are mainly perpendicular to the coast, causing relatively small net longshore sediment transport rates. Perpendicular incoming waves often cause rip currents. The effect of such currents can also be noticed in the sediment transport figure as offshore directed transport. The average annual net sediment transport ranges in the nearshore zone, with water depths smaller than 8 m, from 5 - 50 m^3/m/year.

Total net longshore transport rates within the breaker zone have been estimated at most to be approximately 2,000 m^3/yr (at many locations less), which is very small. This does not mean that the instantaneous transport rates are small, but due to the fact that the average wave direction is perpendicular to the coast, the gross transport rates will result in almost zero net transport rates. With reference to the discussion in Chapter 4, the computed scattered pattern of small net transports should be interpreted as a net negligible transport along the coast.

6.4 Eastside, Gibraltar Impacts

6.4.1 Sediment transport

Similar to the baseline situation (Section 6.3), Figure 6.6b shows the average annual sediment transport for the future situation with the proposed development. In general, the overall sediment transport patterns do not change with respect to the baseline situation. Small differences can be found around the proposed development location.

Figure 6.6c shows the differences in the average annual sediment transport magnitudes. In general, the predicted differences are very small since the net longshore transport is very small in this area. As expected, at the location of the planned proposed development, the transport has decreased. Due to the proposed development, a shadow zone with lower waves and flow velocities can be expected south-west and north-west of the proposed development. This causes also a decrease in sediment transport, which follows clearly from Figure 6.6c. Increases in sediment transport are expected along the future coastline (dotted line in Figure 6.6c), because the wave breaking will take place in front of the future coastline, while in the present situation, the waves are breaking more to the west. Also the contraction of the longshore current contributes to the increase in transport offshore of the proposed development.

Further north of the proposed development, the red coloured areas show that at this location the net sediment transport has increased. This is caused by the sheltering of waves, coming from southern directions, by the proposed development, resulting in a southerly net transport.

South of the proposed development, a decrease in sediment transport is predicted. South of Catalan Bay, a very small southern net transport was calculated for the baseline situation (interpreted as a zero transport). With the proposed development in place, waves coming from the north are blocked, causing a decrease of the transport in the south, so this relative effect can be explained by the wave shielding.

6.4.2 Small-scale morphological impacts and infill rates

Figure 6.7a shows the relative morphological change (pattern after 1 year) due to the future developments (proposed development and borrow pits). In this case relative means that the morphological changes for the current situation have been subtracted from the changes that occur in the future situation. In this way, only the impacts due to the proposed development and borrow pits are visualized. From Figure 6.7a it follows that morphological impacts can only be expected in the area immediately surrounding the proposed development and the southern borrow pits. Figures 6.7b – 6.7d show the same, in more detail, for the proposed development, the northern borrow pit and the southern borrow pit.

Figure 6.7b shows in detail the areas around the proposed development and the Spanish border. Seaward of the proposed development at the toe of the sea defence, some scour is predicted, mainly caused by wave breaking and flow contraction. The vertical erosion amounts about 0.2 – 0.5 m after 1 year. A toe protection is recommended at the toe of the sea defence in order to mitigate the impacts of scour. Furthermore, the beach nourishment suggested in Section 5.4.4 already mitigates to some limited degree the possible impacts of erosion just north and south of Eastside Gibraltar. Southwest and northwest of the proposed development, small-scale sedimentation is expected. Due to wave shielding, the wave induced currents will decrease in those 'armpits' causing some sedimentation, which will amount about 0.2 – 1.0 m after 1 year. At some distance north from the proposed development at some locations small isolated spots of change (sedimentation as well erosion) are indicated by the model. These are related to very small changes in the development of rip currents for both situations (model sensitivity), and are not relevant for the overall coastal impact. The net effect of these sedimentation and erosion spots is negligible.

The Delft3D computations confirm that coastal impacts should be expected in a relatively small area close to the proposed development, as was also found with the shoreline modelling (Chapter 5). The proposed development mainly affects Eastern Beach and Catalan Bay.

To assess the infill rates of the borrow areas, the bed levels at the borrow areas for the future situation were initially deepened by 0.9 m within the northern borrow area and 0.4 m within the southern borrow area. Figures 6.7c and 6.7d show the relative morphological changes (pattern after 1 year) in and around both borrow areas. For both areas, it can be noticed that the edges have become smoother; just outside the borrow area erosion takes place, while just inside the area sediments have been deposited.

The infill rate of the northern borrow area is very limited, see Figure 6.7c. Estimated volumes of total deposited sediment in this borrow area after 1 year range from 50 – 500 m^3, which corresponds to an average (over the entire borrow area) sedimentation of 1 mm at most. However, the main deposition is expected to be close to the edges where gradients are largest. It is concluded that the infill rate is very small.
There are two explanations for this:

1. the deepening is relatively small compared to the total water depth (approximately 6%). Therefore, flow velocities are not significantly affected, transport gradients will be small and little sediment deposition will occur. This also causes the sedimentation to take place for the most part in the shallowest zone of the borrow area;
2. the northern borrow area is planned in an area with water depths > 20 m, where sediment transport is very small (see Figure 6.6b).

The southern borrow area is located in shallower water than the northern borrow area. However, the required deepening will be less than the northern borrow area (0.4 versus 0.9 m), so the relative deepening will be more or less the same for both borrow areas. Only because of a larger sediment transport rate through the shallower water, a higher infill rate is predicted (see Figure 6.7d). It is estimated that the total amount of sedimentation that takes place ranges from 1,400 – 12,000 m^3 after 1 year, corresponding to 1 – 8 mm sedimentation, which is higher than for the northern borrow area. Nevertheless, the infill rate of the southern borrow area is low.

For both borrow areas, most sedimentation is expected in the western parts of the areas, because the relative deepening is largest in the western parts and the sediment transport is larger in more shallow water.
Due to the low percentage of sand trapping by the borrow areas any adverse (erosion) effect on the surrounding seabed is predicted to be a very slow process. In addition, since the main part of these effects occur well below the closure depth, any effects of sand re-distribution around the borrow pits on the coast are expected to be very small.

6.5 In-combination impacts

The additional effects of the Both Worlds project are expected to be negligible. The extension is very small and does not create a noticeable wave shielding effect on Sandy Bay beach or any other beach. It also does not form a noticeable interruption of current and sediment transport patterns. No significant impact on small-scale morphological changes and sediment transport is therefore expected.

6.6 Transboundary impacts

The small-scale morphological impacts and the effects on sediment transport across the Spanish border are negligible small (see Figures 6.6c and 6.7b).

7 Dredging induced sediment plumes

7.1 Objectives

As part of the morphology study the impact of sediment plumes released from dredging and reclamation activities related to the construction of the project has been studied. In this chapter the approach and results of this specific task are described.

The objectives of this part of the study were:

- to assess the dispersion of fine sediment particles that have been released into the water column during the dredging and reclamation activities;
- to predict the area likely to be affected by the dispersion and deposition of sediment plumes arising from a range of source locations representing the various dredging activities.

7.2 Approach

7.2.1 Introduction

The sediment plumes released from dredging and reclamation activities have been simulated using the Delft3D flow model developed as part of this study (see Volume 1, WL | Delft Hydraulics, 2007a). The flow model has been linked with the Delft3D morphological module Delft3D-Online Morphology to include the processes of sediment dispersion and sedimentation.

Sediment dispersion is induced by three processes:

1. advection, which is the transport of material by flowing water;
2. diffusion, which accounts for the spreading of a substance in water, from an area of high concentration to an area of low concentration;
3. dispersion, which is induced by differences in current velocities in the horizontal and vertical dimension.

In this situation, diffusion plays a minor role in comparison with advection and dispersion. The latter can only be properly resolved by means of 3D modelling. Therefore, the 2D flow model developed in this study (Volume 1) has been run in 3D mode for the purpose of this task.

The combined flow and morphological model was run for various environmental and load scenarios representing several typical construction stages (sediment load conditions). For these scenarios the concentrations of suspended material in the water column and deposition were assessed taking into account the sensitive areas as indicated by the Client.

7.2.2 Sensitive areas

Several sensitive areas were indicated by the Client to be considered in this study. These areas are indicated in Figure 7.1 and can be divided into four groups:

1. 24 locations where the limpet Patella Feruginea – a very rare species protected under Annex IV(a) of the EC Habitats Directive and Gibraltar law - is found. These locations are allocated into two locations: L1 and L2 (see Figure 7.1);
2. the southern Waters of Gibraltar Marine Nature Reserve of 2600 ha (see yellow shaded area Figure 7.1);
3. a set of three outcropping rock habitats (H1, H2 and H3) that are important as ecological sites;
4. five wreck sites, allocated into two groups (W1, and W2) that are important as ecological and/or archaeological sites.

Besides the sensitive areas, possible effects across the Spanish border (dashed line in Figure 7.1) are considered.

The above indicated areas have been plotted on top of the sediment plume modelling results to indicate whether these areas are affected by the dredging activities.

7.2.3 Model set-up

7.2.3.1 Flow and wave modelling

The hydrodynamic model that was set up for the flushing modelling of the proposed development (see Volume 1) has been used as basis for this part of the study. This model has been divided into two sub-domains by means of domain decomposition. The overall domain consist of a 2DH domain, while a smaller 3D domain (with 5 equally divided vertical layers) is placed in the area around the future proposed development (see Figure 7.2).

The model is driven by the same tidal boundary conditions (14-days spring-neap cycle) as used in the flow study (Volume 1). Sensitivity computations were carried out to check the influence of wind driven currents on the plume dispersion by simulating two typical wind scenarios (similar to flow model study, Volume 1).

Besides the tidal forcing at the boundaries of the flow model, the impact of wave action has been taken into account by means of coupling with Delft3D-WAVE through the Online Wave module within Delft3D-FLOW. Considering the wave climate in this area, wave action could have a significant influence on sediment dispersion. Due to the wave-induced velocities and turbulence, the settling distances of the sediment will be larger than without waves. Furthermore, waves induce orbital velocities, causing higher bed shear stresses, especially in the shallow parts where the dredging activities are planned. Higher bed shear stresses could increase re-suspension. During this process, the actual bed shear stress exceeds the critical shear stress, causing deposited sediments to be brought in suspension.
The same wave model as used for the small scale coastal impacts and infill rates (see Chapter 6) is applied. For this study, only one wave condition is selected: condition 9 (see

Table 6.1). This is a common wave condition (~ 90 days/year) and is – with a nearshore significant wave height of 0.48 m – quite moderate. Rougher conditions are not selected, because such conditions cause high natural sediment concentrations in the water column in the breaker zone. The contribution of sediments from the dredging works is therefore less significant in these conditions.

7.2.3.2 Sediment dispersion modelling

It is assumed that gravel and coarse sand fractions will settle in the vicinity of the dredging location and will not reach the sensitive areas, because of their relatively large weights and hence large fall velocities. Therefore, this study considers only silt and fine sand, since these particles have such small fall velocities that they can be transported over significant distances (suspended sediment). For this study 5 different silt fractions (ranging from 10 µm to 63 µm) and one sand fraction (100 µm) have been be modelled. The modelled sediment fractions and their fall velocities under the modelled conditions are summarized in Table 7.1.

Sediment fraction D50 [µm]	**Fall velocity [m/day]**
10 (≤ 10)	7
20 (10 – 20)	30
30 (20 – 30)	67
38 (30 – 38)	107
63 (38 – 63)	294
100 (63 – 100)	742

Table 7.1: Modelled sediment fractions and accompanying fall velocities of particles

EBG provided limited information about the distribution of the fractions smaller than 100µm for the different dredge and dump locations. For the locations with no data, the percentages of fractions below 100µm have been assumed to be distributed linearly over the modelled fractions.

To regain the total excess sediment concentration of the modelled fractions, all above mentioned fractions are summed after the model computations.

It is noted that sediment concentrations of the plume below the background concentration are considered too low to be adequately distinguished from natural fluctuations in background concentrations and are therefore neglected. However, no information on the background suspended sediment concentrations in the area were available to the project. For the post-processing it is desirable to relate the concentrations to some kind of reference concentration. Therefore, an arbitrary reference concentration of 1 mg/l has been used in this study for post-processing purposes.

7.2.4 Model scenarios

7.2.4.1 Overview of dredging activities

As input for this study EBG provided a technical note (EBG, 2007b) with the proposed work method for the reclamation activities for the proposed development and beach nourishments at Catalan Bay. This note quantifies the spills during dredging and dumping of sediment as expected by EBG and it provides a global time schedule for the reclamation works.

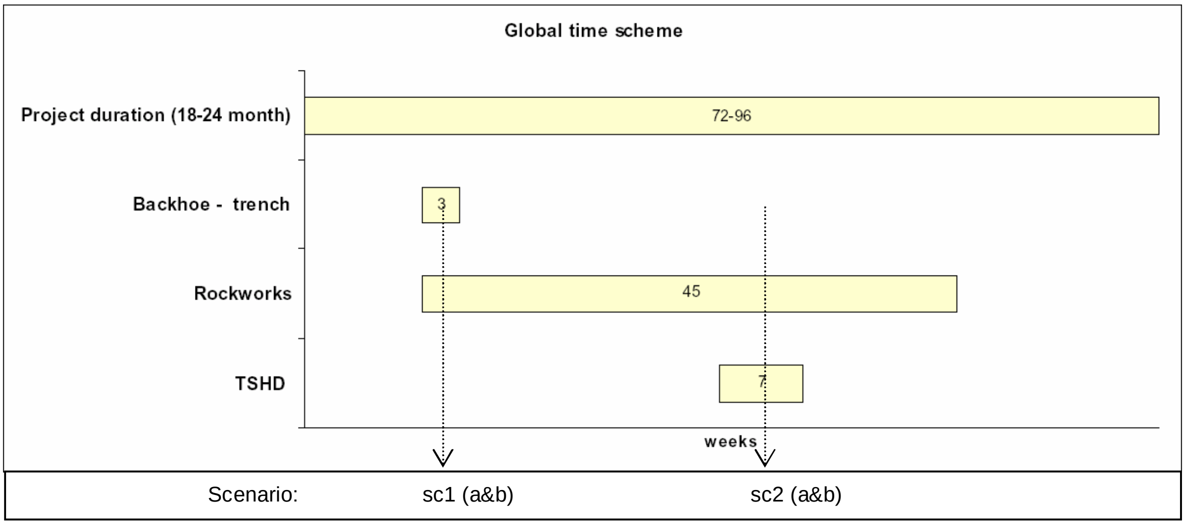
The main types of work have been specified in the technical note. Roughly three types of work can be distinguished:

1. Dredging trenches for sea defences by a Backhoe Dredger (BHD) and dumping of the dredged material by a Split Hopper Barge (SHB);
2. Dumping of rock for the sea defence, mainly by a Side Stone Dumping Vessel (SSDV);
3. Dredging of sand from the borrow areas and dumping of the dredged material by a Trailing Suction Hopper Dredger (TSHD);

A global time schedule for the works is provided below. The duration of each activity is presented in weeks in each time frame. A Backhoe Dredger (BHD) will start dredging at the beginning of the project, it will dredge the trenches for the rockworks which will take about 3 weeks. The dredged material (~35,000m3) will be disposed in the reclamation by a SHB.

At the same time the rockworks will be started, a Side Stone Dumping Vessel (SSDV) and land-based equipment will take care of this. The installation of all rockworks will take about 45 weeks.

When the sea defences are partially completed, a Trailing Suction Hopper Dredger (TSHD) will fill up the reclamation area with 600,000 m3 dredged from one of the borrow areas. The TSHD will dredge the total 600,000m3 in about 7 weeks.



Global time scheme of main works and scenario selection (see Section 7.2.4.2)

For this study, several scenarios were selected from above dredging activities. These scenarios and associated sediment spills are described below.

7.2.4.2 Scenario selection

Based on the global time schedule, four scenarios have been selected for modelling. The scenarios are selected in consultation with EBG in order to adjust the modelling scenarios with the actual construction phases of the project. Each of these scenarios represents a typical construction stage. The scenarios are indicated in the global time scheme presented above. The number in each bar chart presents the duration (in weeks) of each activity. In this section, a short description is given for each scenario including spill rates and locations. In general the presented spills are calculated with the following formula:

$$S = C \cdot s \cdot f \cdot \rho$$

with:
S: spill rate of fines (< 100 µm) [kg/s]
C: dredge/dump capacity [m^3/hr]
s: spill [%]
f: fines (< 100 µm) fraction [%]
ρ: in-situ dry density [kg/m^3]

The spills are not modelled as average continuous discharges, but as a block function to simulate the actual dredge and dump cycles, as described in EBGs technical note. Since it is not feasible to model an entire area for dredging or dumping, several points within these areas have been selected. The selection of these points is based on the most likely dredging locations indicated by EBG.

Scenario 1: construction of trench and sea defences

Scenario 1 represents the dredging activities of the BHD in combination with the SHB for the construction of the trenches. Also the SSDV activities are included in this scenario. Since sensitive areas are located north as well as south of the planned development (Eastside, Gibraltar), this scenario is divided into two sub-cases. It is assumed that the construction of the sea defence will be started in the south. The two sub-cases are: Scenario 1a for the construction of the southern part of the sea defence and Scenario 1b for the northern part.

- Scenario 1a (construction of southern part of sea defence):
  - dredge location of BHD: 289625E, 4002165N;
  - dump location of SSDV: 289625E, 4002165N;
  - dump location of SHB: 289665E, 4002215N.

- Scenario 1b (construction of northern part of sea defence):
  - dredge location of BHD: 289790E, 4002935N;
  - dump location of SSDV: 289790E, 4002935N;
  - dump location of SHB: 289665E, 4002215N.

NB: as a conservative approach, the SSDV dump location is chosen the same as the BHD dredge location, although it is expected that the SSDV will be active with the northern part of the sea defence during a later stage of the project.

The applied layout for this scenario will be the baseline layout (present situation). The spill locations that will be applied in the model are indicated in Figure 7.3a, together with the model layout for Scenario 1.

Further characteristics are:
- insitu dry density of dredged material: 1900 kg/m3;
- BHD capacity: 175 m^3/ operational hour;
- SHB capacity: 750 m^3;
- dredge / loading duration: 240 minutes within 270 minutes cycle;
- dumping duration: 5 minutes within 270 cycle;
- BHD spill: 5% (= 4.7 kg/s) during operational hours;
- SHB spill: 5% (= 222 kg/s) during dumping;
- applied distribution of dredge spill: As northern borrow area, see EBG technical note;
- fines (<100µm) in dredge spill: 10%;
- applied distribution of dump spill: 100% < 100µm, 40% < 63µm;
- SSDV dumping: 1hr within 5 hrs SSDV cycle;
- SSDV spill: 1% (= 2.8 kg/s) during operational hours;
- applied distribution of spill: 50% sand (100µm), 50% < 63µm.

The duration of Scenario 1 is 3 weeks (1.5 weeks for Scenario 1a, and 1.5 weeks for Scenario 1b).

Scenario 2: reclamation activities

Scenario 2 represents the fill of the reclamation area for the planned development (Eastside, Gibraltar), which includes the dredging of sand from one of the borrow areas and the dumping of the material at the reclamation site. During this scenario, the SSDV will still work on the sea defence. The dumping of the TSHD is also included in this study. Although it is assumed that this will take place behind the sea defence, the spills during this activity are modelled as if the spills occur outside the sea defence. The dumping locations of the SSDV and the TSHD are chosen the same.

Again, a subdivision is proposed into Scenario 2a, with dredging from the northern borrow area and Scenario 2b with dredging from the southern borrow area.

- Scenario 2a (with dredging from the northern borrow area):
  - dredge location TSHD: 290590E, 4003010N;
  - dump location TSHD: 289852E 4002583N
  - dump location SSDV: 289852E 4002583N

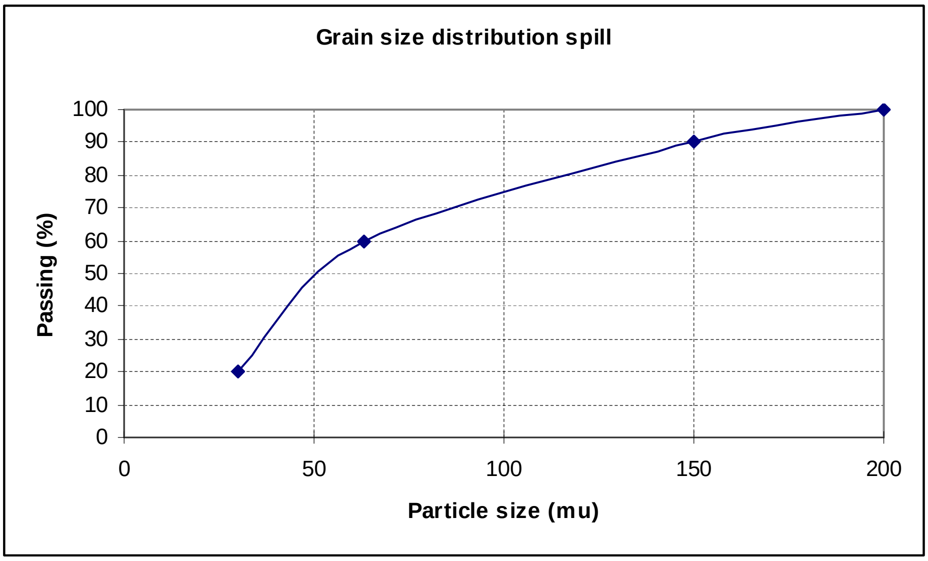
- Scenario 2b (with dredging from the southern borrow area):
  - dredge location TSHD: 289860E, 4000010N;
  - dump location TSHD: 289852E 4002583N

– dump location SSDV: 289852E 4002583N

Figure 7.3b indicates the spill locations and the model layout that will be applied. It is assumed that during this scenario, the construction of the sea defence is partially completed.

Further characteristics of this scenario are:

- in-situ dry density of dredged material: 1950 kg/m^3;
- dredge capacity: 2175 m^3 per cycle;
- dredge duration: 45 minutes within 165 minutes TSHD cycle;
- dump duration: 50 minutes within 165 minutes TSHD cycle;
- spill during dredging from northern borrow area: 10% (= 157 kg/s);
- spill during dredging from southern borrow area: 7% (= 110 kg/s);
- applied distribution of dredge spill: see Figure below;
- fines (< 100µm) in spill: 75%;
- spill during dumping: 7 kg/s;
- applied distribution of dump spill: this spill only consists of particles with grain size < 63µm. The distribution that will be applied is linear over the fractions ≤ 63µm.
- SSDV dumping: 1hr within 5 hrs SSDV cycle;
- SSDV spill: 1% (= 2.8 kg/s) during operational hours;
- applied distribution of spill: 50% sand (100µm), 50% < 63µm.



Grain size distribution TSHD spill (generated from Table in Section 3.2 EBG technical note)

The total duration of Scenario 2 is 7 weeks. Unlike Scenario 1, the total duration of Scenario 2 is applied to both sub-scenarios (2a and 2b), because only one of the sub-scenarios will be performed.

Wind scenario

First, all four scenarios have been modelled for spring and neap tide conditions including waves, without wind influences. Next, scenario 2a (worst case scenario, with the longest duration) has been reassessed with two typical wind conditions (similar to conditions applied in the flow model study, Volume 1):

- wind direction WSW and wind speed of 10 m/s
- wind direction ENE and wind speed of 10 m/s

7.3 Eastside, Gibraltar Impacts

The results of the plume dispersion modelling are presented in three types of contour plots. The first set of plots (the so-called ‘time-percentage plots’) shows isolines of the percentages of time that the excess sediment concentration due to the dredging activities is above the assumed reference concentration of 1 mg/l (=10^{-3} kg/m^3) in the bottom layer.

The second set of plots shows isolines of the maximum (averaged over turbulence timescales of seconds to minutes) sediment concentrations that are expected to occur in the bottom layer around the dredging location. These plots indicate where sediment concentrations are expected to be the highest. Note that these concentrations are the result of summing the concentrations of all fractions. Hence, these plots do not show how the grain size is distributed. This method is suitable for assessing the maximum expected sediment concentrations. In case of assessing the effects on light extinction in water, which was not the specific objective of this study, it is of importance to know the grain size distributions. A certain concentration of a 10 µm fraction has a much larger effect on light extinction than a 100 µm fraction. In the results described below, some remarks about the expected grain size distribution at the sensitive areas are added, based on the results of the separate simulations of the different fractions.

Note that the isolines presented in these two sets of plots, sometimes show discontinuities at the boundary of the two model domains. This is caused by the transition from a 2D to a 3D domain. When the sediment concentration is not fully mixed in the water column, the sediment concentrations in the bottom layer of the 3D-domain will differ slightly from the depth averaged concentration of the 2D-domain. This does not have any consequences for the results at the sensitive areas, since these are all covered by the 3D-domain.

A third set of plots shows the sediment thickness of deposited sediments originating from the dredging activities after each scenario. Note that this plot is only a snapshot in time, immediately after the completion of the considered scenario. Due to the process of re-suspension, in a later stage more sediment can be deposited at the sensitive areas.

The plots presented in this report are a selection from the total sets of figures. For each scenario, model simulations have been carried out during spring and neap tidal conditions. In this report, only the plots for the worst tidal case (spring or neap) are presented for each scenario. The entire set of figures will be delivered on a CD-ROM and can be viewed by means of the WL | Delft Hydraulics Plot Browser. In addition to the results at the bottom layer, the CD-ROM also contains figures presenting results at the surface and middle layers of the model.

Section 7.3 provides extensive descriptions of the impacts of the dredging induced sediment plumes and the accompanying figures. All presented values are summarized in Table 7.2 – Table 7.4 in Section 7.6.

7.3.1 Scenario 1: construction of trench and sea defences

Figure 7.4c shows the time-percentage plot for spring tide for Scenario 1a (construction of southern part of the sea defence). The maximum expected concentrations during spring tide are shown in Figure 7.6c. The following time-percentages (i.e. the percentage of time that the sediment concentration is above the reference concentration of 1 mg/l) and maximum expected sediment concentrations are found at the sensitive areas:

location	**time-percentage**	**maximum expected sediment concentration**
W1	70-80%	8 mg/l (distribution: ≤38 µm 100%)
W2	< 5%	1-2 mg/l (distribution: ≤38 µm 100%)
L1	90%	16-32 mg/l (distribution: ≤38 µm 75% and 25% larger fractions)
L2	80-90%	8 mg/l (distribution: ≤38 µm 90% and 10% larger fractions)

The effects are also noticeable across the Spanish border. Just across the border, the reference concentration is exceeded during 70% of the time, while the frequency decreases to 5% at about 1 km north of the boundary. The maximum expected concentrations in Spanish waters are 4-8 mg/l (distribution: ≤38 µm 100%). Within the Gibraltar Marine Nature Reserve, the reference concentration is exceeded during 80-90% of time. The expected maximum concentrations in this area are about 8-16 mg/l (distribution: ≤38 µm 100%) for this case. Only a narrow band along the coast (about 500m wide) within the Gibraltar Marine Nature Reserve is affected (excess concentrations > 1 mg/l) by the sediment plume and the plume will extend to the south tip of Gibraltar.

During neap tide, the effects at all sensitive locations are less or equal. The habitat locations are not affected by the dredging activities during Scenario 1.

The sediment deposition after Scenario 1a (2 weeks) is presented in Figure 7.8. Around the dredge and dump locations, the sedimentation is above 0.1 m. But at none of the sensitive areas, significant sedimentation is expected.

For Scenario 1b (construction of northern part of the sea defence), the findings are very similar to the results of Scenario 1a. Since some of the activities for this scenario are shifted about 500 northward, it can be roughly concluded that the expected impacts are shifted over the same distance. This means that only for the wreck site W1 and across the Spanish border higher concentrations are expected than during Scenario 1a. Figure 7.10c shows the time percentage plot for Scenario 1b during neap tide. It is shown that at W1 and across the Spanish border the reference concentration is about to be exceeded during 80-90% of time. From Figure 7.12c it follows that the maximum expected concentrations at W1 are expected to be 8 mg/l (distribution: ≤38 µm 75%, larger fractions 25%) and 4-8 mg/l (distribution: ≤38 µm 88%, larger fractions 12%) across the border.

Also the sedimentation is partially shifted northward, as can be seen in Figure 7.13. Again none of the sensitive areas is affected by significant sedimentation.

7.3.2 Scenario 2: reclamation activities

For the case with dredging from the northern borrow area (Scenario 2a), the worst case is during neap tide conditions. Figure 7.15c shows the time-percentage plot for this scenario and the maximum expected sediment concentrations during neap tide are presented in Figure 7.17c. The following values are expected at the sensitive areas:

location	time-percentage	maximum expected sediment concentration
W1	>90%	8-16 mg/l (distribution: ≤38 µm 100%)
W2	70%	32-64 mg/l (distribution: ≤38 µm 90%, and larger fractions 10%)
L1	>90%	16 mg/l (distribution: ≤38 µm 95%, and larger fractions 5%)
L2	80-90%	8 mg/l (distribution: ≤38 µm 100%)
H1	50-60%	32 mg/l (distribution: ≤38 µm 95%, and larger fractions 5%)
H2	30%	16 mg/l (distribution: ≤38 µm 100%)
H3	5%	2-4 mg/l (distribution: ≤38 µm 100%)

Within parts of the Gibraltar Nature Marine Reserve the reference concentration is exceeded during more than 90% of time. Figure 7.15c shows that the 5% -contour covers approximately 50% of the Gibraltar Marine Nature Reserve. The maximum expected sediment concentration is 16 mg/l (distribution: ≤38 µm 100%). The reference concentration across the Spanish boundary is exceeded during more than 90% of the time. The longshore length of the area with exceedance > 5% in Spain is about 1.75 km. The maximum concentrations are expected to be 64 – 128 mg/l (distribution: ≤38 µm 70%, and larger fractions 30%).

For spring tide conditions all effects are less, but the length of the 5% area in Spain has significantly increased to 3.3 km, which is near the harbour of La Atunara. It is noticed that the plot with the maximum expected sediment concentrations for this scenario during spring tide shows that the 1 mg/l contour line covers a very large area, which stretches out in northern direction more than 5 km across the Spanish border. Southward, approximately more than 60% of the Gibraltar Proposed development Nature Reserve is covered by the 1 mg/l contour, which reaches several kilometres south of the south tip of Gibraltar.

The sediment deposition after Scenario 2a (7 weeks) is presented in Figure 7.18. Due to the relative high spill rates of the TSHD, significantly more sedimentation is expected than after Scenario 1. This follows clearly from the large patch (see Figure 7.18) with an expected sedimentation above 100 mm. At locations W1, L1, L2, H2 and H3 the sedimentation will be limited and is expected not to exceed 5 mm. At habitat location H1 the expected sedimentation just after Scenario 2a amounts approximately 10 mm, while at W2 the maximum sedimentation is expected to be in the order of 50 mm. The maximum transboundary sedimentation amounts about 100 mm at a water depth of about 15 m. Within the area of the Gibraltar Proposed development Nature Reserve, it is expected that the sedimentation will not exceed 10 mm.

For the case with dredging at the southern borrow area (Scenario 2b) and neap tide conditions, the following time percentages (Figure 7.20c) and maximum sediment concentrations are expected (Figure 7.22c):

location	time-percentage	maximum expected sediment concentration
W1	>90%	8 mg/l (distribution: ≤38 µm 90%, larger fractions 10%)
W2	5-30%	2-4 mg/l (distribution: ≤38 µm 100%)
L1	>90%	8-16 mg/l (distribution: ≤38 µm 90%, larger fractions 10%)
L2	>90%	16 mg/l (distribution: ≤38 µm 100%)
H1	5%	2 mg/l (distribution: ≤38 µm 100%)
H2	5%	2 mg/l (distribution: ≤38 µm 100%)
H3	<5%	2 mg/l (distribution: ≤38 µm 100%)

Across the Spanish border, the reference concentration is exceeded during 80-90% of time and the affected area is much smaller than for Scenario 2a. The maximum expected concentrations in Spanish waters is 4-8 mg/l (distribution: ≤38 µm 100%). Since the dredging location is situated within the Gibraltar Marine Nature Reserve, the reference concentration is exceeded during more than 90% of time in this area. The expected maximum concentrations in the Gibraltar Marine Nature Reserve are high, up to 512 mg/l and very locally, in the vicinity of the spill location, the maximum concentration is higher than 512 mg/l. Approximately 80% of the Gibraltar Marine Nature Reserve is affected by the sediment plume (with excess concentrations above 1 mg/l) and the plume will extend far around the south tip of Gibraltar.

For the situation during spring tide, all effects are less or equal except for locations L2, W2 and Spain. The maximum expected concentration during spring tide at W2 and in Spain is 8 mg/l and at location L2 it is 16 mg/l. During 20-30% of time, the reference concentration is expected to be exceeded at location W2.

The sediment deposition after Scenario 2b (7 weeks) is presented in Figure 7.23. Similar patterns can be observed as for Scenario 2a, except for the large patch of sedimentation around the TSHD dredge spill location, since dredging took place from the southern borrow area. This causes the largest sedimentation to take place within the area of the Gibraltar Marine Nature Reserve (above 100 mm, water depth > 10 m). Again, it will mainly takes place within the borrow area. At the habitat locations, negligible sedimentation is expected. At the wreck sites and near the limpet locations, the sediment will be limited to 5 mm. Across the Spanish border very little sediment deposition is expected due to this scenario.

Scenario 2a has also been modelled with two different wind conditions. The results of this assessment are discussed here in short. The impact of wind on the sediment plumes that will be presented here for Scenario 2a, can be used for the other scenarios to make a rough prediction of wind influences in those other cases.
The figures presenting the results of these wind simulations are not included to this report, but are added to the figures on the CD-ROM.

Besides the no-wind simulations that have been carried out for Scenario 2a, sensitivity runs have been done to determine the effect of wind on the sediment dispersion. In general, the wind from WSW moves the plume dispersion to the north and closer to shore and the wind from ENE moves the plume dispersion more to the south.

With the WSW-wind condition, the plume (with maximum expected excess concentration > 1 mg/l) covers a much wider area across the Spanish border than for the situation without wind. In an area which extends several kilometres north of the harbour of La Atunara, the reference concentration is exceeded during more than 80% of time while with no-wind conditions, this area crosses the Spanish border only 1 km. Although the effects on the maximum expected sediment concentrations at the other sensitive areas are limited, the sediment deposition is clearly affected by the wind influence, resulting in a stretching and a northwards shift of the patch in which sedimentation is expected.

Concerning the ENE-wind condition, little effect is expected on the maximum sediment concentrations at the sensitive areas. As for the case with the WSW-wind conditions, the extent of the plume (with maximum expected excess concentration > 1 mg/l) changes. For this case it covers a larger part of the Gibraltar Marine Nature Reserve but the changes are less than for the WSW-wind case. The sediment deposition is expected more to the south with the ENE-wind condition, resulting in a higher sedimentation within the Gibraltar Marine Nature Reserve.

It is noted that the considered wind scenarios are on the conservative side considering that dredging and other construction activities may already be stopped if these wind conditions occur.

7.4 In-combination impacts

The additional effects of the Both Worlds project are expected to be negligible. The extension is very small and does not create a noticeable change in wave or current behaviour near the dredge and dump locations. No effects of the Both Worlds project are therefore expected on the dispersion of dredge induced sediment plumes.

7.5 Transboundary effects

In previous sections, a complete description of the effects for each scenario is provided, including the transboundary effects. In this section, a summary of the transboundary effects is given for each scenario. In general, the transboundary effects are more intense during spring tide conditions than during neap tide, i.e. the affected transboundary area is larger during spring tide conditions and the maximum expected concentration in Spanish waters is higher for all scenarios.

Scenario 1

Most transboundary effects during Scenario 1 are expected during Scenario 1b. Just across the border, the reference concentration is exceeded during more than 80% of the time (during neap tide), while the frequency decreases to 5% at about 1.5 km north of the boundary (during spring tide).

The maximum expected concentrations in Spanish waters are 4-8 mg/l.

For Scenario 1, negligible sedimentation is expected across the Spanish border.

Scenario 2

For Scenario 2, case a (northern borrow area) produces the most adverse impacts across the Spanish border. During both spring and neap tide, the reference concentration is exceeded during more than 90% of time in the Spanish territory. The maximum concentrations are expected to be in the order of 64 – 128 mg/l. The affected area in terms of excess concentrations above 1 mg/l extends 5 km northward from the border (spring conditions).

The expected sedimentation just after Scenario 2 can be up to 100 mm just across the border, at water depths around 15 m.

For Scenario 2b, the transboundary effects are less.

7.6 Discussion

The results of the modelling study are valid given the applied assumptions and conditions. It should be noted, however, that when there is a (significant) change in these assumptions (different dredging method, different dredging and dumping locations, capacity, sediment gradation or environmental conditions), the results may change. For instance, if the percentage of fines appears to be larger, plume concentrations and sediment deposition at the sensitive areas might be larger. When relatively small differences in the fraction of fines are expected, linear scaling of the results is possible.

As discussed in the introduction of Section 7.3, the results that have been presented concerning the sediment deposition, correspond with the situation right at the end of each scenario. This means that the presented patterns of sediment deposition are only a snapshot and these patterns may change in time due to resuspension caused by waves and currents, especially during the extreme conditions. Due to resuspension effects, it is expected that the area with deposited sediments will increase while the deposition thickness will decrease.

The objective of this study was to assess the mid-field dispersion of the sediment plume during dredging activities and to determine if sediment particles could reach sensitive areas. The approach in this study has been adapted to this objective (model resolution, environmental conditions, assumptions about dredging method, etc.). The results from this study are therefore not valid for near-field assessments (within 100 – 200 m of the dredging operation).

7.7 Conclusions

On the basis of the provided plume modelling results for the considered scenarios it is concluded that:

- from the modelling results it can be concluded that the least effects are expected during Scenario 1 compared to the other scenario;
- most sedimentation is expected after Scenario 2, due to the relative long duration and high spill rates that occur during this scenario;
- the scenarios which included wind influences show that the dredge plume extents can change significantly due to wind effects;

- on the basis of interpretation of the modelling results it is expected that sedimentation rates in all above mentioned sensitive areas will be in the order of mm's – cm's only. Only for the scenarios with a TSHD dredging from the northern borrow area, sediment deposition may be higher (several cm's – 0.1 m) at location W2 and across the Spanish border, with a water depth of approximately 15 m. With a TSHD dredging from the southern borrow area, the expected sedimentation within the Gibraltar Marine Nature Reserve will be above 0.1 m;
- Tables 7.2, 7.3 and 7.4 summarize for each scenario respectively the maximum sediment concentrations, the percentages of time of exceeding the reference concentration and the sediment deposition that can be expected at the sensitive areas.

	Sc 1a	Sc 1b	Sc 2a	Sc 2b
W1	8	16-32	8-16	8
W2	1-2	1-2	32-64	8
L1	16-32	8	16	16
L2	8	4-8	8	16
H1	< 1	< 1	32	2
H2	< 1	< 1	16	2
H3	< 1	< 1	2-4	2
Spain	4-8	4-8	64-128	8
Reserve	8-16	8	16	> 512

Table 7.2: Maximum expected sediment concentrations [mg/l] at sensitive areas for all scenarios. NB: the presented values for Spain and the Nature Reserve are maximum values in these areas which may occur very locally

	Sc 1a	Sc 1b	Sc 2a	Sc 2b
W1	70-80%	90%	>90%	>90%
W2	5%	<5%	70%	20-30%
L1	90%	90%	>90%	>90%
L2	80-90%	70-80%	80-90%	>90%
H1	<5%	<5%	50-60%	5%
H2	<5%	<5%	30%	5%
H3	<5%	<5%	5%	<5%
Spain	70%	80-90%	>90%	80-90%
Reserve	80-90%	80-90%	90%	> 90%

Table 7.3: Percentages of time that the excess sediment concentration is above reference concentration for all scenarios. NB: the presented values for Spain and the Nature Reserve are maximum values in these areas which may occur very locally

	Sc 1a	Sc 1b	Sc 2a	Sc 2b
W1	<2	<2	2-5	<2
W2	<2	<2	10-50	<2
L1	<2	<2	2-5	5-10
L2	<2	<2	<2	5
H1	<2	<2	10-20	<2
H2	<2	<2	2-5	<2
H3	<2	<2	<2	<2
Spain	<2	<2	90	<2
Reserve	<2	<2	5-10	> 100

Table 7.4: Expected sediment deposition [mm] for all scenarios. NB: the presented values for Spain and the Nature Reserve are maximum values in these areas which may occur very locally

8 Conclusions and recommendations

In the study reported here the following coastal morphology issues related to the Environmental Impact Assessment have been considered:

- Coastal impact (alongshore and cross-shore), including recommendations for beach maintenance works (nourishments).
- Infill rates of borrow pits, proposed development basin and dredged areas (e.g. navigation channel).
- Area affected by dredging induced sediment plumes.

In the following the main conclusions and recommendations for each issue are presented.

Coastal impact

The impact on the adjacent beaches has been predicted. Special attention has been given to cross-border erosion (erosion occurring in Spanish territory).

Shoreline analysis (satellite pictures) and numerical sediment transport modelling both indicate that the net longshore sand transport in the study area is negligible.

The predicted impact of the planned development on the adjacent shoreline is presented in Figure 5.11. Due to a combination of reorientation of the shorelines and offshore sand loss in these areas, a maximum coastline displacement north of the planned development (Eastern Beach) of 60 to 70 meter in seaward direction should be expected within the first years after construction, while for Catalan Bay a maximum seaward displacement of the coastline of 20 to 30 meter is expected during this period. Consequently erosion in the order of 15 to 25 meter is expected south of the central groyne at Eastern Beach and 5 to 10 meter erosion at the south of Catalan Bay. Over the longer term this erosion is expected to increase an additional 20 meter after 25 years, due to offshore losses caused by the planned development. The morphological impact on the second section of Eastern Beach (north of the central groyne) is much smaller and consists of a maximum accretion of at most some meters near the central groyne and a shoreline retreat of similar magnitude near the northern groyne. No significant impact is predicted north of the northern groyne of Eastern Beach or south of Catalan Bay.

The long term influence of the proposed development on the coast across the border with Spain is predicted to be negligible. This prediction is based on our best insight into the coastal system. However, since some inaccuracy in the prediction may lead to small deviations of predicted shoreline migration trends, it is recommended to monitor the beach just south of the border after construction of the planned development. If any (temporary) adverse effects are found (although no adverse transboundary effects due to the planned development are expected), it could be considered (as an added contingency measure) to mitigate these effects by means of a small sand buffer north of the northern Eastern Beach groyne.

The predicted local erosion at parts of Catalan Bay and Eastern Beach can be mitigated by regular re-nourishment with sand. Consideration could also be given to forming the equilibrium shape of the beach in one initial sand nourishment operation immediately after construction of the proposed development, as indicated in Figure 5.12. The required volume of sand to do this is about 30,000 to 60,000 m^3 north of the planned development and 20,000 to 30,000 m^3 south of the planned development.

The assessment of in-combination effects (with the "Both Worlds" project) shows cross-border effects are predicted to be negligible.

The planned development, both with and without in-combination effects is not predicted to adversely affect the shoreline of Sandy Bay.

Infill rates dredged areas and small scale impacts

The small scale impacts will only occur in the vicinity of the proposed development. Offshore of it some scour is predicted, caused by wave breaking and flow contraction. In the armpits of the development, on the northwest and southwest side, sedimentation may occur.

The infill rate of the northern borrow area is very limited. Estimated volumes of total deposited sediment in this borrow area after 1 year range from 50 – 500 m^3, which corresponds to an average (over the entire borrow pit area) sedimentation of 1 mm at most. However, the main deposition is expected to be close to the edges where gradients are largest.

The infill rates in the southern borrow area are also very low. It is estimated that the total amount of sedimentation ranges from 1,400 – 12,000 m^3 after 1 year (corresponding to 1 –8 mm sedimentation).

Due to the low percentage of sand trapping by the borrow areas any adverse (erosion) effect on the surrounding seabed is predicted to be a very slow process. In addition, since the main part of these effects occur well below the closure depth, any effects on the coast of sand re-distribution around the borrow pits are expected to be very small.

Areas affected by dredging induced sediment plumes

On the basis of the sediment plume modelling results it is concluded that maximum suspended sediment concentrations of more than 512 mg/l[3] are expected within the Gibraltar Marine Nature Reserve, when the southern borrow area is used. If dredging takes place from the northern borrow area, maximum sediment concentrations, of about 64 – 128 mg/l, are expected across the Spanish border.

[3] *In this report at several locations it is indicated that concentrations exceed 512 mg/l. This value is exceeded very locally around spill locations. Further quantification of such high local concentrations would require a different type of modelling than the applied midfield modelling.*

From the modelling results it can also be concluded that least effects are expected during Scenario 1 (construction of trench and sea defences, see Figure 7.3a) compared to the other scenarios.

Sedimentation rates in the considered sensitive areas are estimated to be in the order of mm's – cm's only. Only for the scenarios with TSHD dredging, sediment deposition may be higher; during dredging at the northern borrow area, sediment deposition at wreck location W2 (see Figure 7.1) and across the Spanish border may be up to 0.05 m at water depths of 15 m. During dredging within the southern borrow area, the total volume of the sedimentation within the Gibraltar Marine Nature Reserve after 7 weeks of dredging is expected to be approximately 100,000 m^3, of which 80,000 m^3 is expected in an area of 1,000,000 m^2 (average sedimentation of approximately 0.1 m). Most of this sedimentation will take place within the southern borrow area, which will have already been disturbed by the deepening.

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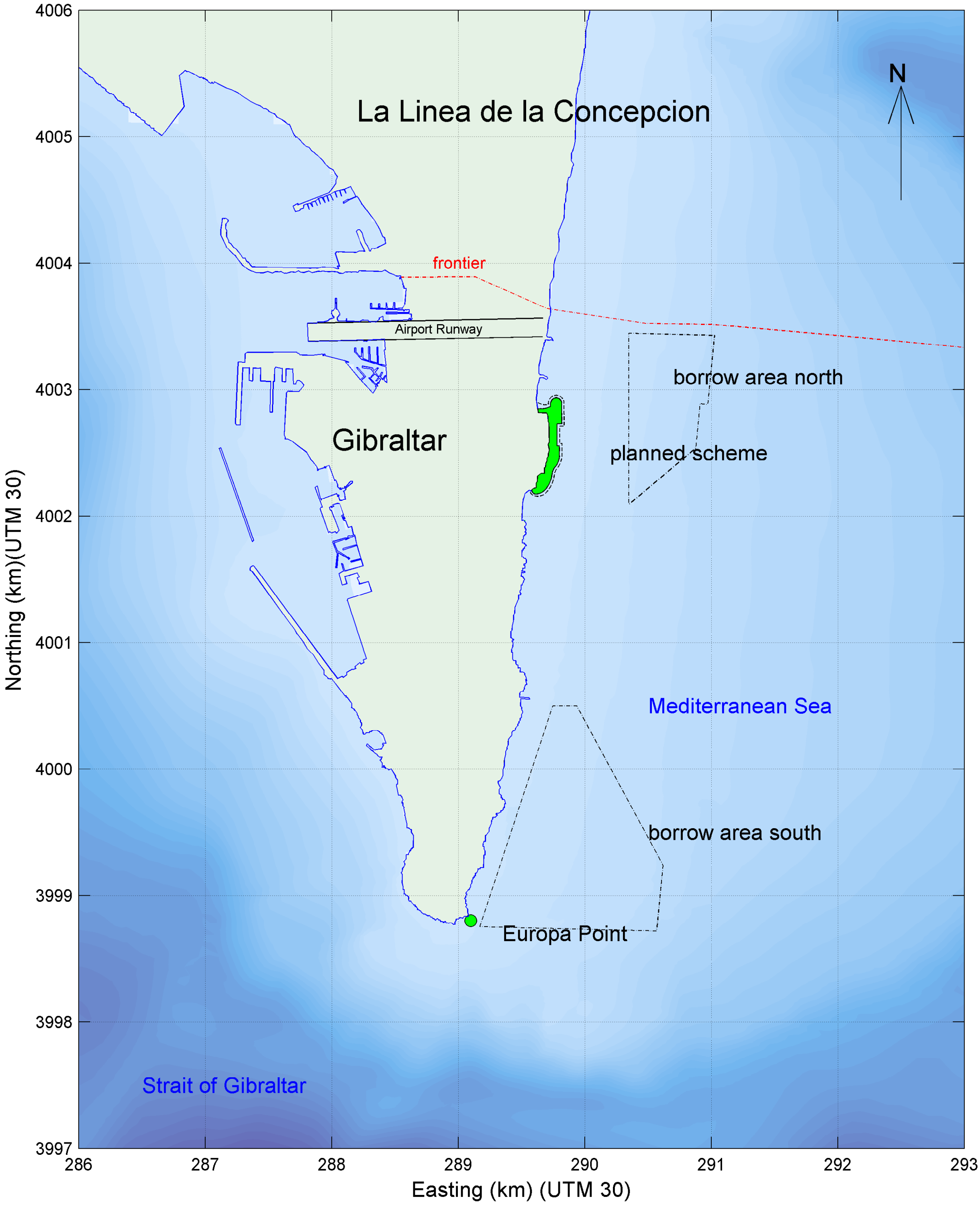
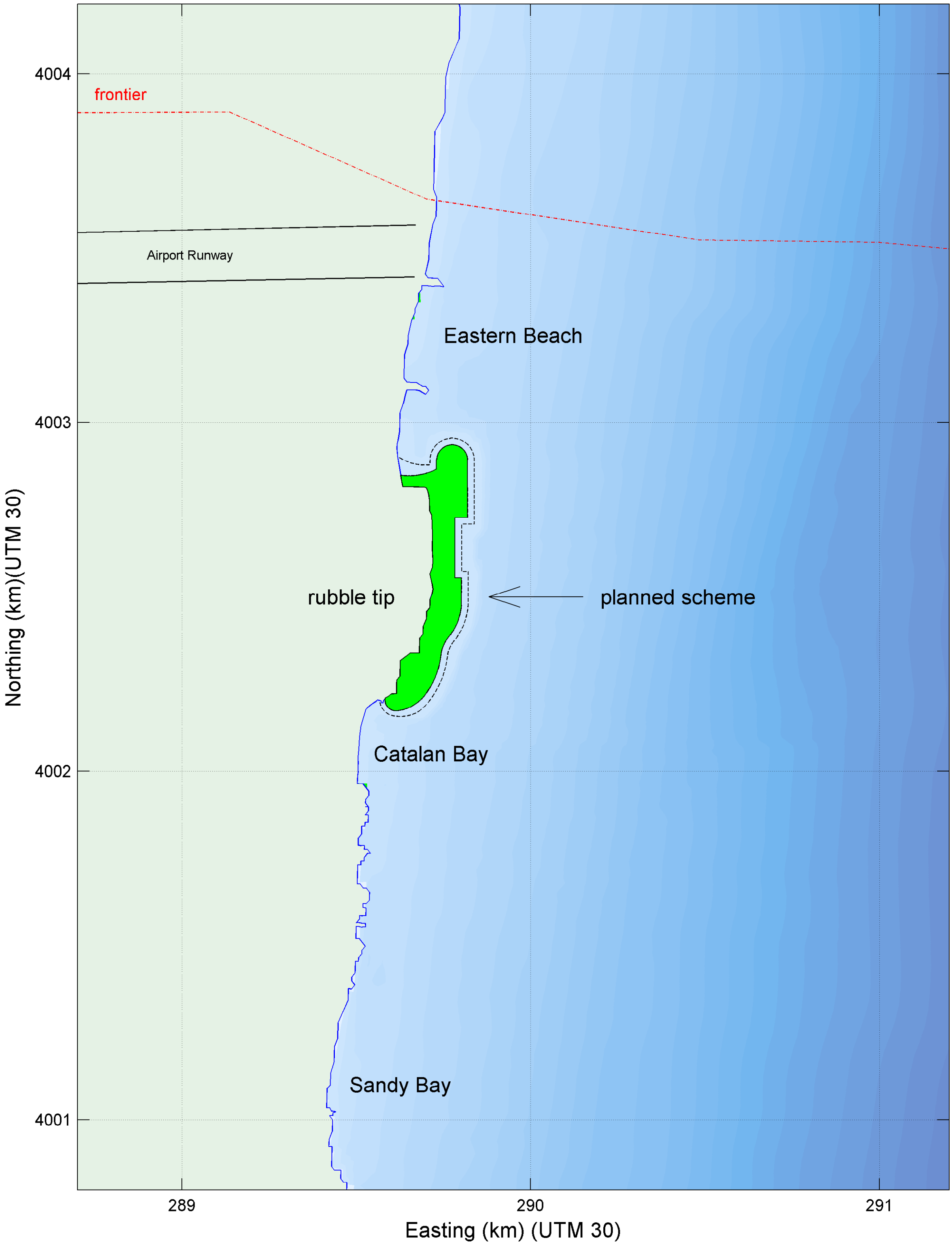
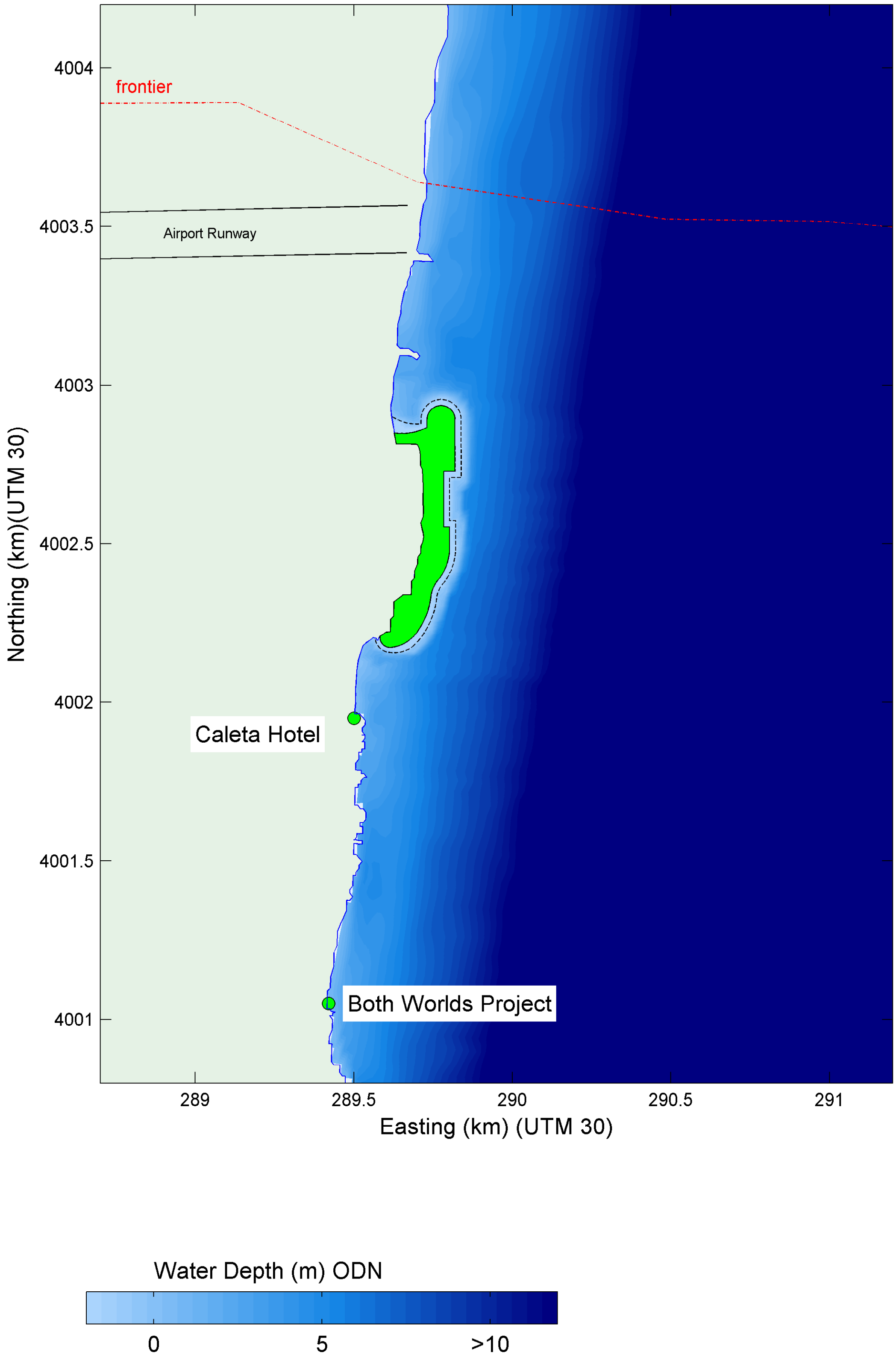
Figures



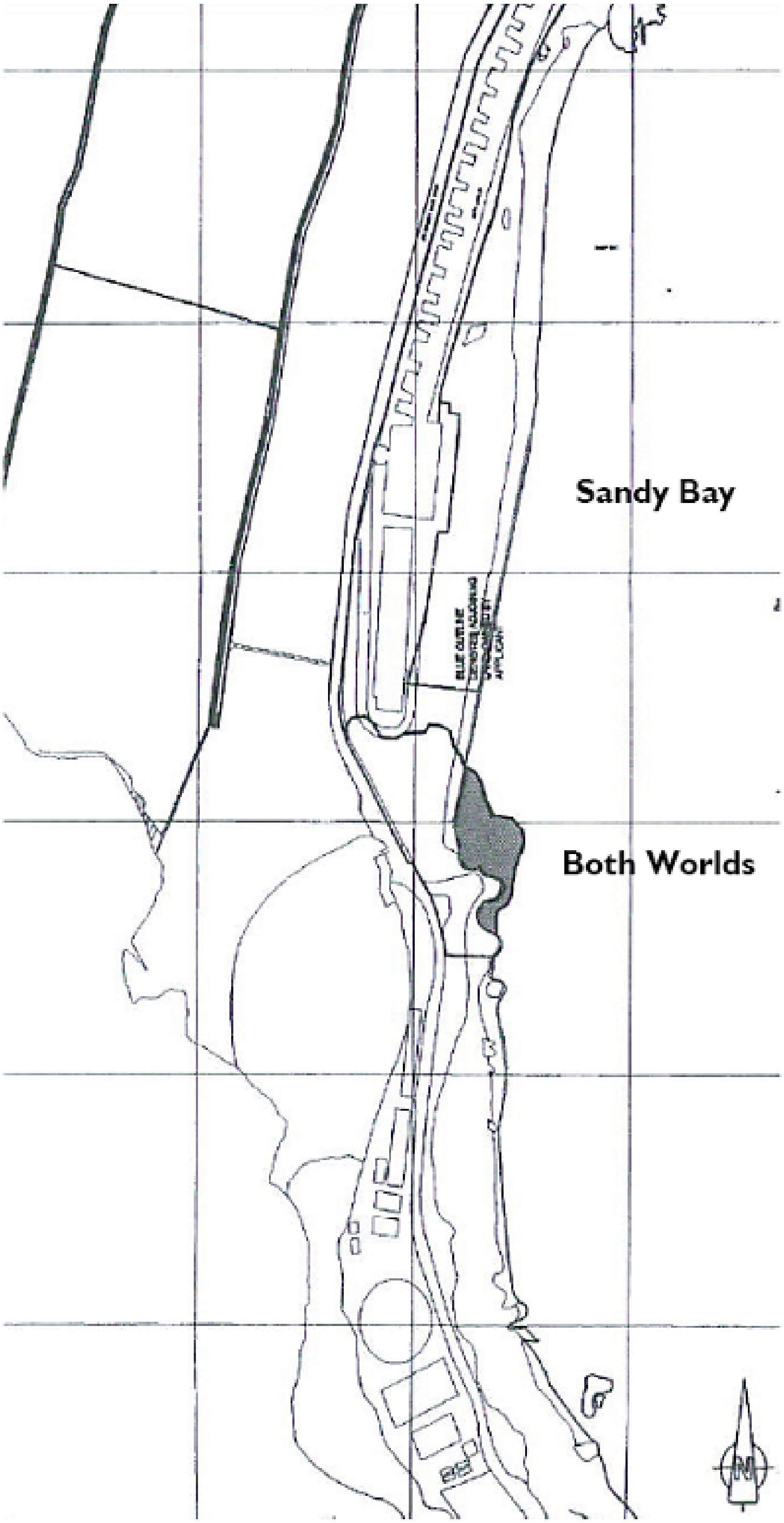
Eastside, Gibraltar Project location		
WL | DELFT HYDRAULICS	H4725	Fig. 1.1



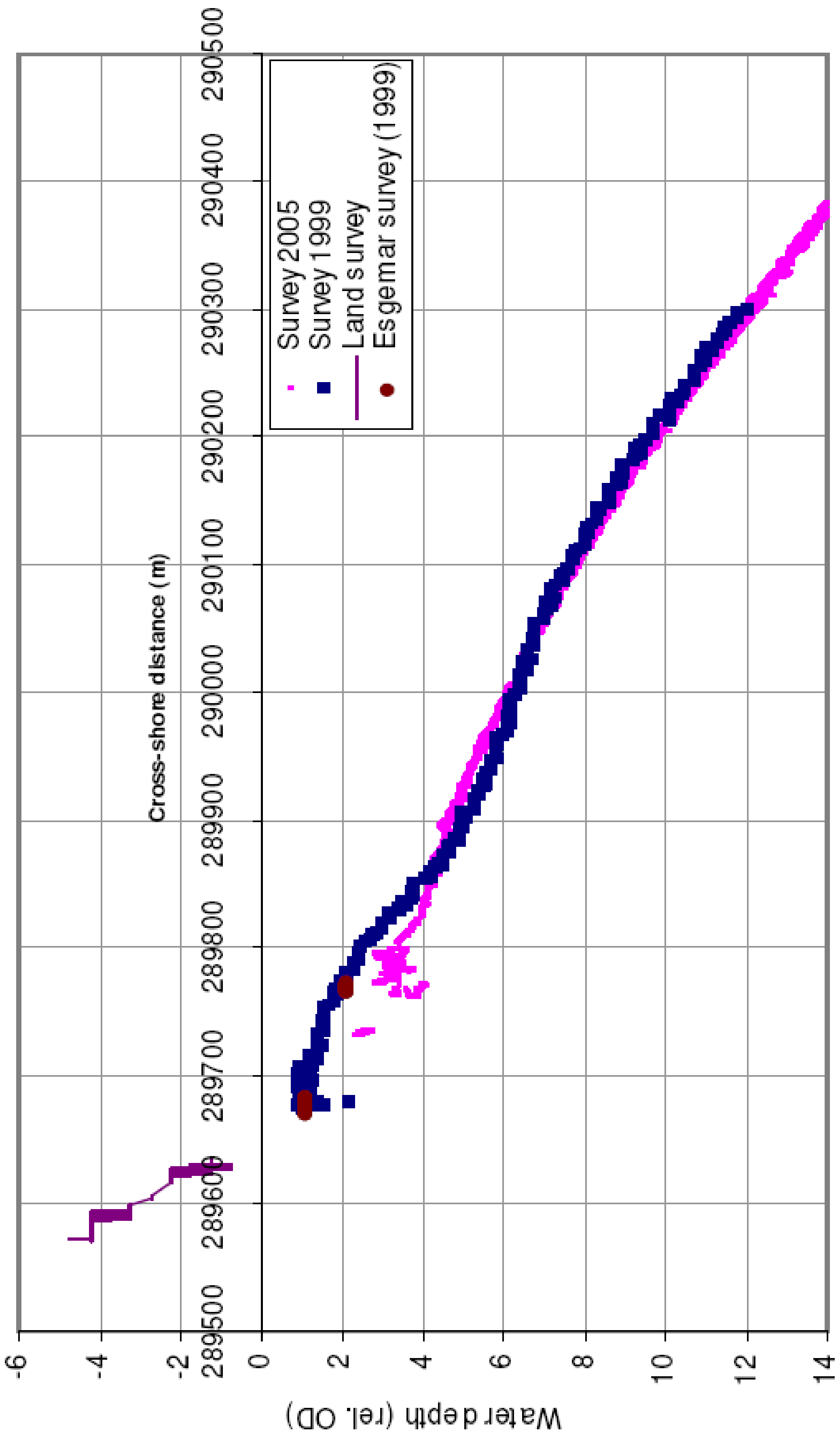
Eastside, Gibraltar Project site : future situation with scheme		
WL | DELFT HYDRAULICS	H4725	Fig. 1.2



Development scheme in combination with Both Worlds Project		Delft3D
WL | DELFT HYDRAULICS	H4725	Fig. 1.3

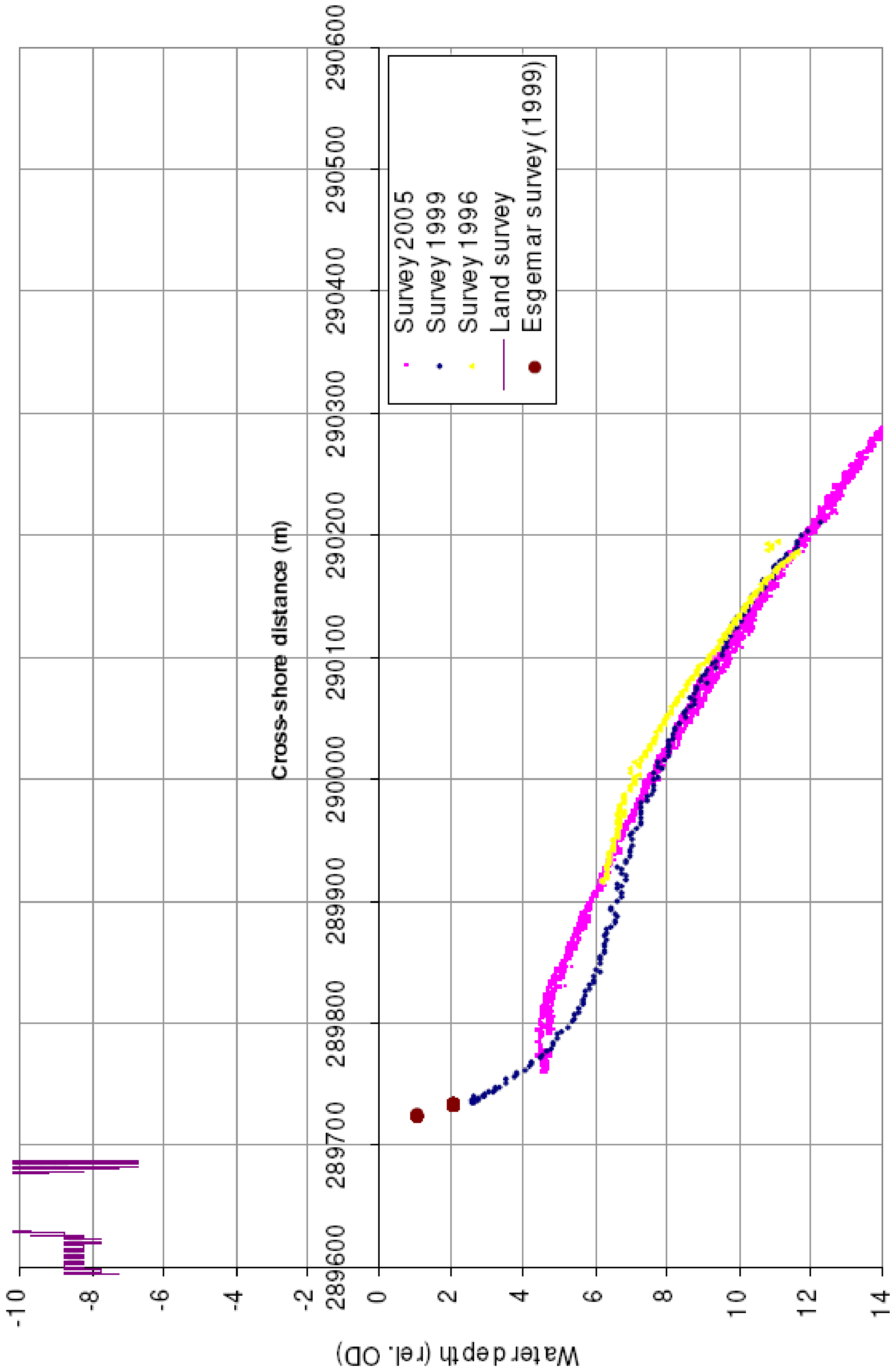


"Both Worlds" project		
WL | DELFT HYDRAULICS	H4725	Fig. 1.4



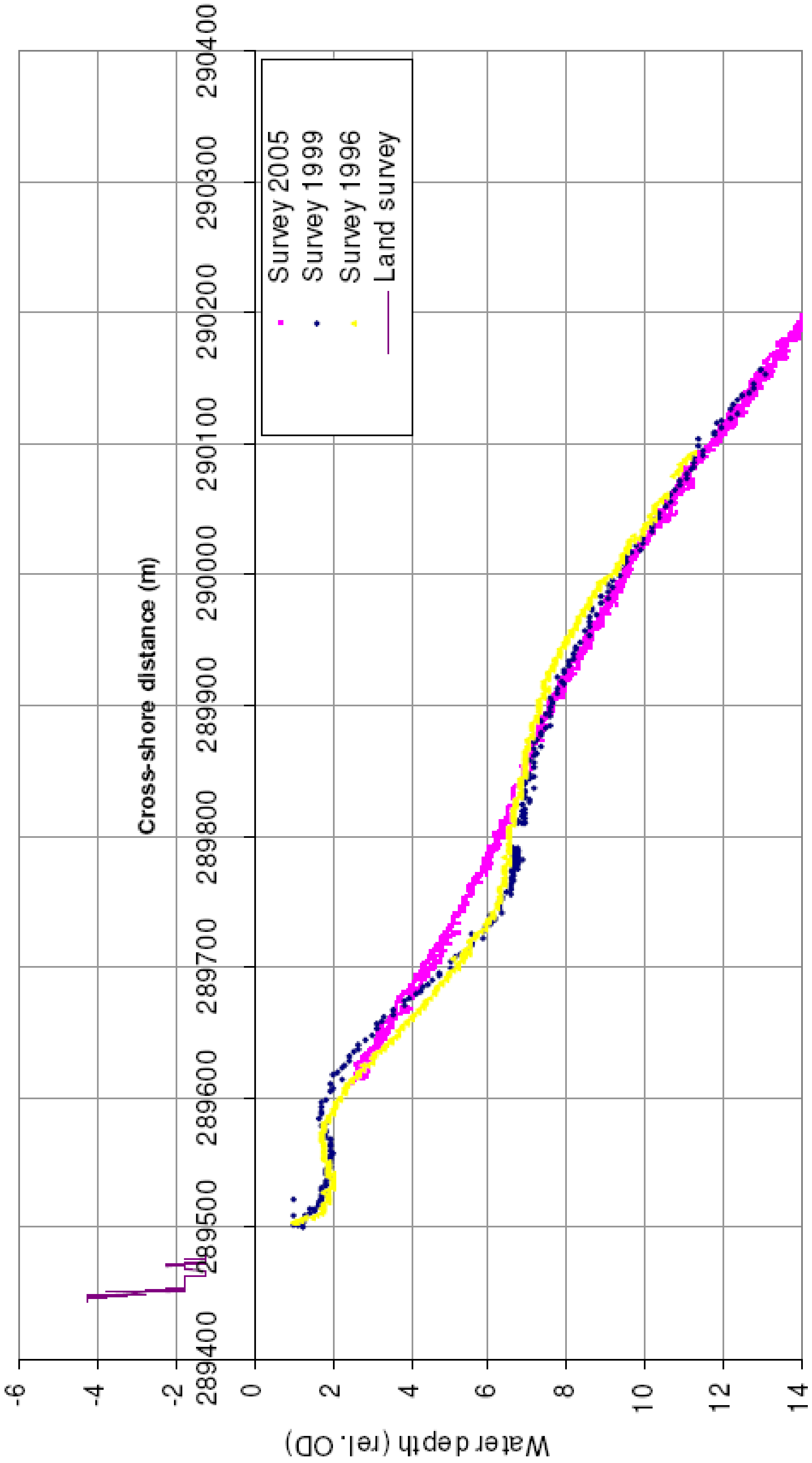
Profile surveys at Eastern Beach
(development over time)

WL | DELFT HYDRAULICS | H4725 | Fig. 3.1



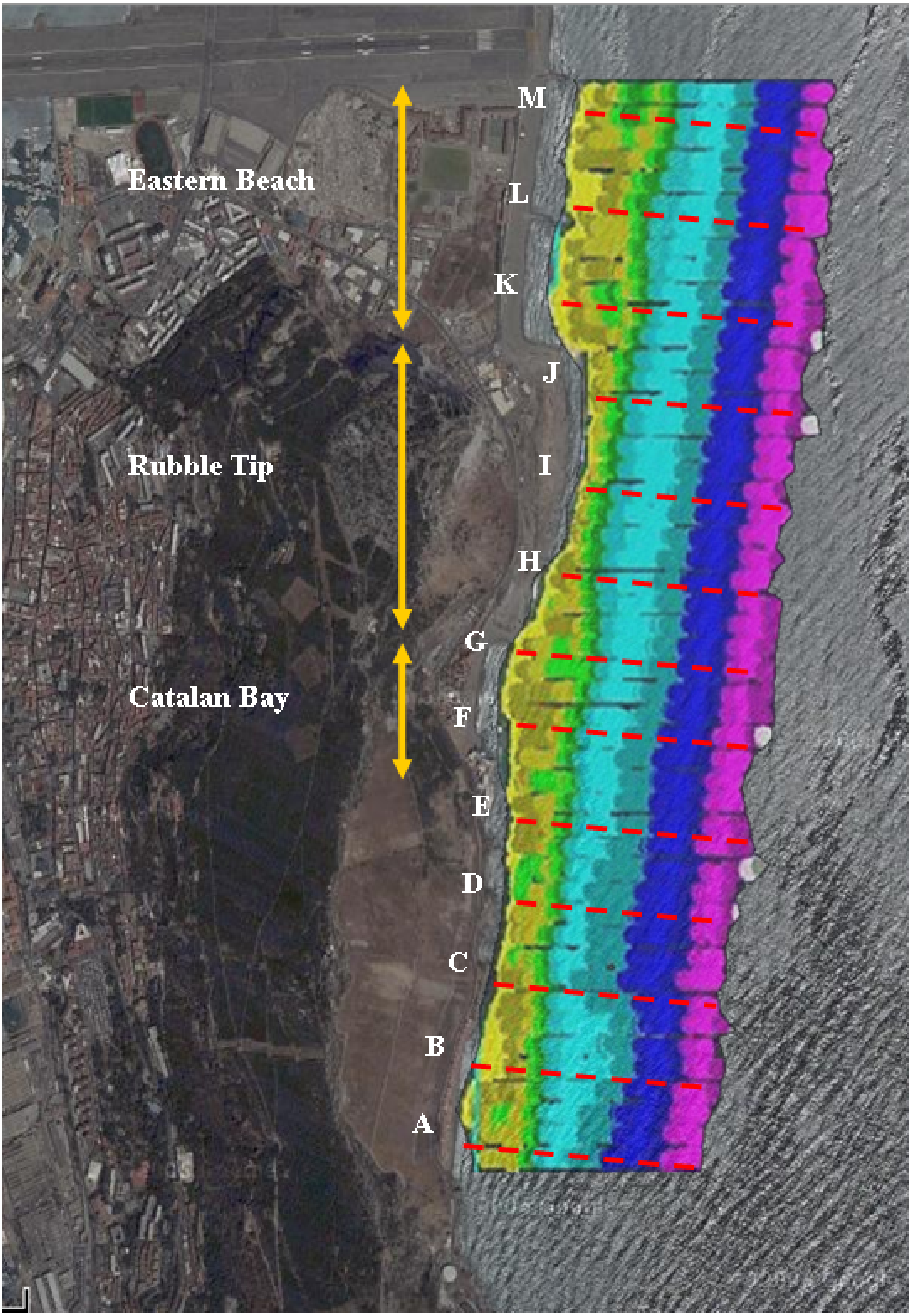
Profile surveys at the Rubble Tip
(development over time)

WL | DELFT HYDRAULICS	H4725	Fig. 3.2



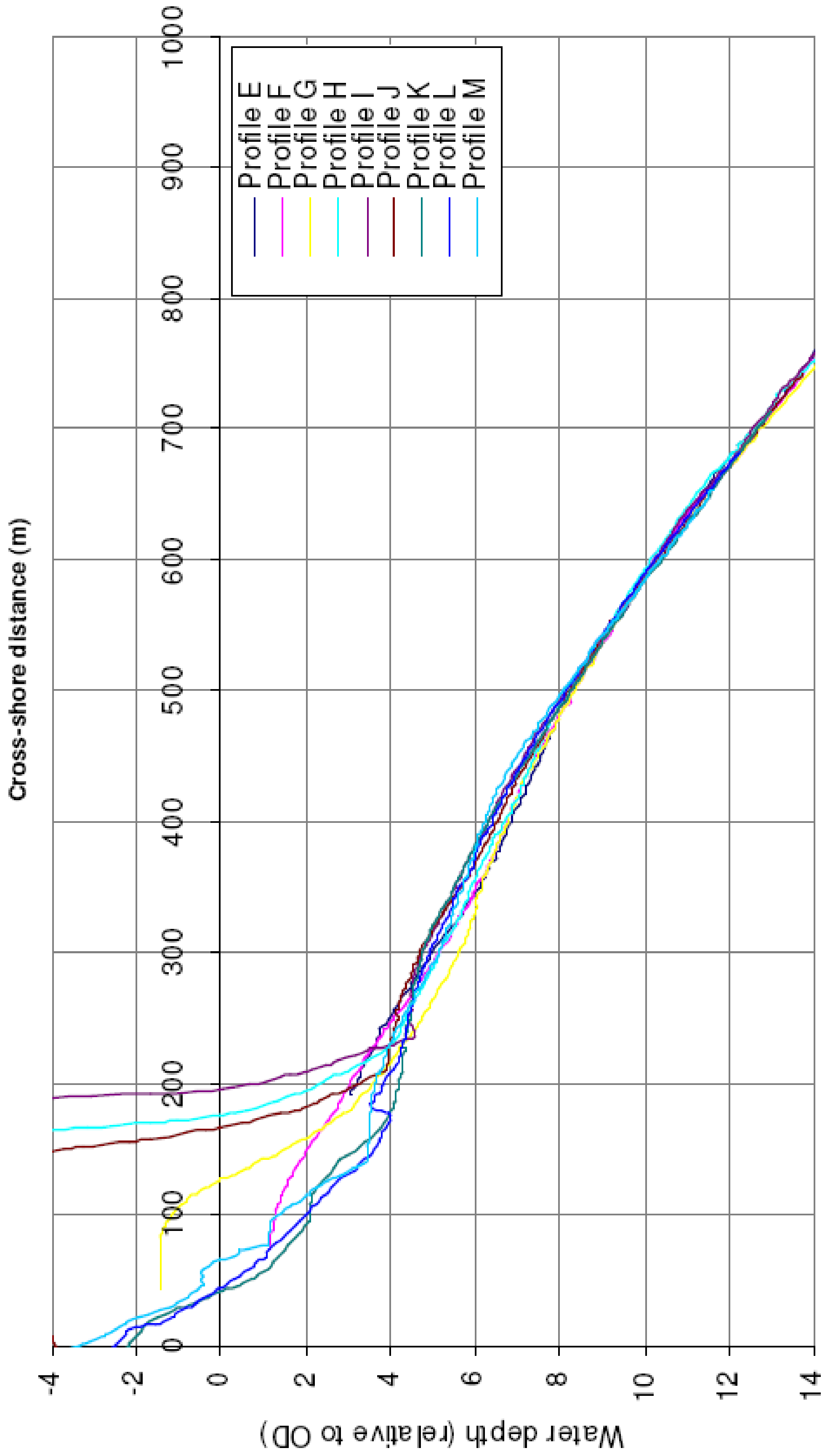
Profile surveys at Catalan Bay
(development over time)

WL | DELFT HYDRAULICS	H4725	Fig. 3.3

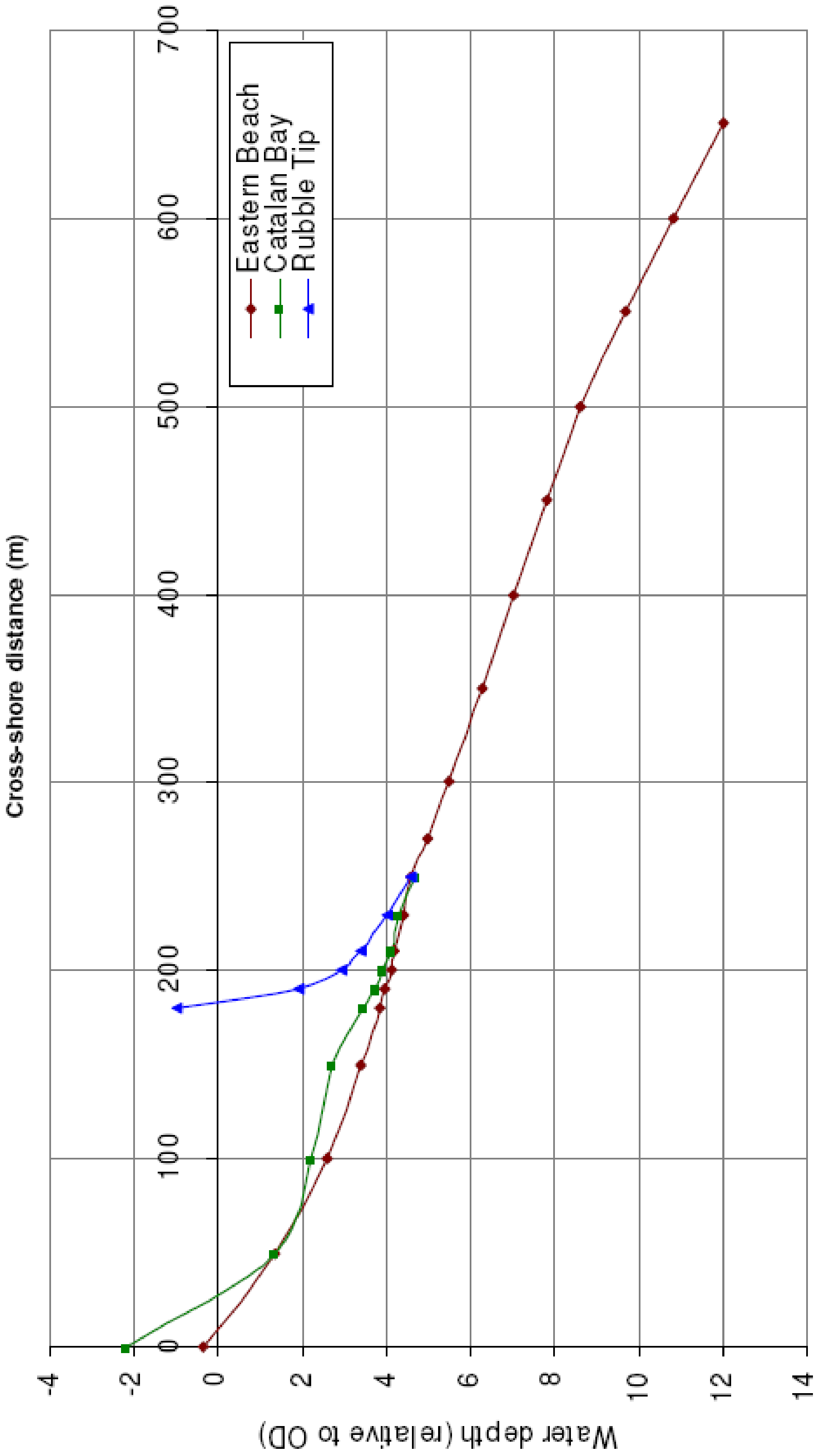


Location of profile samples (survey 2005) and rays

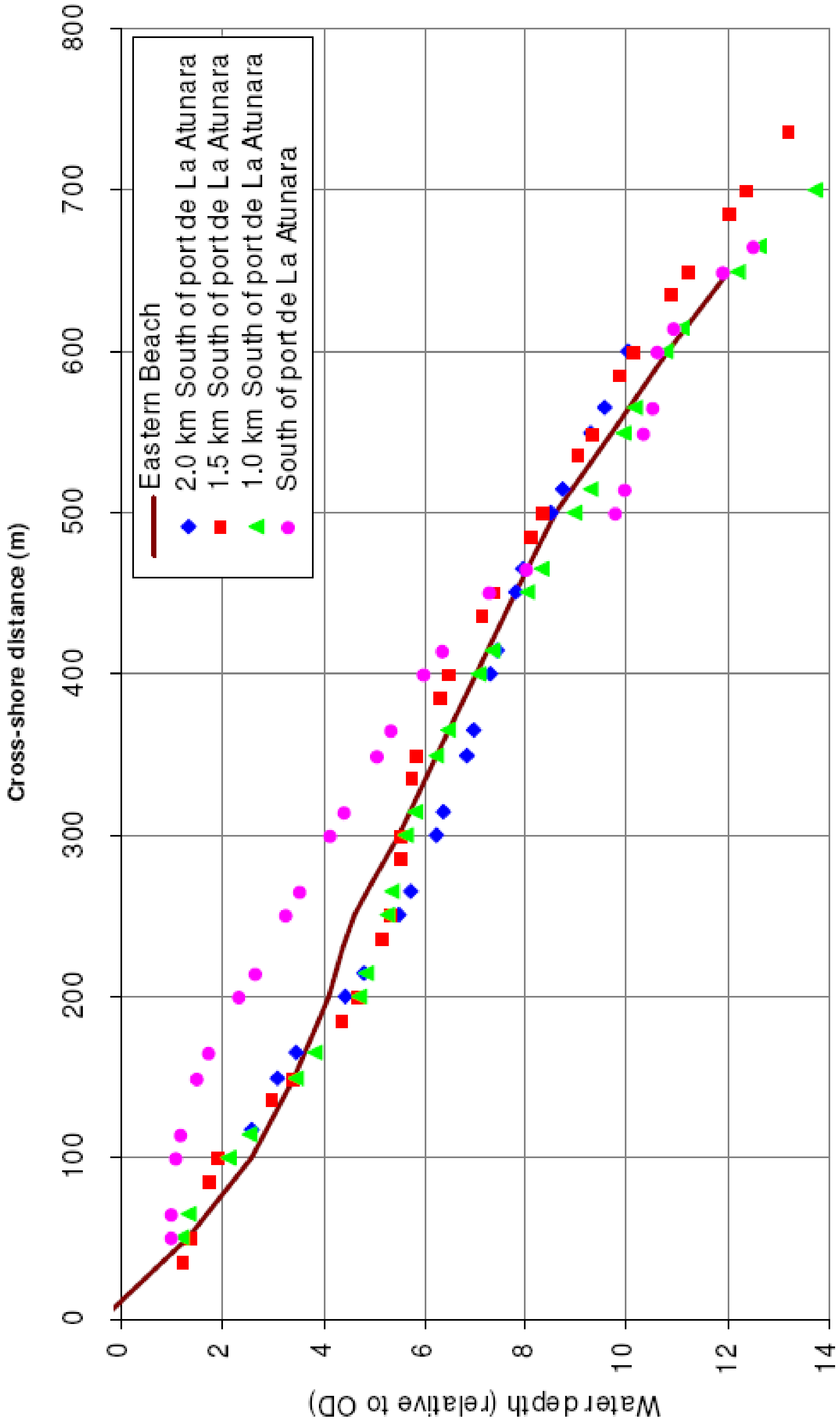
WL | DELFT HYDRAULICS	H4725	Fig. 3.4



Interpolated cross−shore profiles at different rays		
WL | DELFT HYDRAULICS	H4725	Fig. 3.5

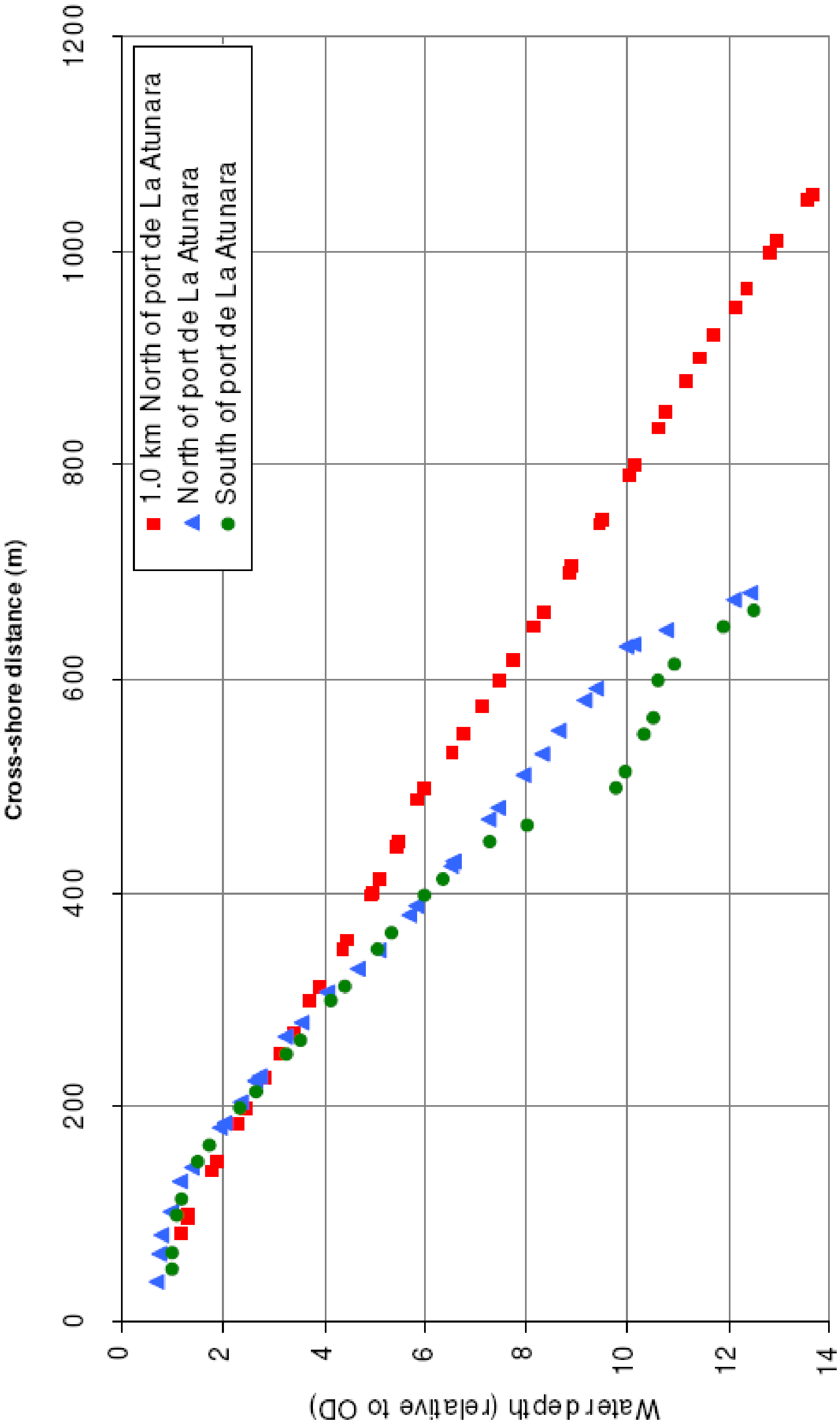


Schematization of the cross−shore profiles along the Gibraltar coast		
WL | DELFT HYDRAULICS	H4725	Fig. 3.6

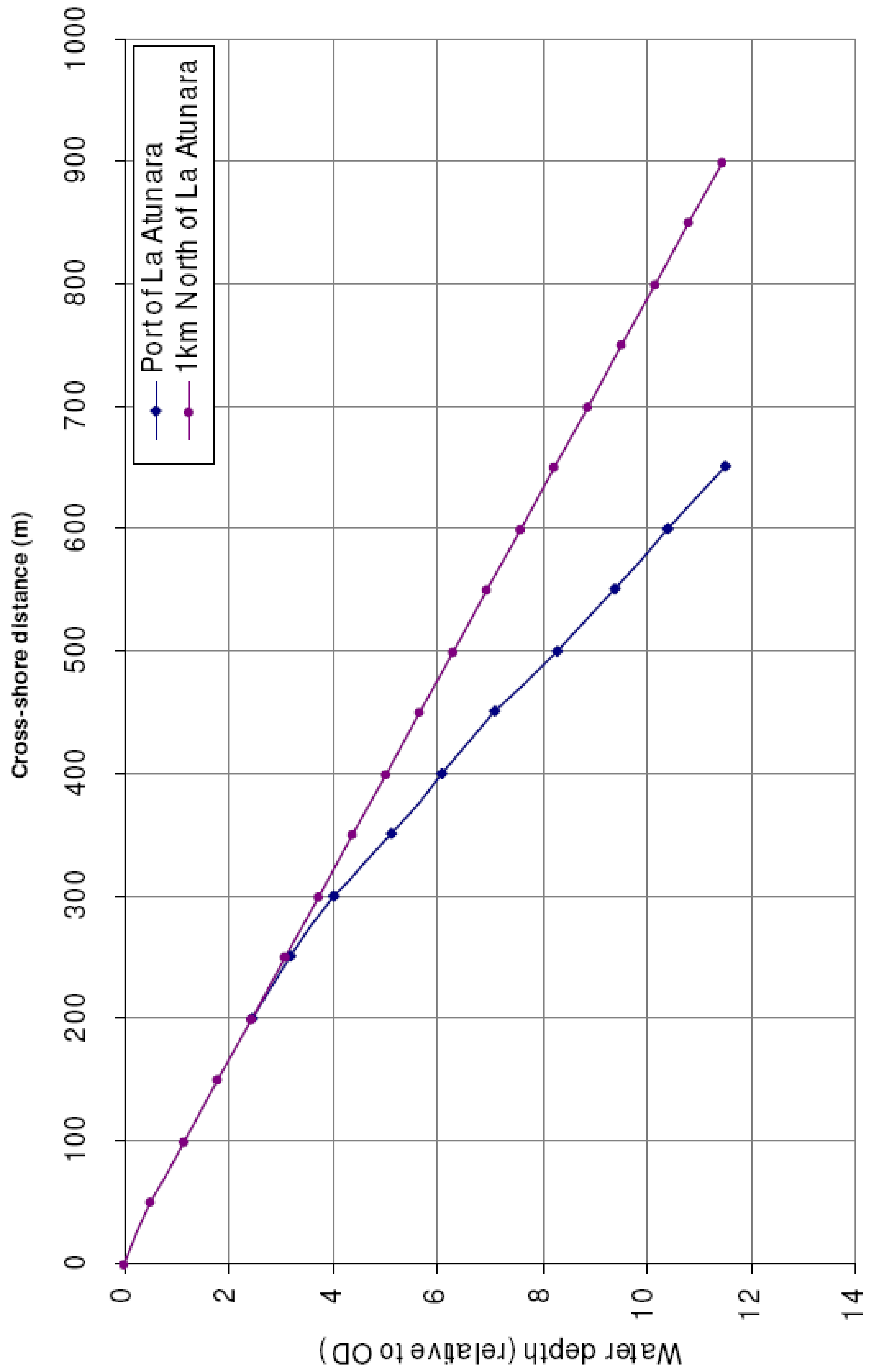


Interpolated cross−shore profiles between Eastern Beach and La Atunara harbour

WL | DELFT HYDRAULICS	H4725	Fig. 3.7

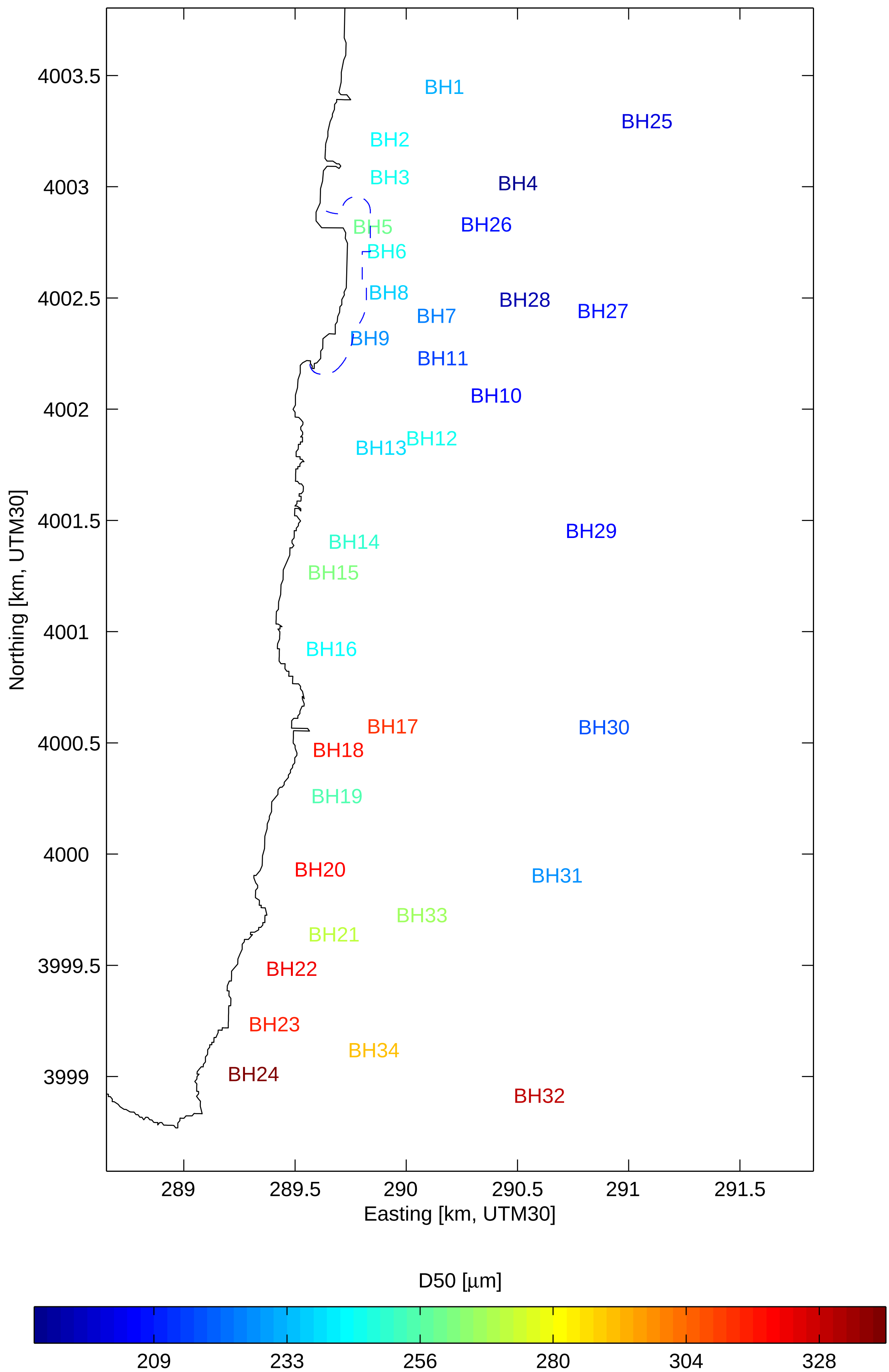


Interpolated cross−shore profiles close to and 1km North of la Atunara harbour		
WL | DELFT HYDRAULICS	H4725	Fig. 3.8



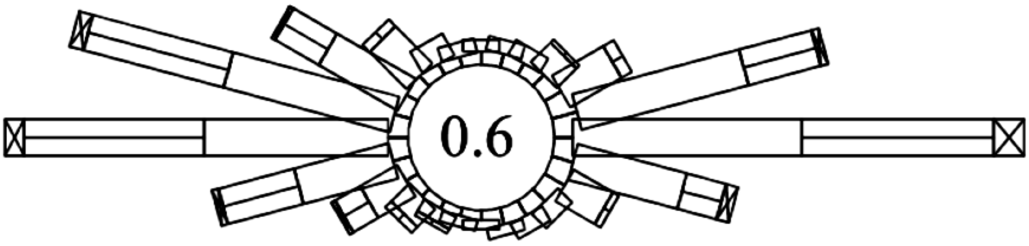
Schematization of the cross−shore profiles close to La Atunara harbour and 1km North of La Atunara harbour

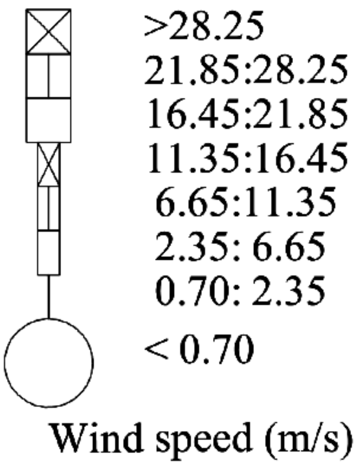
WL | DELFT HYDRAULICS | H4725 | Fig. 3.9



Spatial distribution D50

WL | DELFT HYDRAULICS	H4725	Fig. 3.10



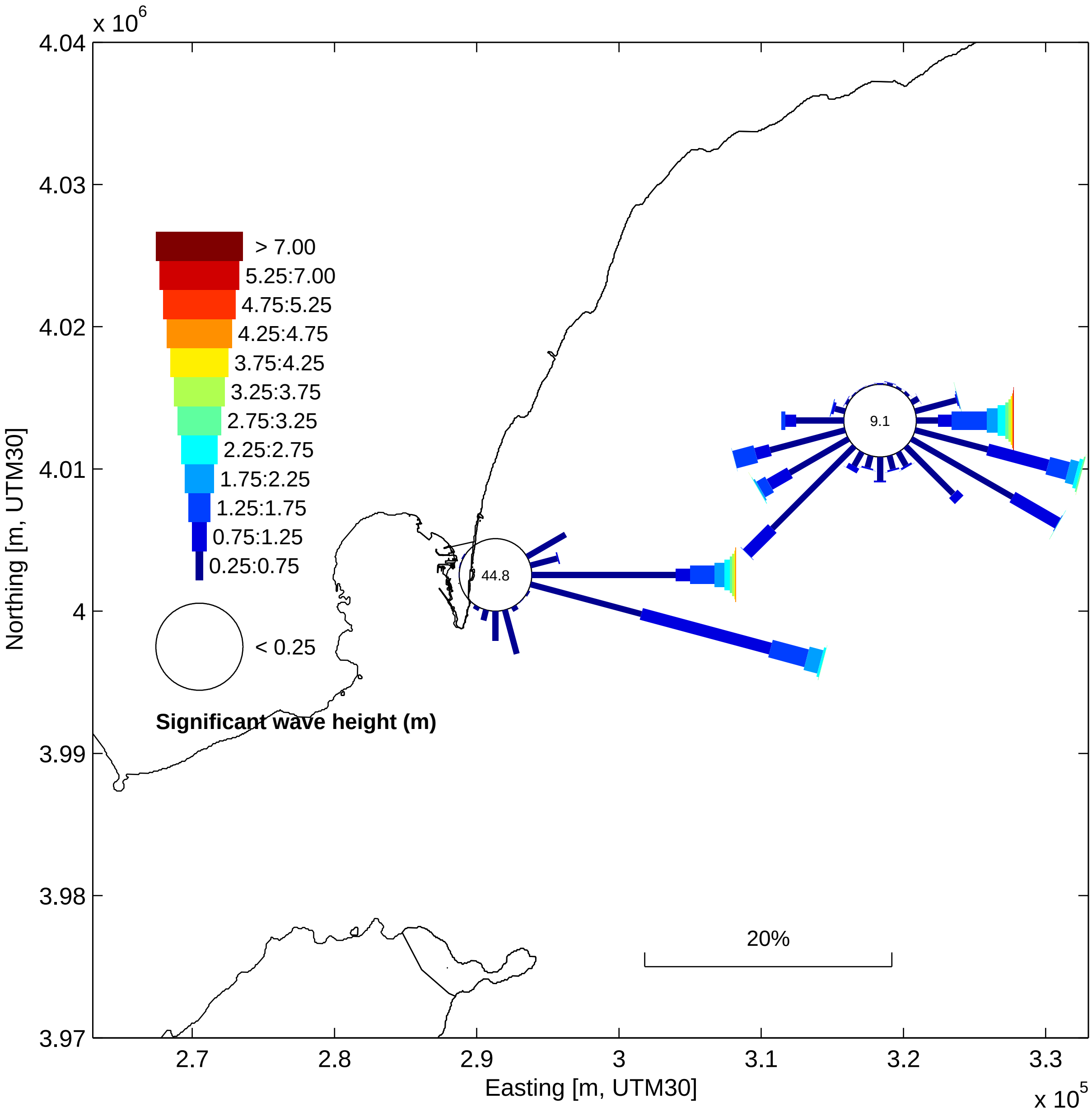


EXPLANATION

Item:	Represents:
Type of bar	Height / Speed
Direction of bar (to centre of rose)	Direction
Length of bar	Occurrence (%
Number in centre of rose	Occurrence (%) in lowest class

20.0 %

Characteristic wind rose

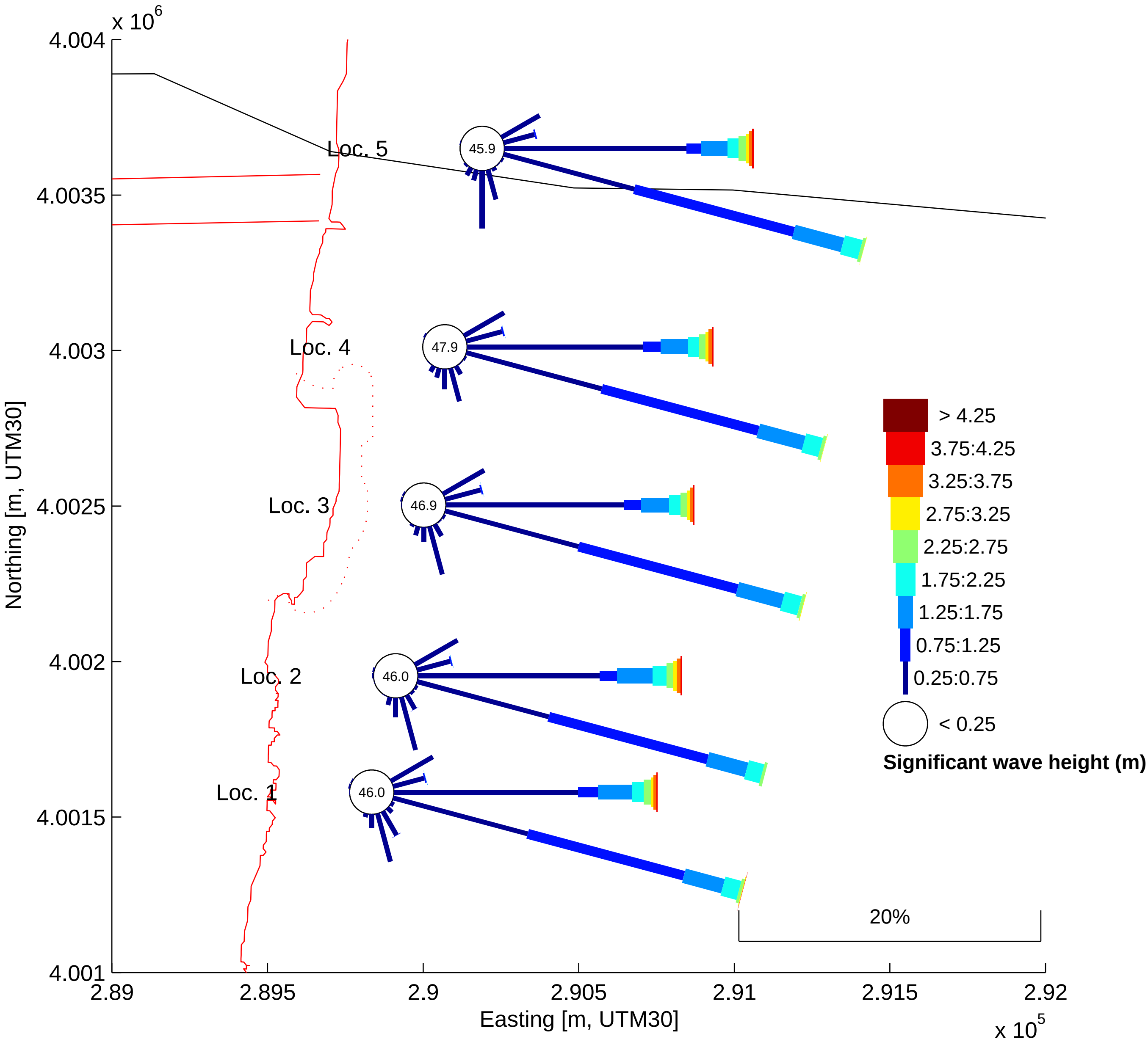


EXPLANATION

Item:	Represents:
Type of bar Direction of bar (to centre of rose) Length of bar Number in centre of rose	Significant wave height (m) Direction Occurence (%) Occurence (%) in lowest class

Characteristic offshore wave rose

WL | DELFT HYDRAULICS | H4725 | Fig. 3.12

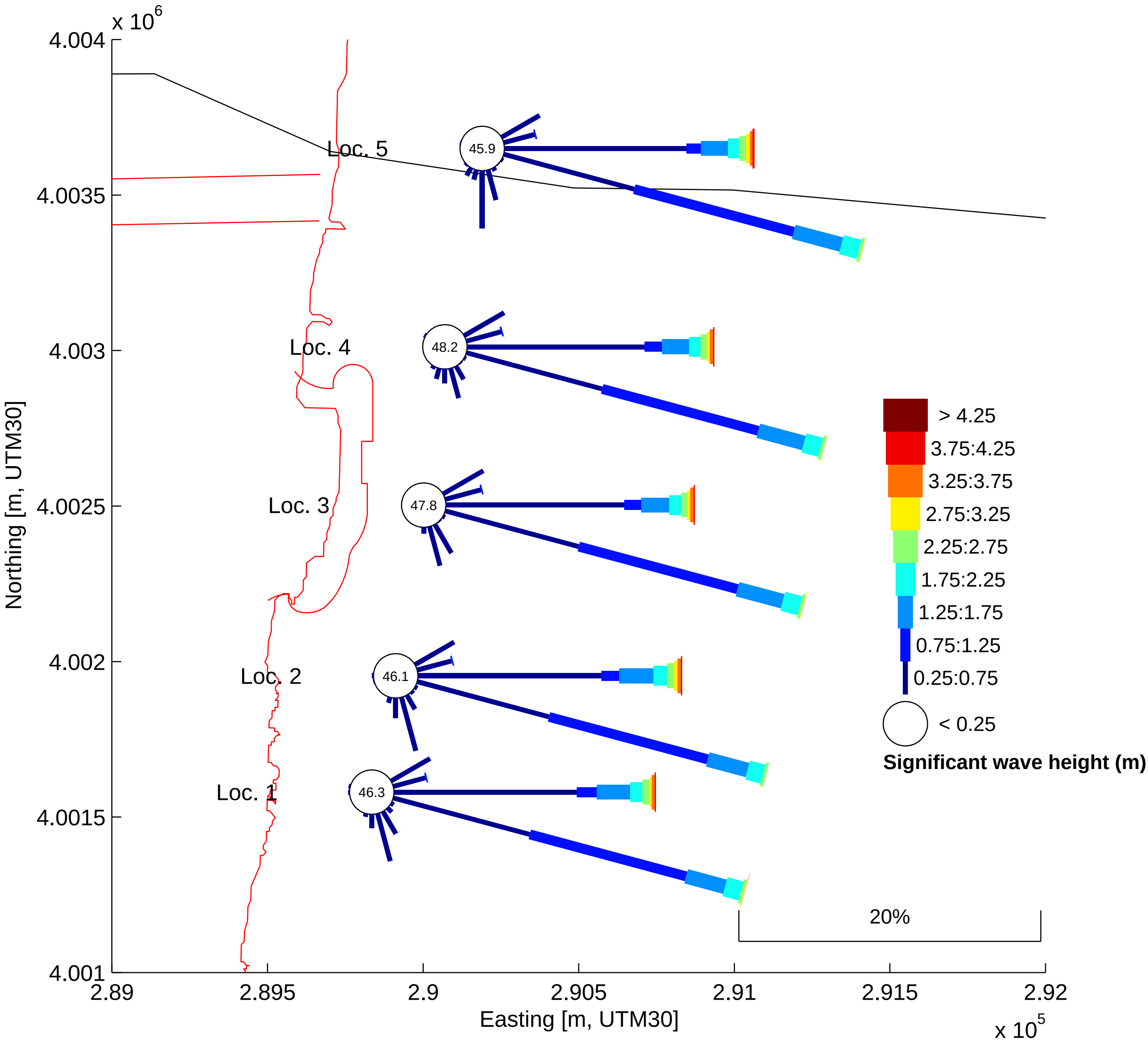


EXPLANATION

Item:	Represents:
Type of bar	Significant wave height (m)
Direction of bar (to centre of rose)	Direction
Length of bar	Occurence (%)
Number in centre of rose	Occurence (%) in lowest class

Nearshore wave roses without the scheme

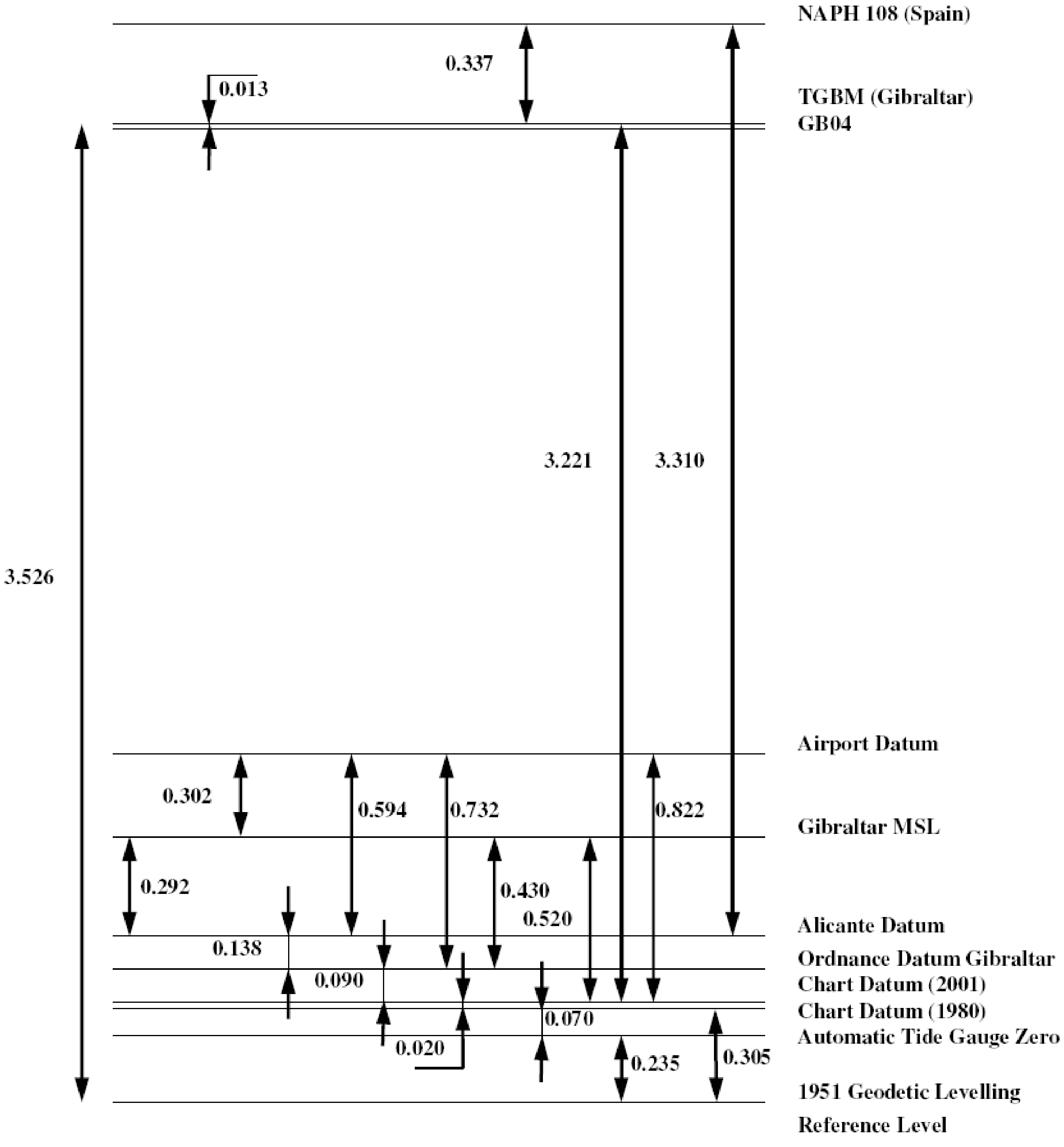
WL | DELFT HYDRAULICS | H4725 | Fig. 3.13



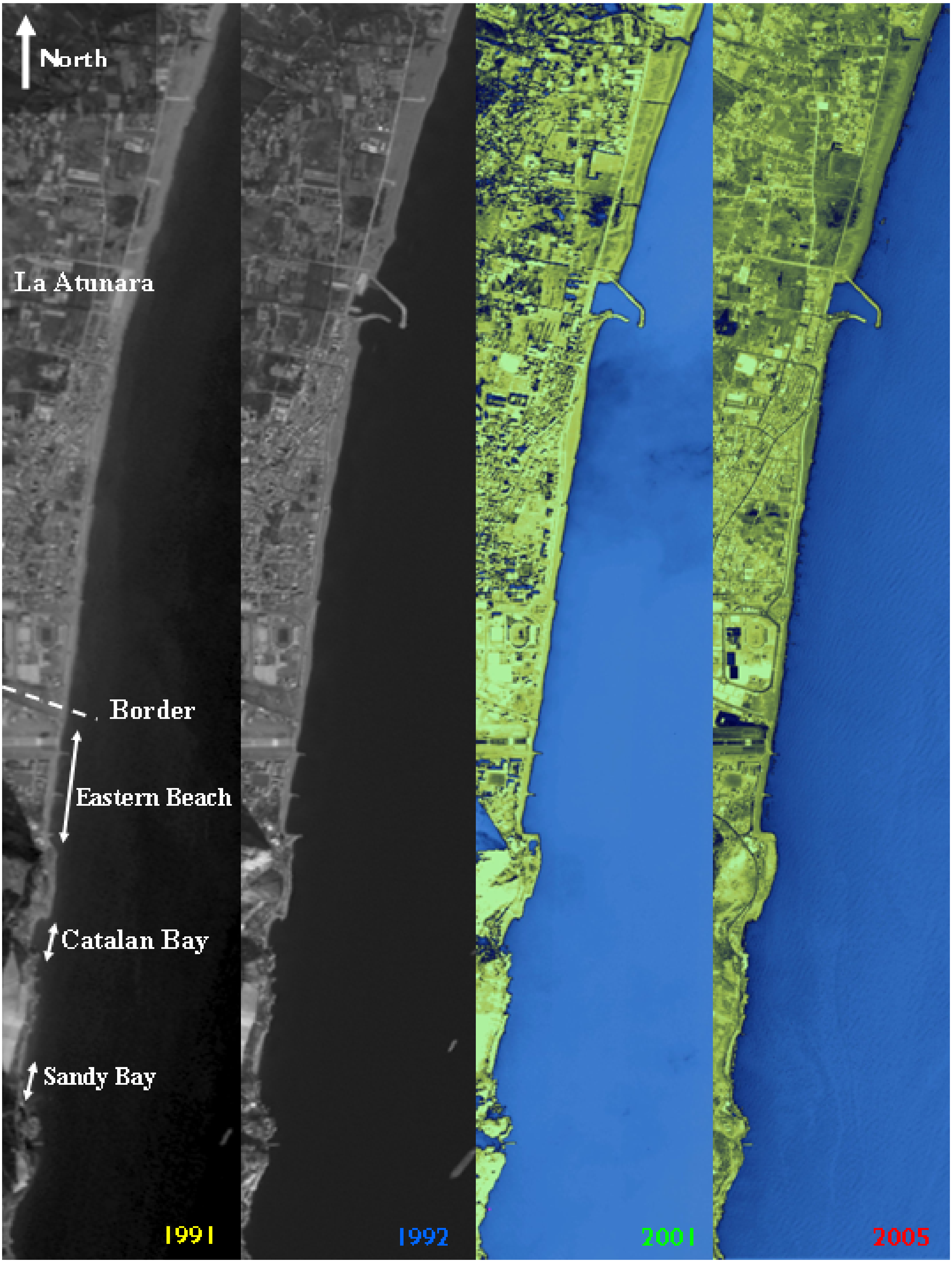
EXPLANATION

Item:	Represents:
Type of bar	Significant wave height (m)
Direction of bar (to centre of rose)	Direction
Length of bar	Occurence (%)
Number in centre of rose	Occurence (%) in lowest class

Nearshore wave roses with the scheme		
WL | DELFT HYDRAULICS	H4725	Fig. 3.14

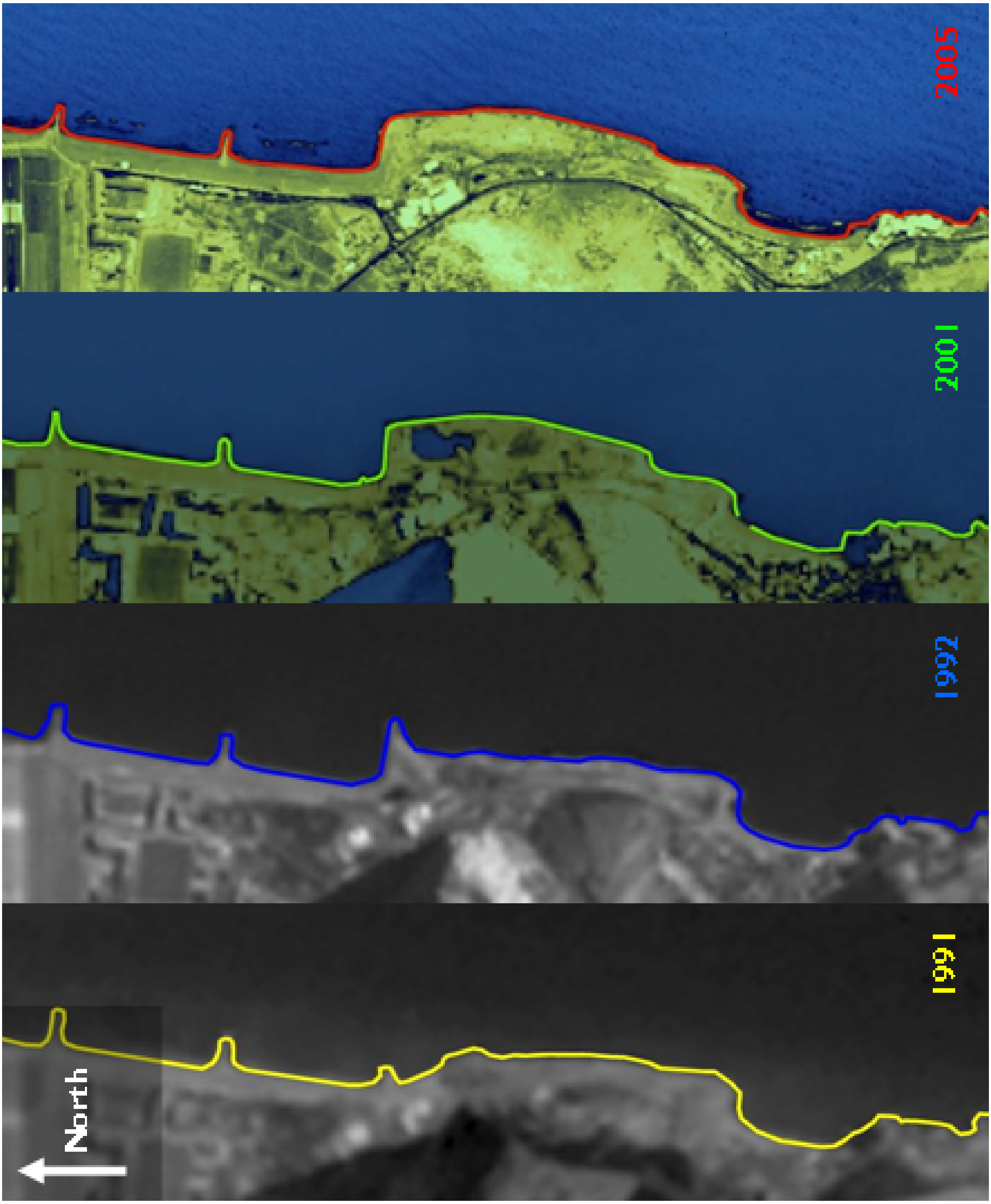


Vertical datums (in meters)		
WL | DELFT HYDRAULICS	H4725	Fig. 3.15



Coastline of East Gibraltar and adjacent Spanish coast
(satellite images)

WL | DELFT HYDRAULICS	H4725	Fig. 4.1



Coastline around Rubble Tip area		
WL | DELFT HYDRAULICS	H4725	Fig. 4.2

Coastline at Catalan Bay		
WL | DELFT HYDRAULICS	H4725	Fig. 4.3

Catalan Bay, inaccuracy ranges of the coastlines

WL | DELFT HYDRAULICS | H4725 | Fig. 4.4

Catalan Bay, envelopes of observed shorelines

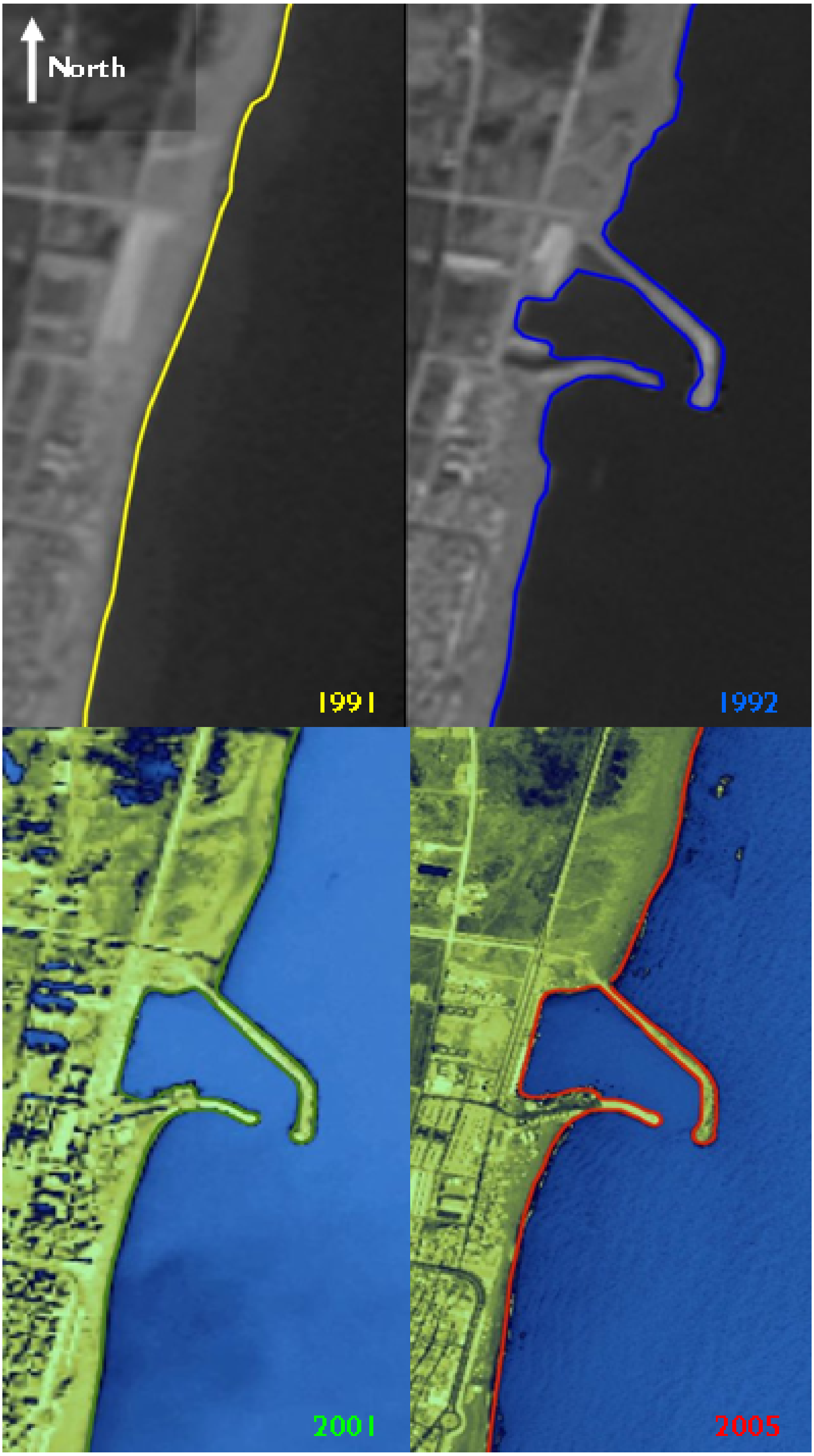
WL | DELFT HYDRAULICS | H4725 | Fig. 4.5

Coastline at Eastern Beach

WL | DELFT HYDRAULICS	H4725	Fig. 4.6

Eastern Beach, inaccuracy ranges of the coastlines		
WL | DELFT HYDRAULICS	H4725	Fig. 4.7

Eastern Beach, envelopes of observed shorelines		
WL | DELFT HYDRAULICS	H4725	Fig. 4.8



Coastline at La Atunara harbour

WL | DELFT HYDRAULICS | H4725 | Fig. 4.9

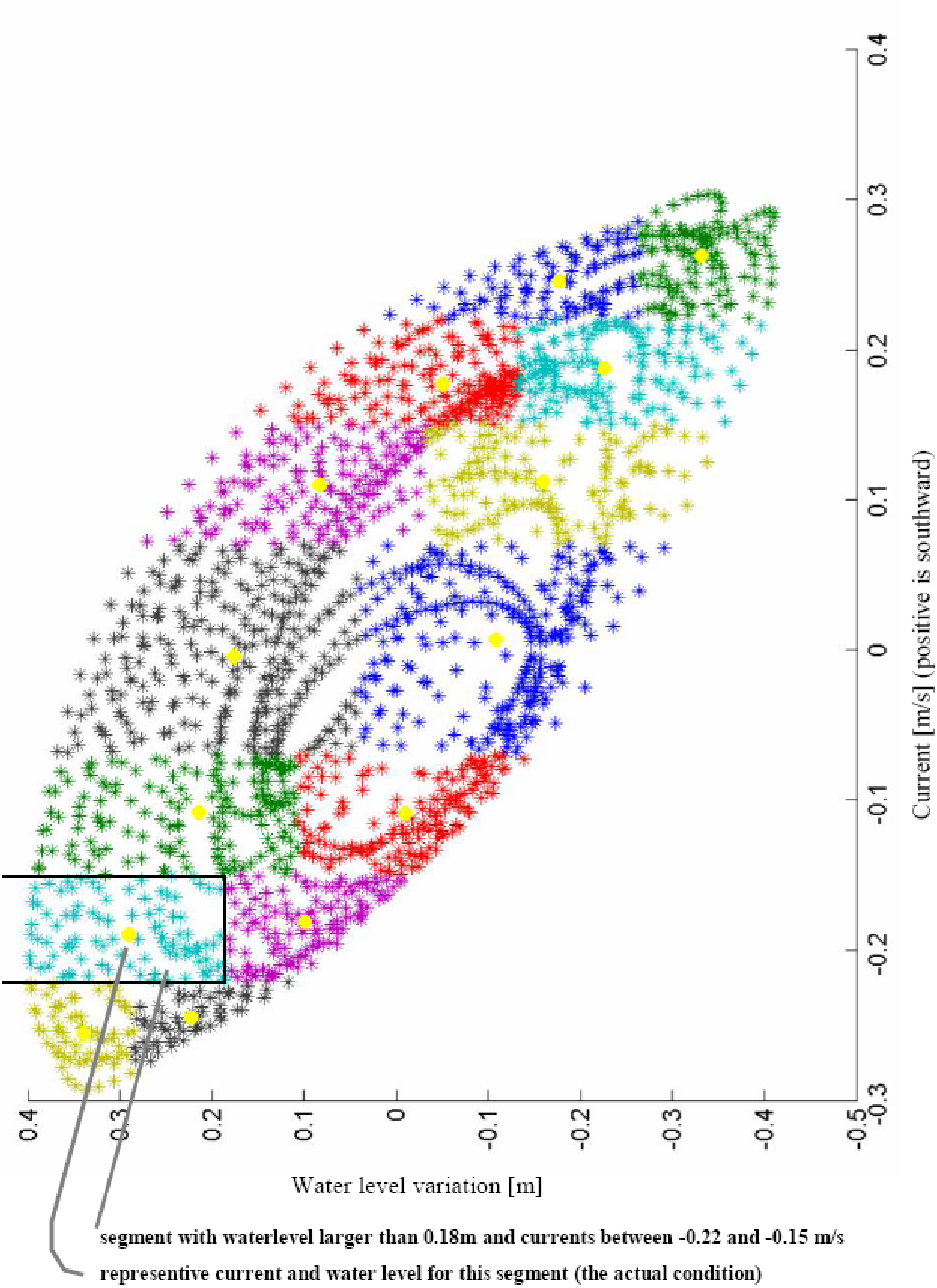
Coastline development at La Atunara harbour

WL | DELFT HYDRAULICS	H4725	Fig. 4.10

La Atunara harbour, inaccuracy ranges of the coastlines

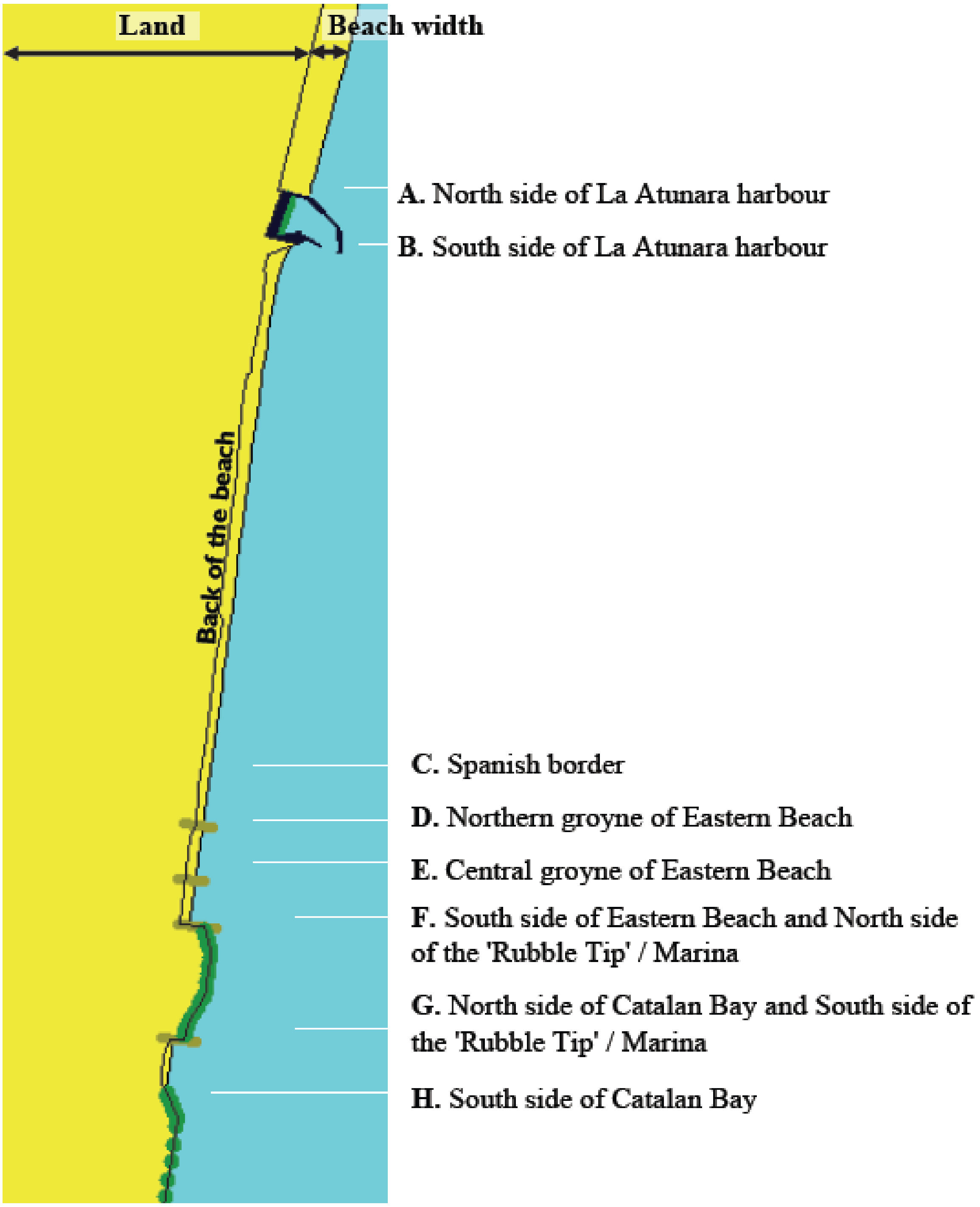
WL | DELFT HYDRAULICS	H4725	Fig. 4.11

Coastline schematization in Unibest−CL+ (with locations of waveclimate samples)		
WL | DELFT HYDRAULICS	H4725	Fig. 4.12

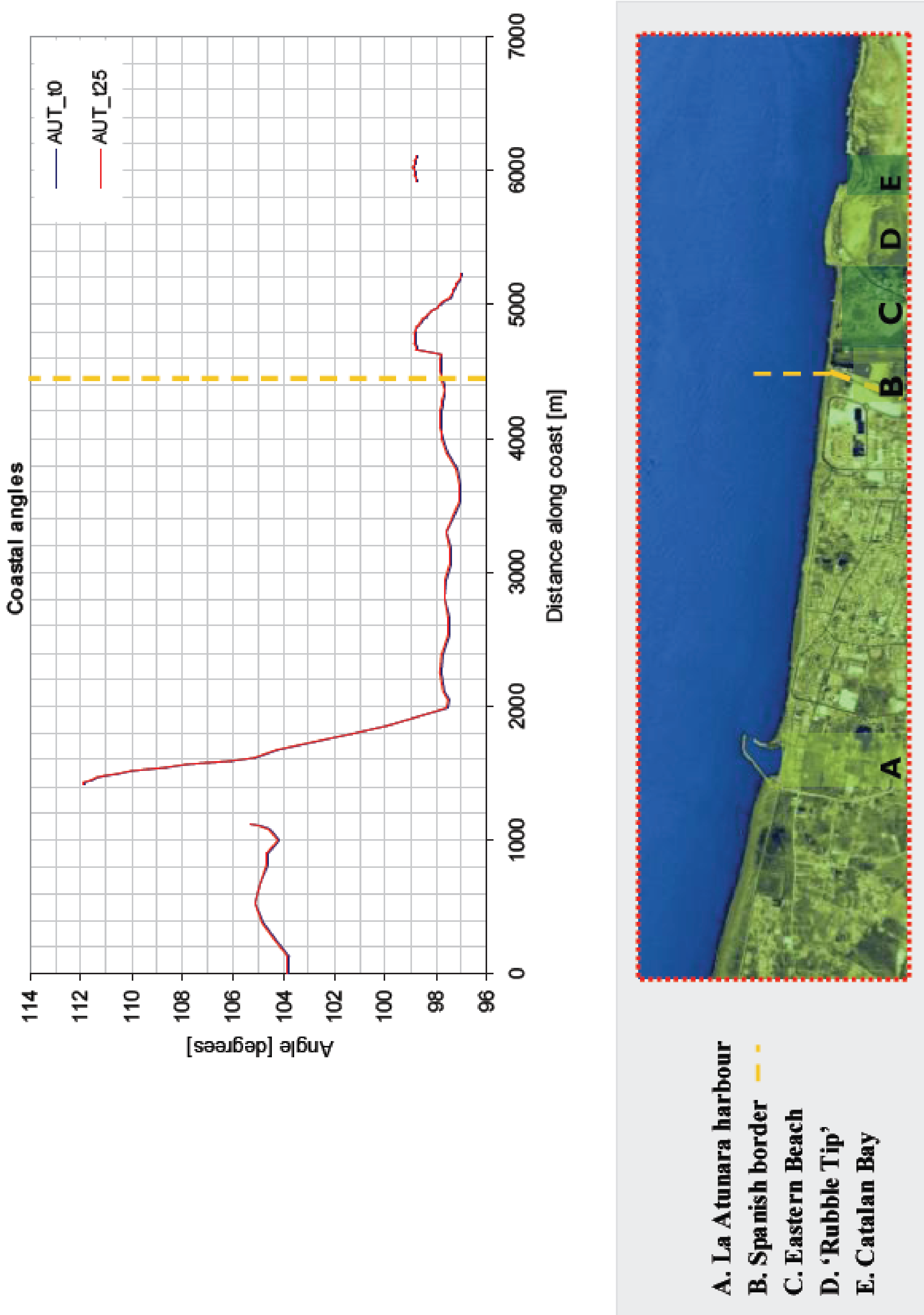


Classification of tidal currents at Eastern Beach (every color represents a condition, see Table 4.4)		
WL | DELFT HYDRAULICS	H4725	Fig. 4.13

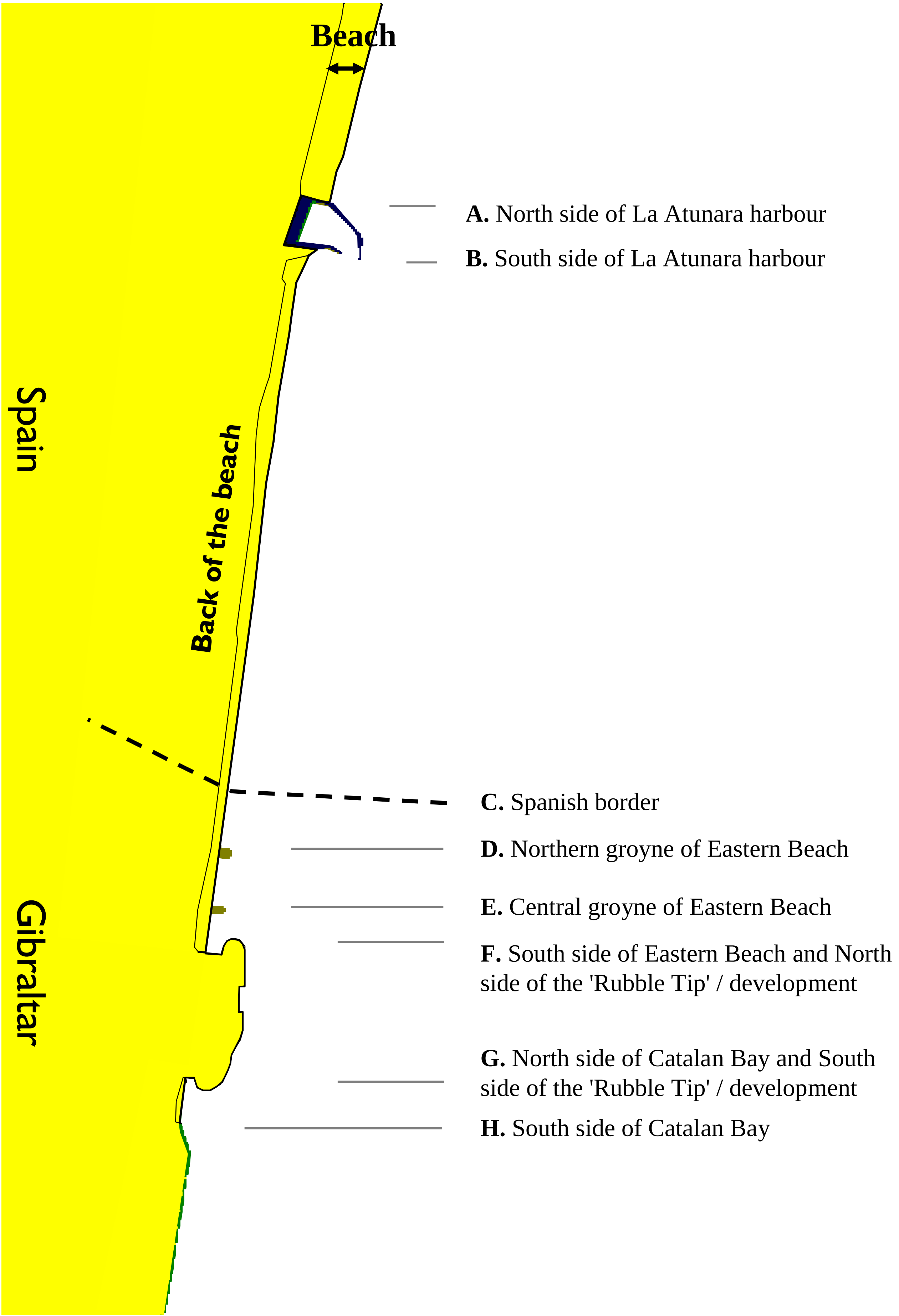
x	Location	
1140	A	North side of La Atunara harbour
1410	B	South side of La Atunara harbour
4430	C	Spanish border
4670	D	Northern groyne at Eastern Beach
4970	E	Central groyne at Eastern Beach
5220	F	South side of Eastern Beach and North side of the 'Rubble Tip'
5840	G	North side of Catalan Bay and South side of the 'Rubble Tip'
6130	H	South side of Catalan Bay



Characteristic locations in the model
(x' relative to the northern boundary of the model area)



Coast angles for the baseline prediction		
WL | DELFT HYDRAULICS	H4725	Fig. 4.15



Coastline schematiation in Unibest-CL+ with scheme

WL | DELFT HYDRAULICS	H4725	Fig. 5.1

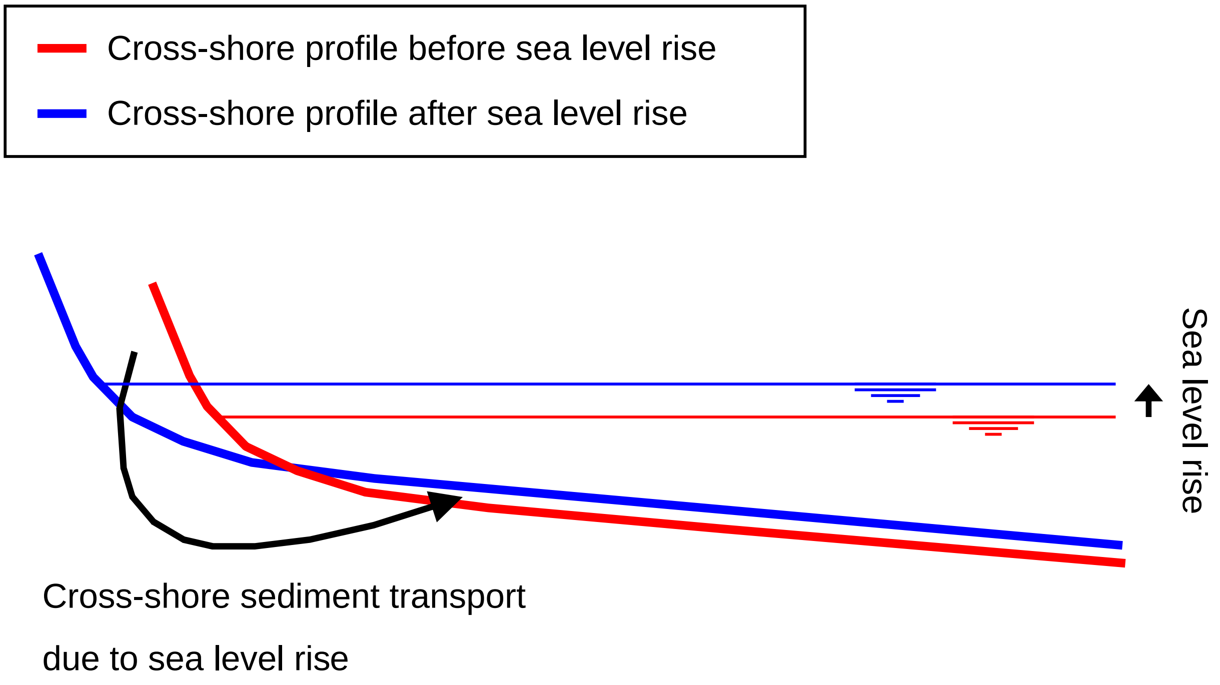
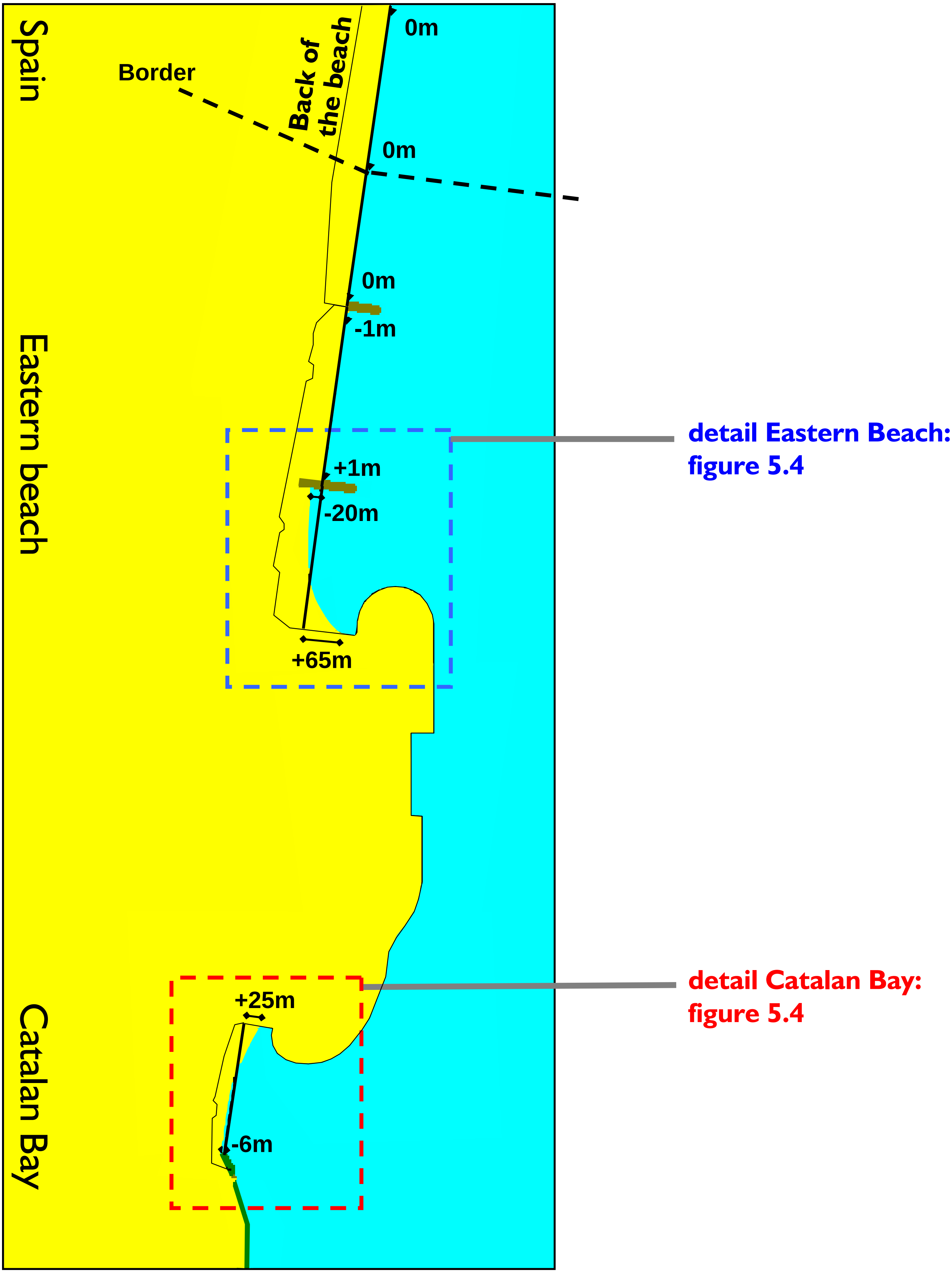


Illustration of coastline retreat due to sea level rise

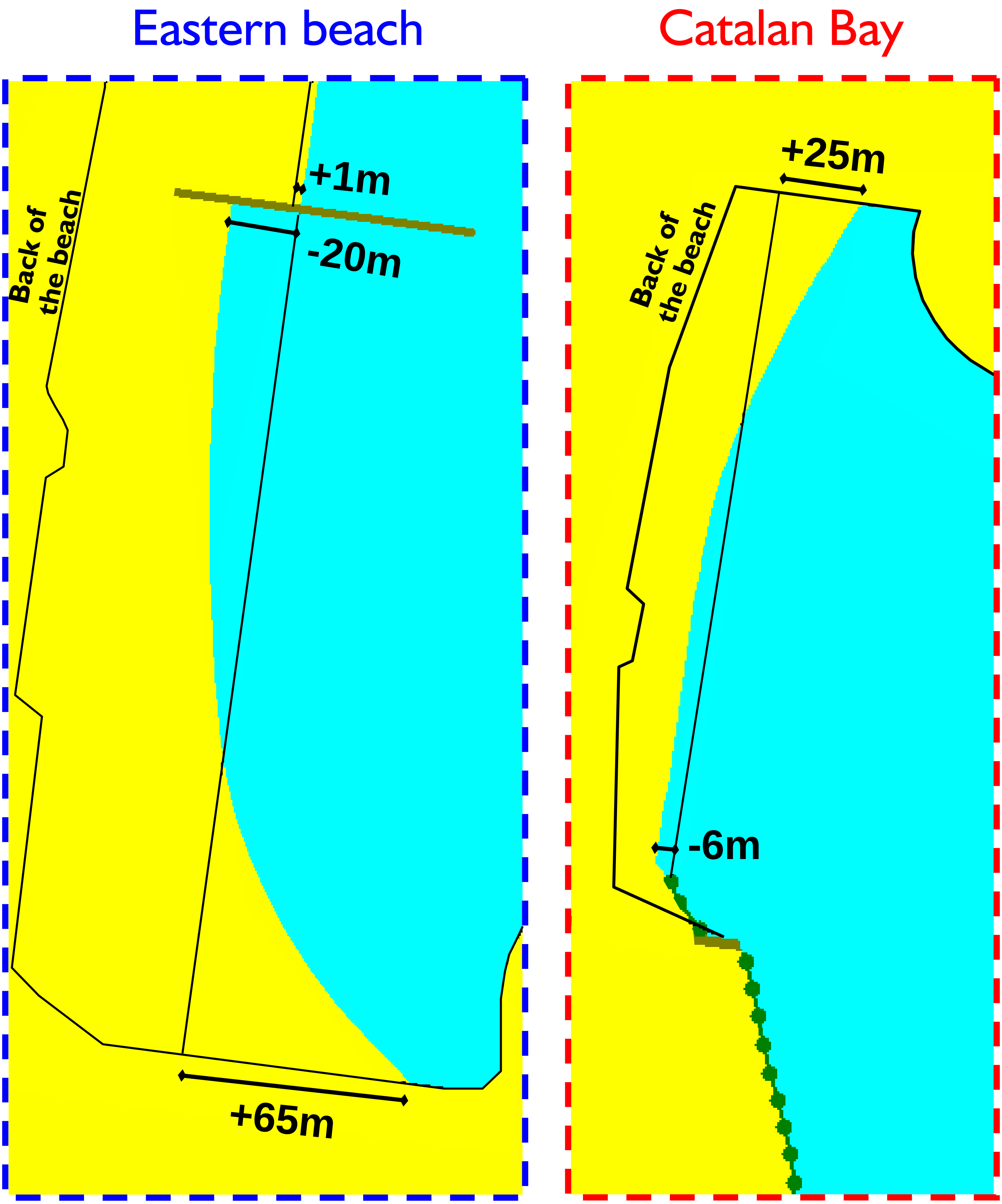


Coastline change over 5 years in Unibest-CL+
for situation with planned scheme (longshore effects only)

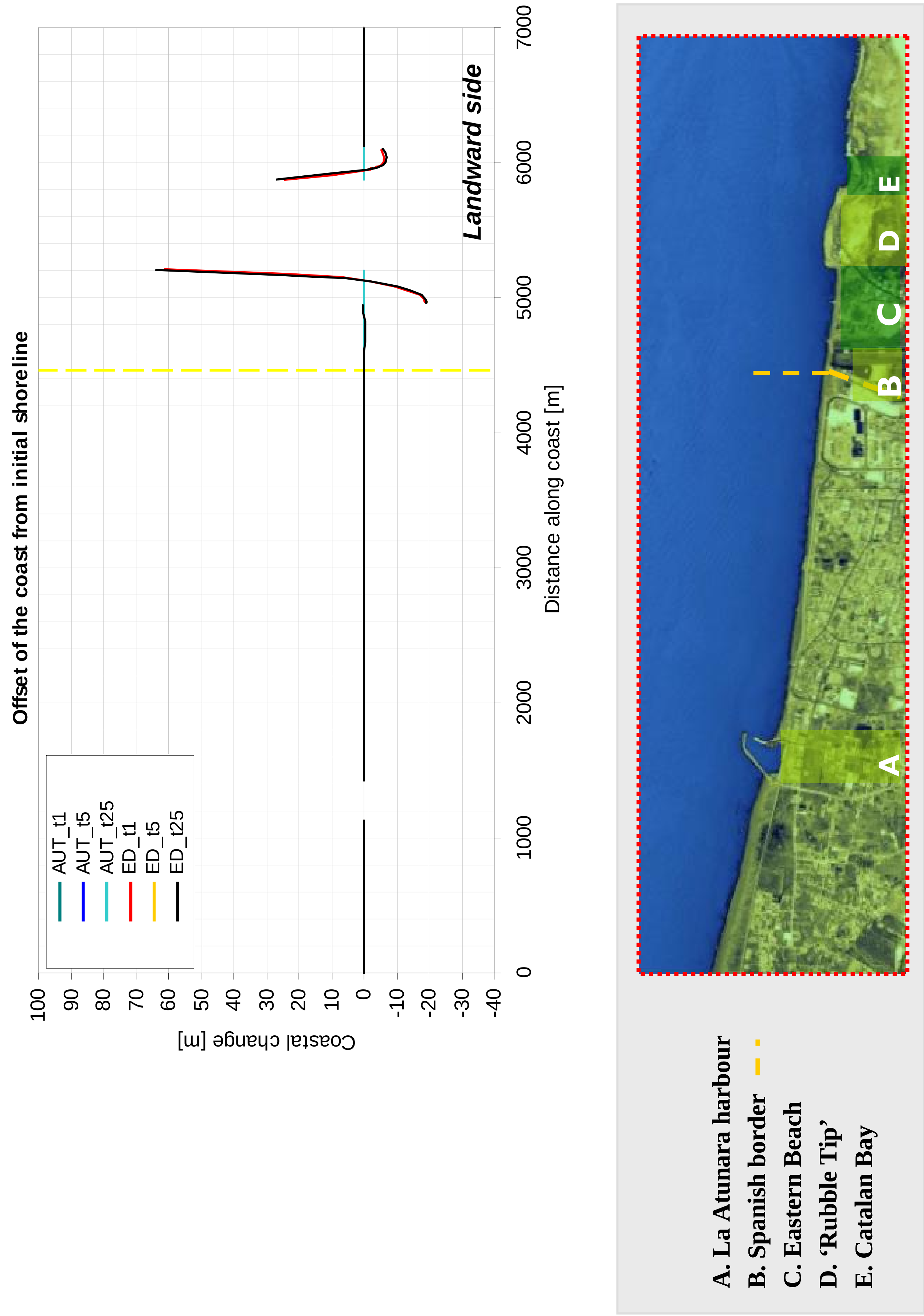
WL | DELFT HYDRAULICS

H4725

Fig. 5.3



Coastline change over 5 years in Unibest-CL+ at Eastern Beach and Catalan Bay (longshore effects only)		
WL | DELFT HYDRAULICS	H4725	Fig. 5.4



Impact of planned development (longshore effects only)

WL | DELFT HYDRAULICS	H4725	Fig. 5.5

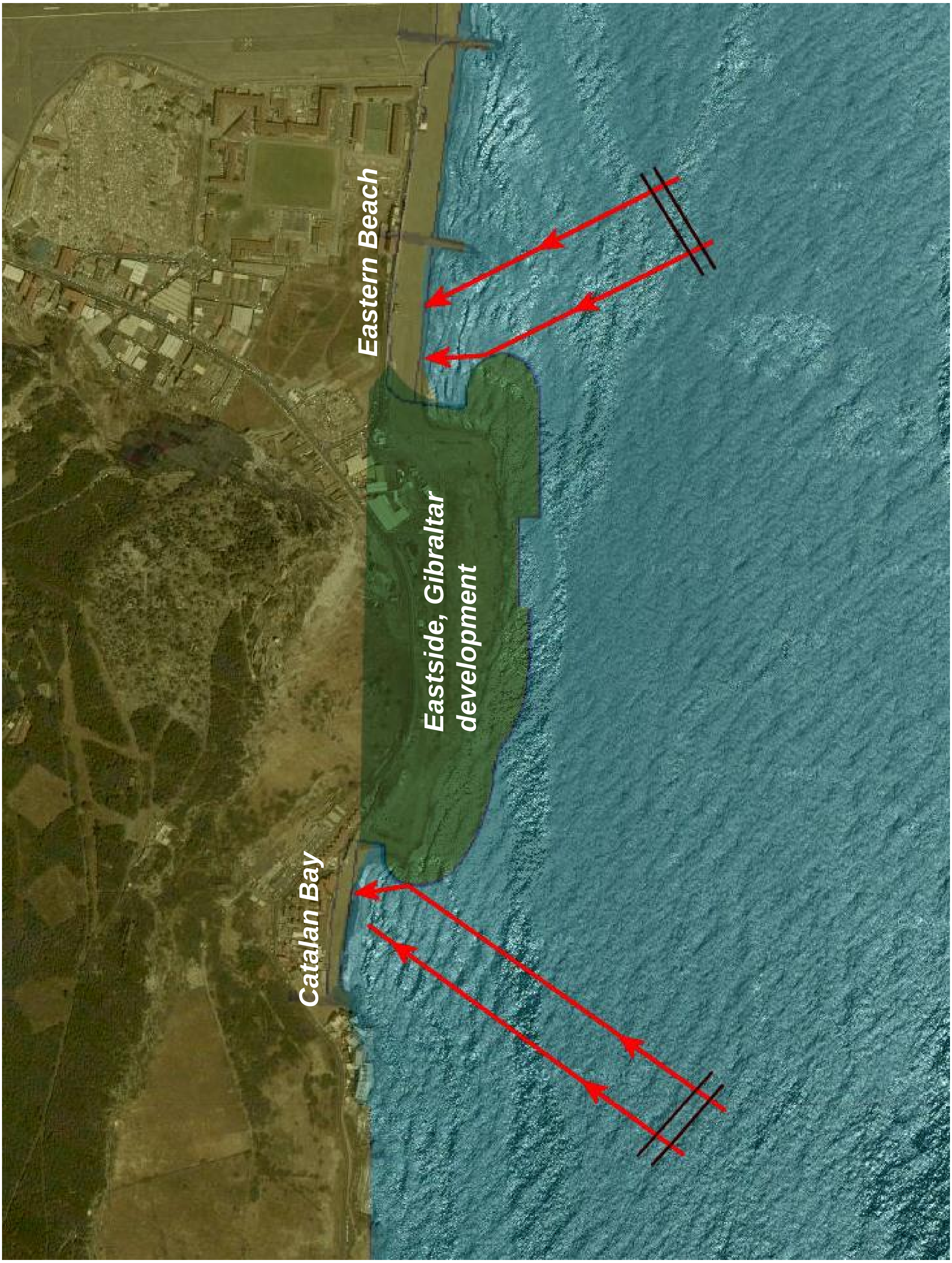
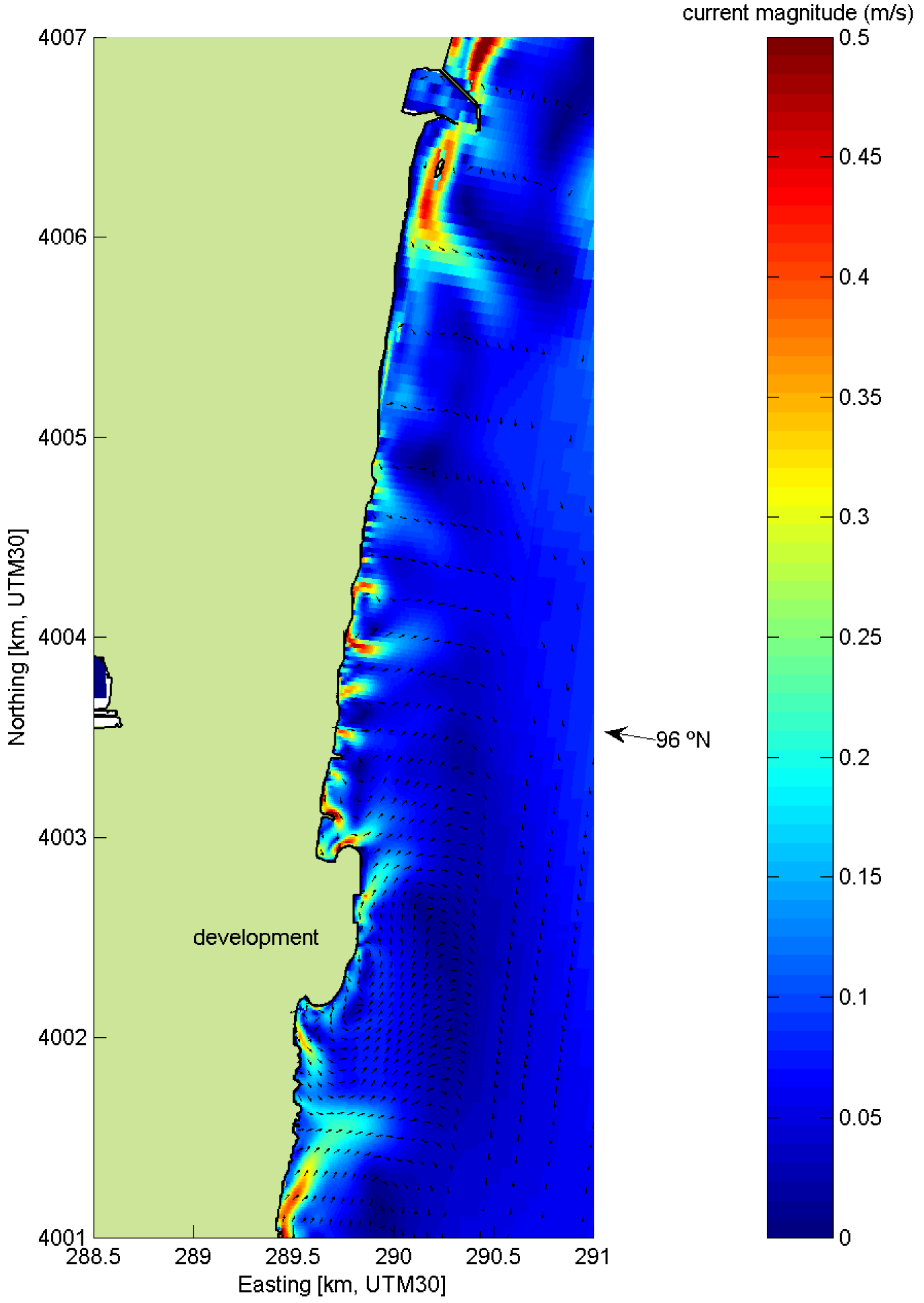


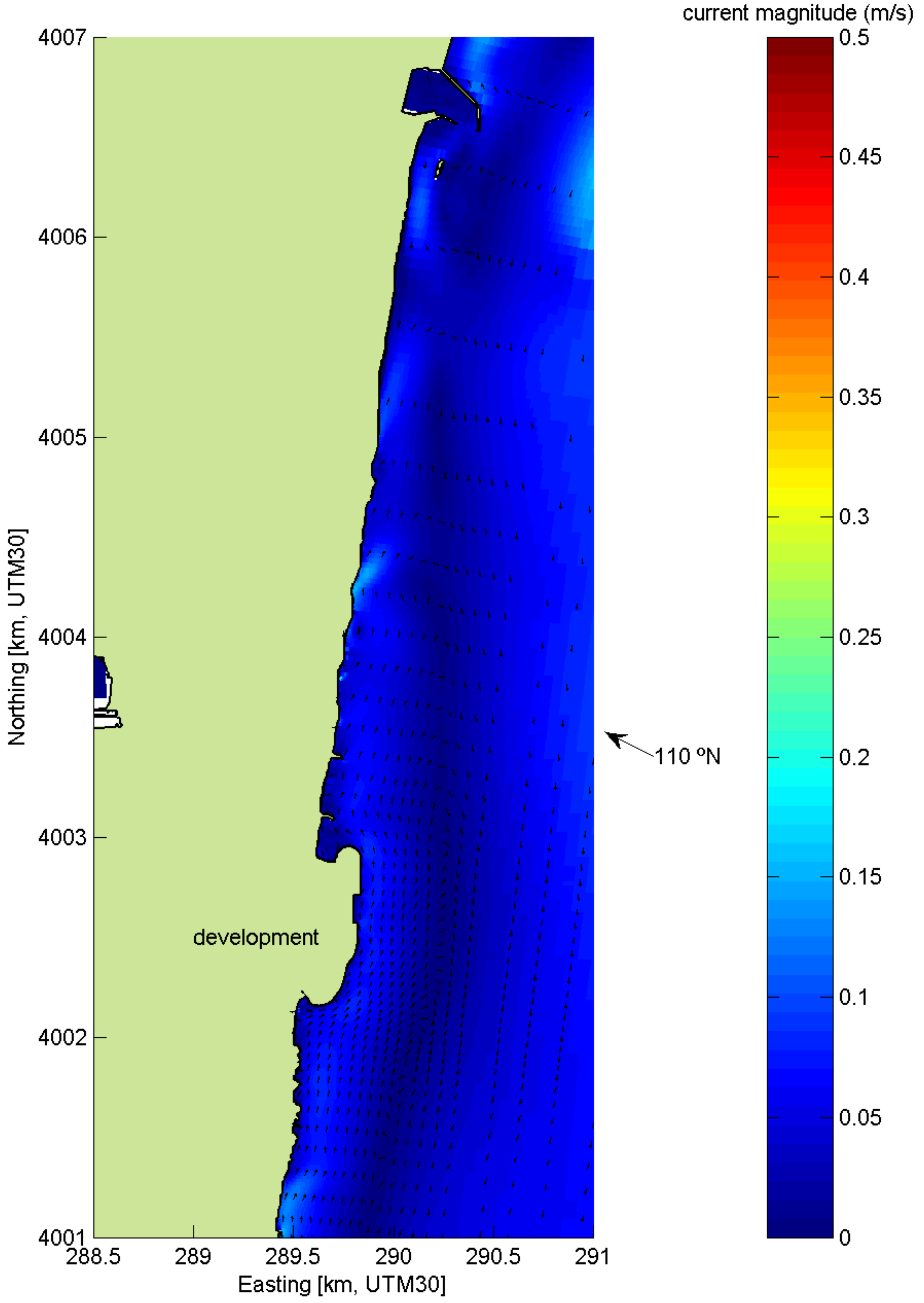
Illustration of wave reflection against the planned development
(towards the beaches of Eastern Beach and Catalan Bay)

WL | DELFT HYDRAULICS	H4725	Fig. 5.6



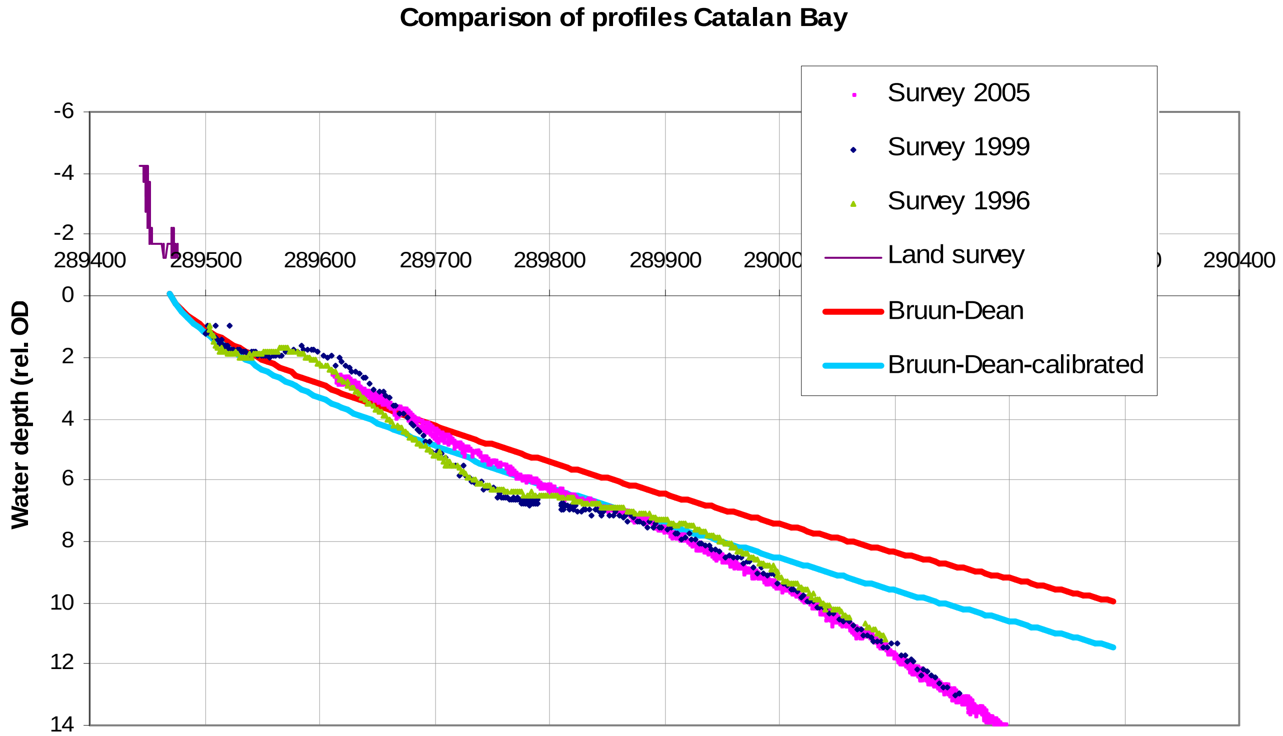
Flow patterns for situation with pronounced rip currents
1 year morphological simulation, wave condition 4

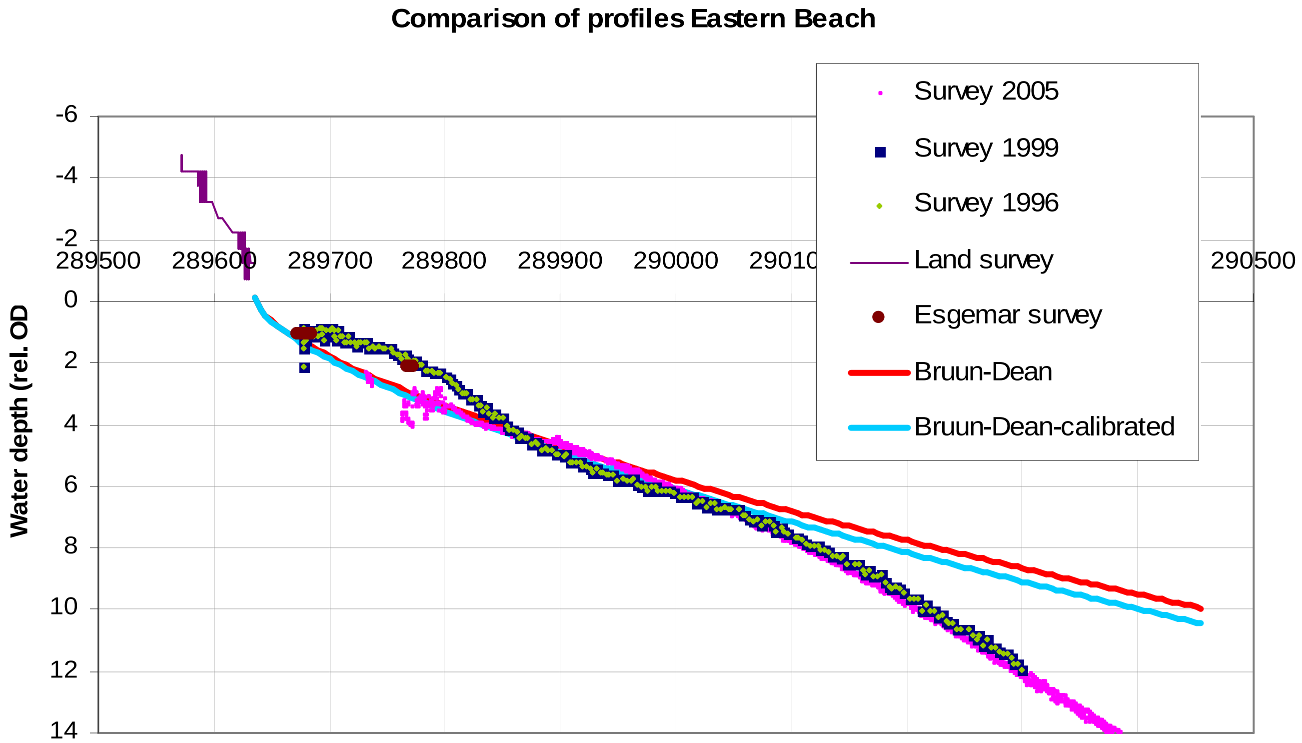
WL | DELFT HYDRAULICS	H4725	Fig. 5.7



Flow patterns for situation without rip currents
1 year morphological simulation, wave condition 9

WL | DELFT HYDRAULICS	H4725	Fig. 5.8





Calibrated Bruun-Dean profile and measured profiles

WL | DELFT HYDRAULICS	H4725	Fig. 5.9

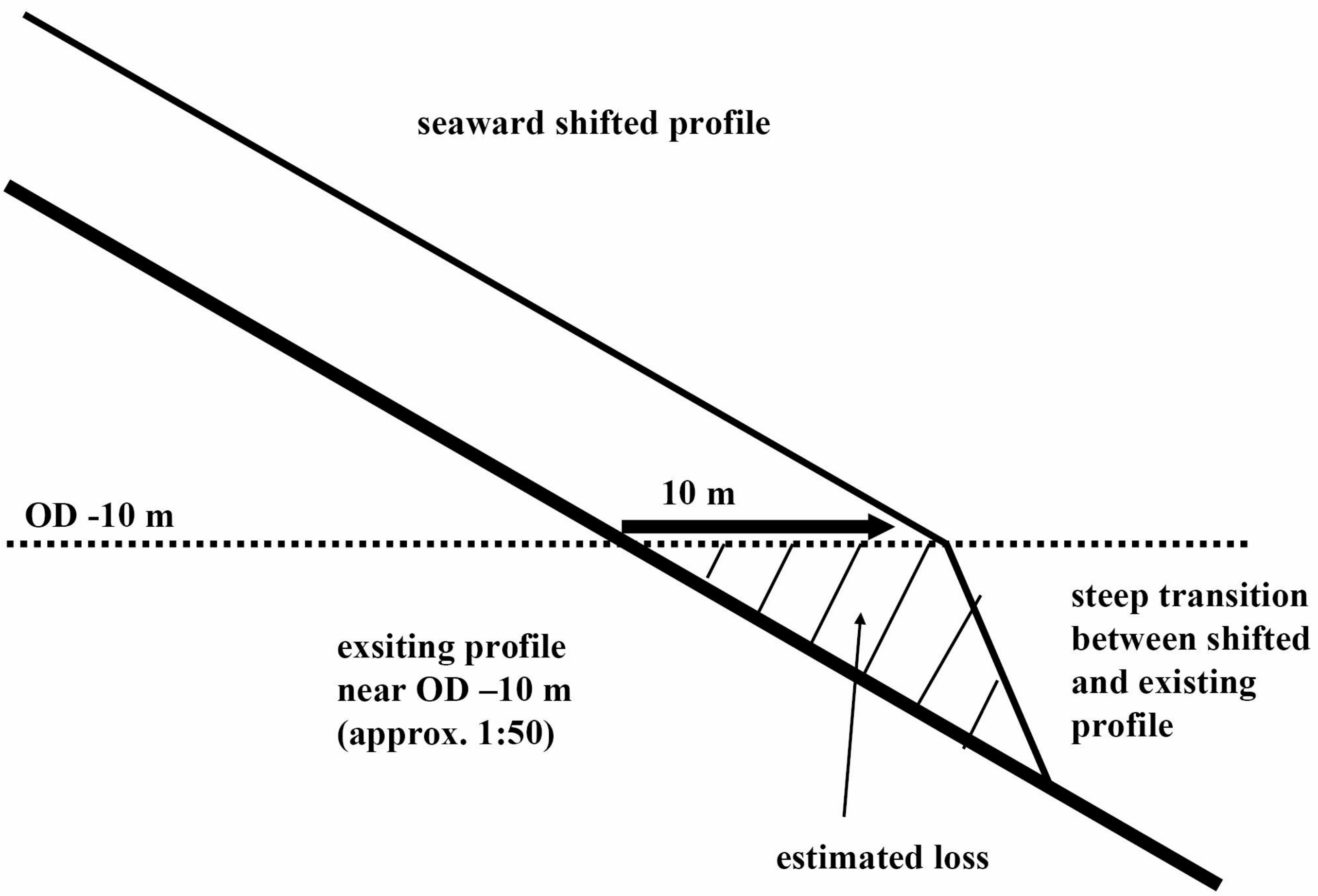
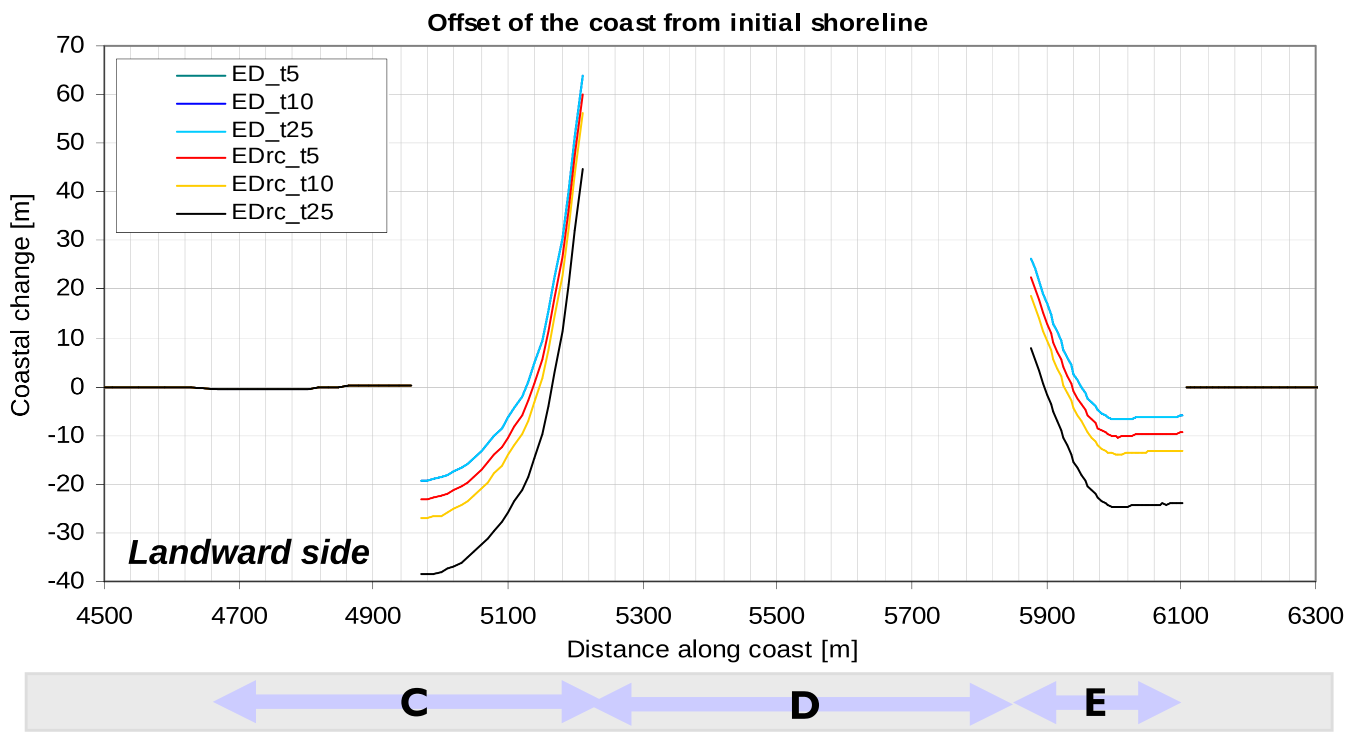
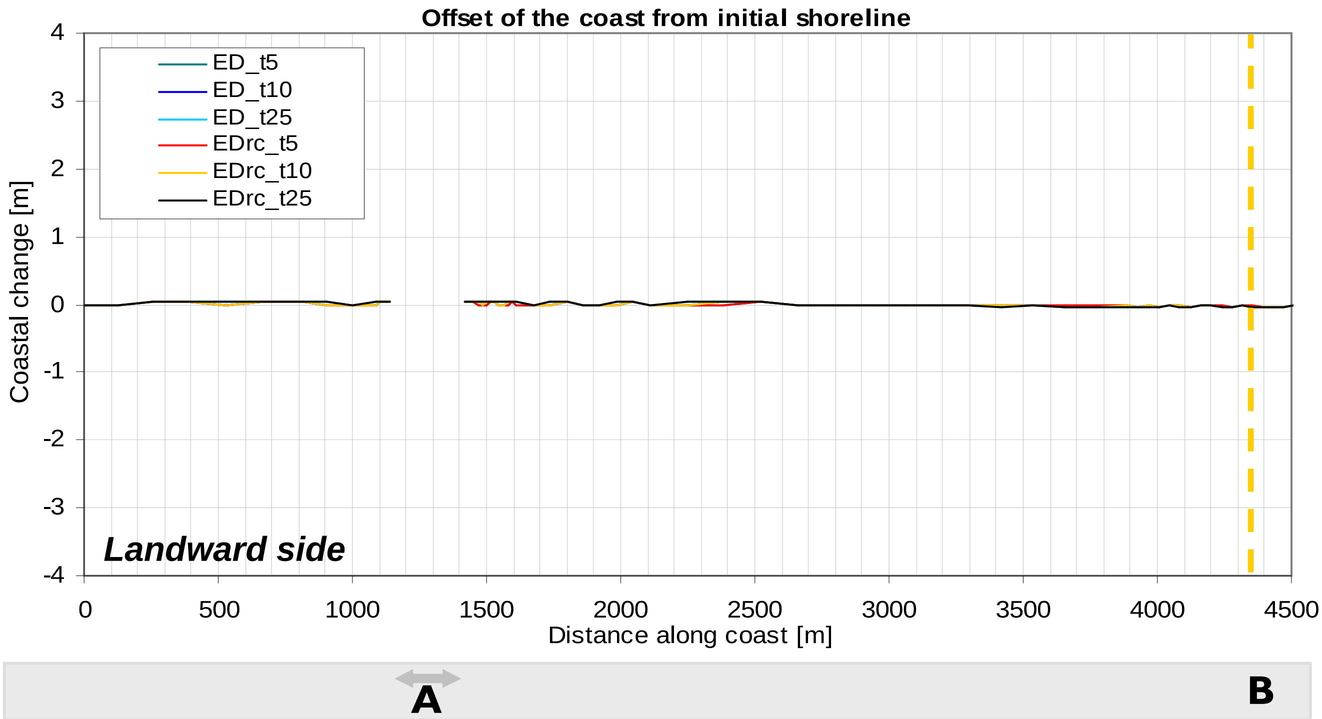


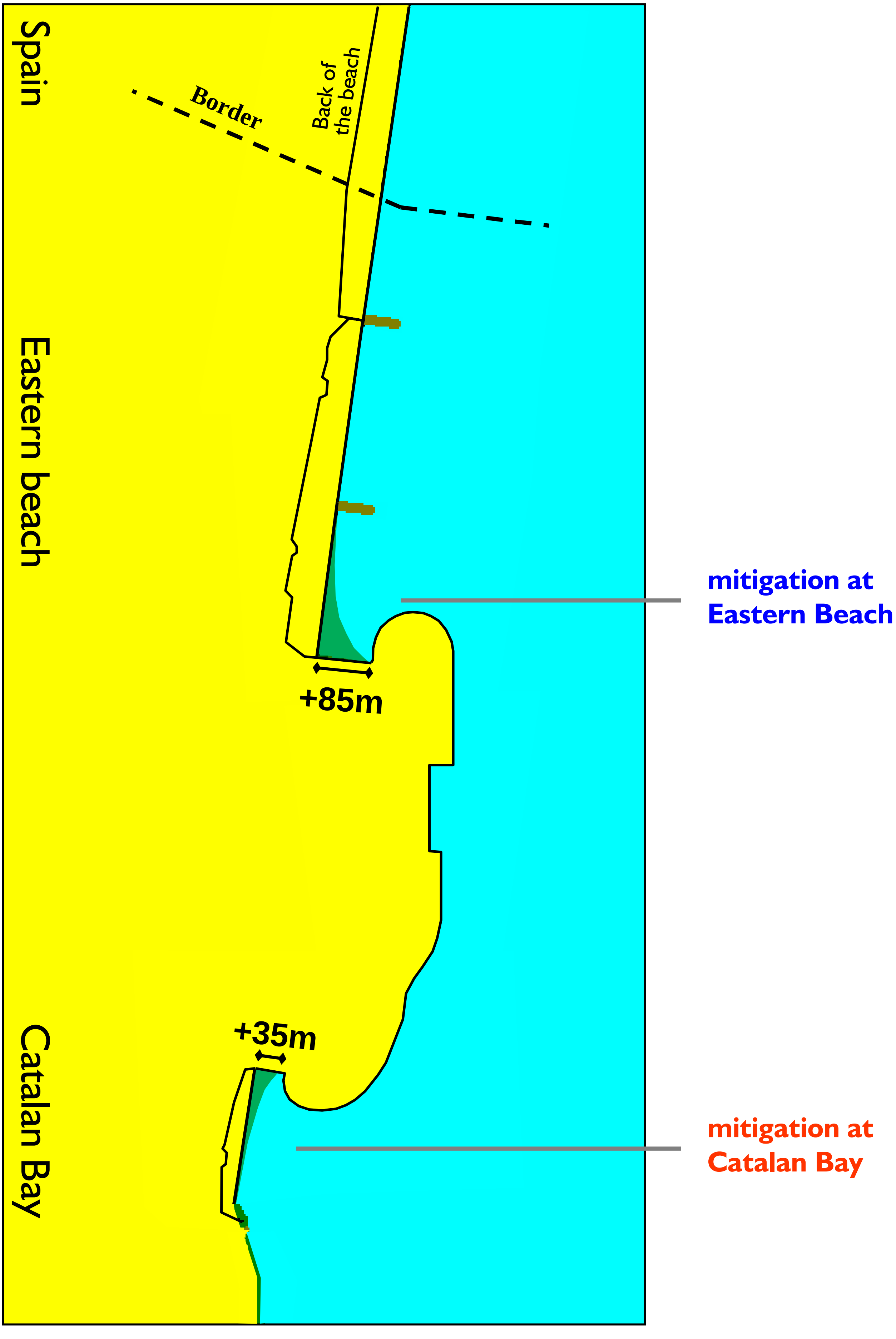
Illustration of offshore loss over OD -10m contour





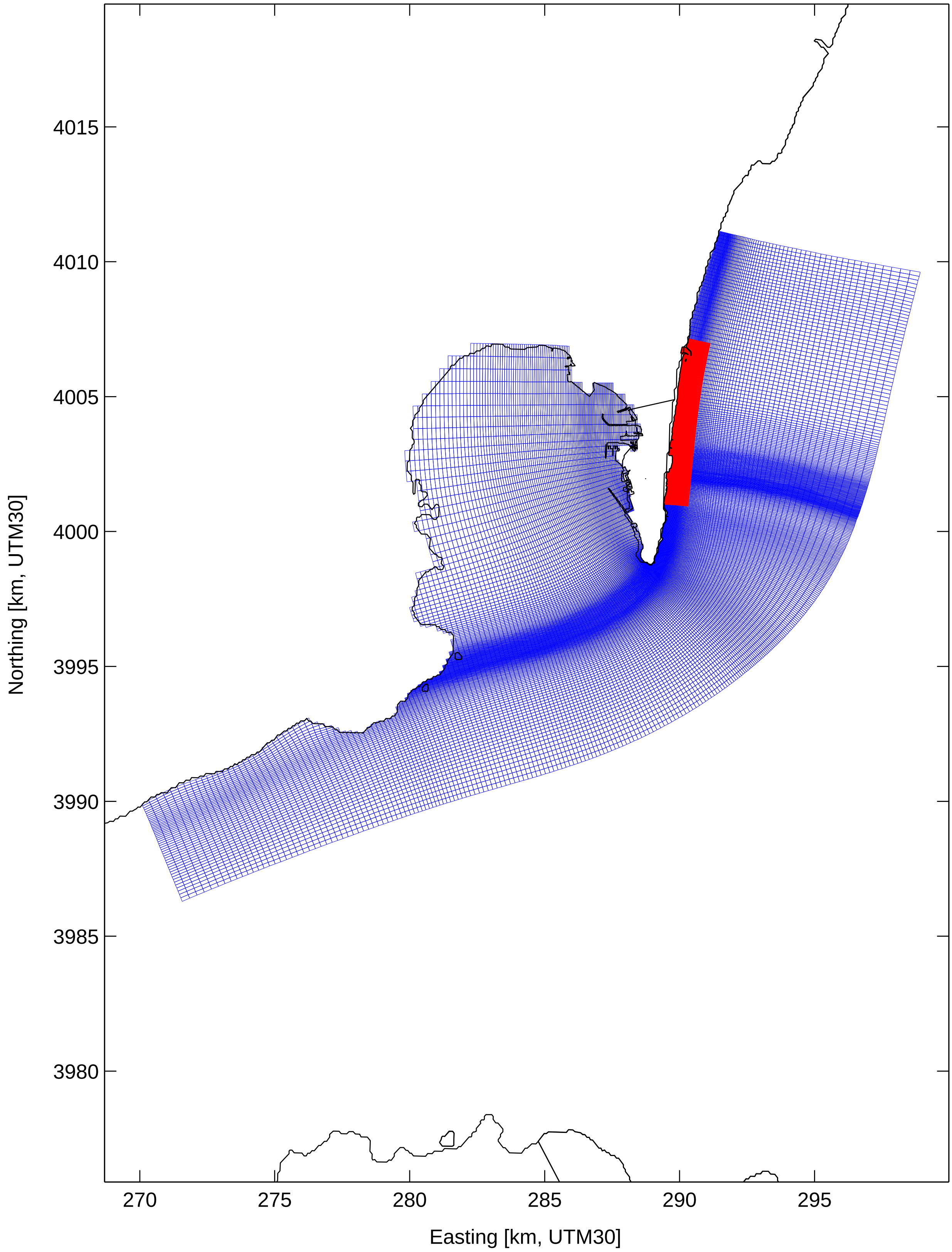
A. La Atunara harbour
B. Spanish border
C. Eastern Beach
D. 'Rubble Tip'
E. Catalan Bay

Coastal impact (after 5, 10 and 25 years) based on longshore and cross-shore effects		
WL | DELFT HYDRAULICS	H4725	Fig. 5.11



Nourished equilibrium beaches

WL | DELFT HYDRAULICS	H4725	Fig. 5.12



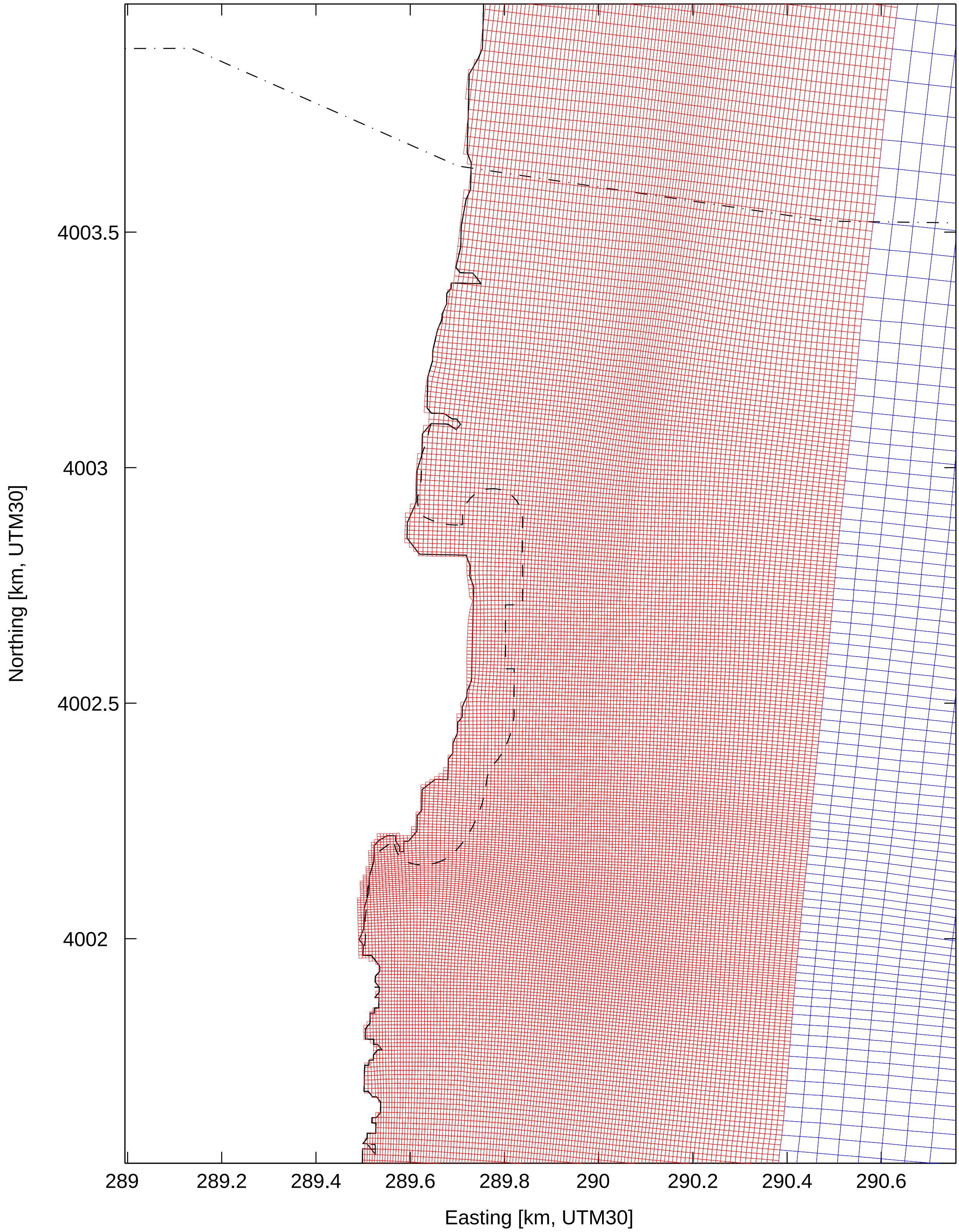
Computational grid for flow model
Blue: overall grid, red: detailed grid

Deltf3D − Morphology

WL | DELFT HYDRAULICS

H4725

Fig. 6.1a



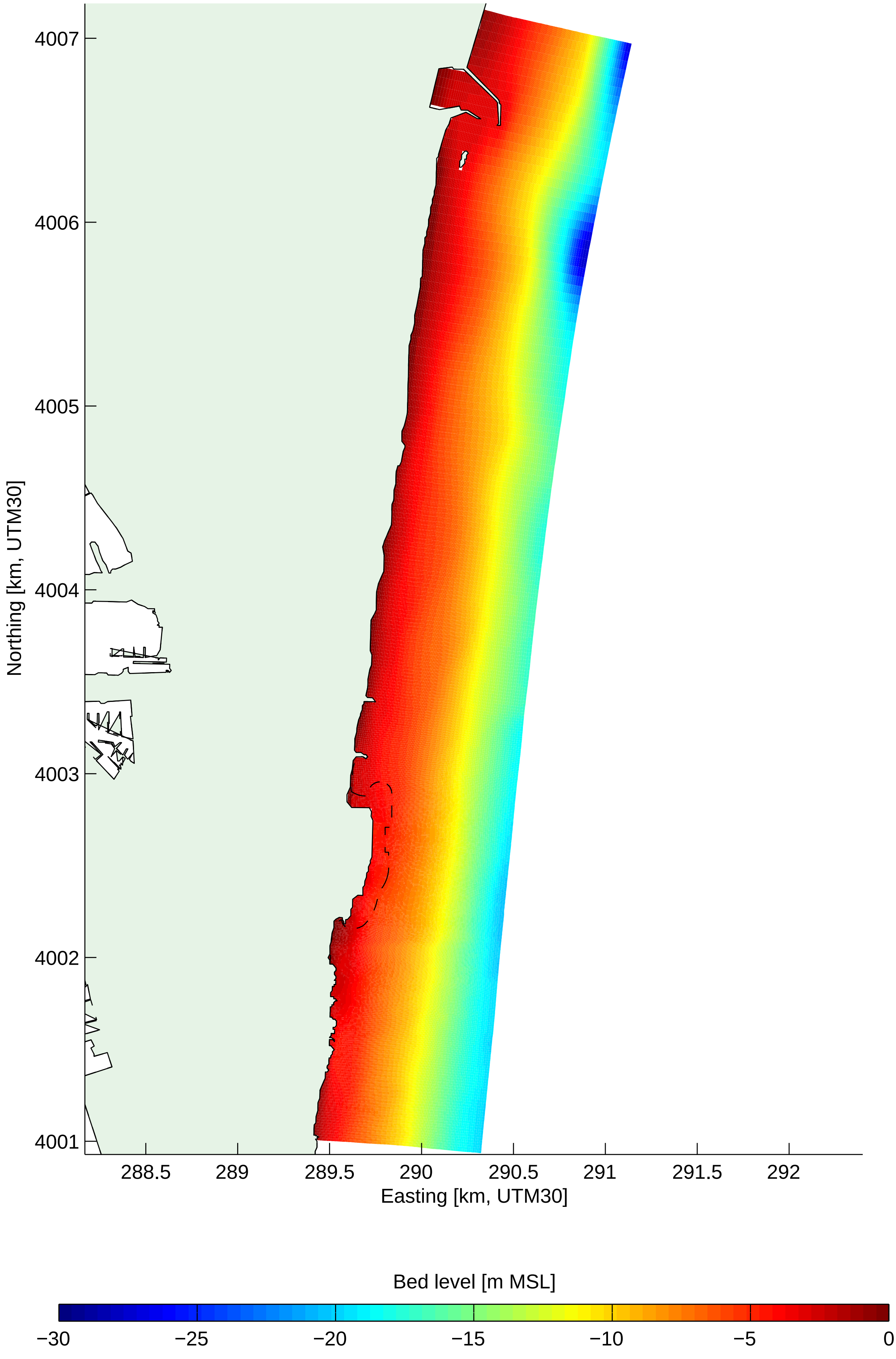
Computational grid for flow model
Detail of the grids around the rubble tip

Deltf3D − Morphology

WL | DELFT HYDRAULICS

H4725

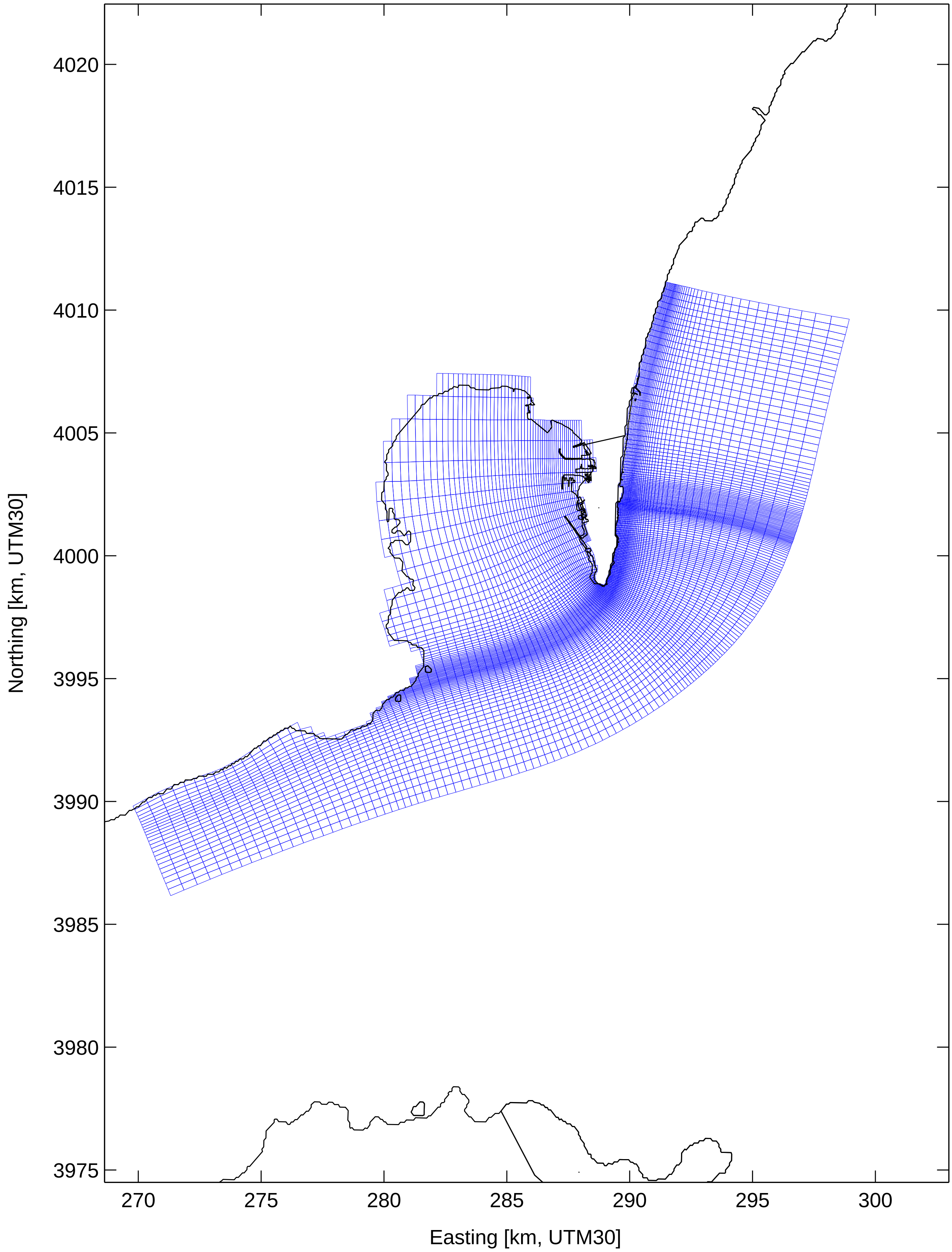
Fig. 6.1b



Bathymetry for the detailed domain of the flow model

Deltf3D − Morphology

WL | Delft Hydraulics | H4725 | Fig. 6.2



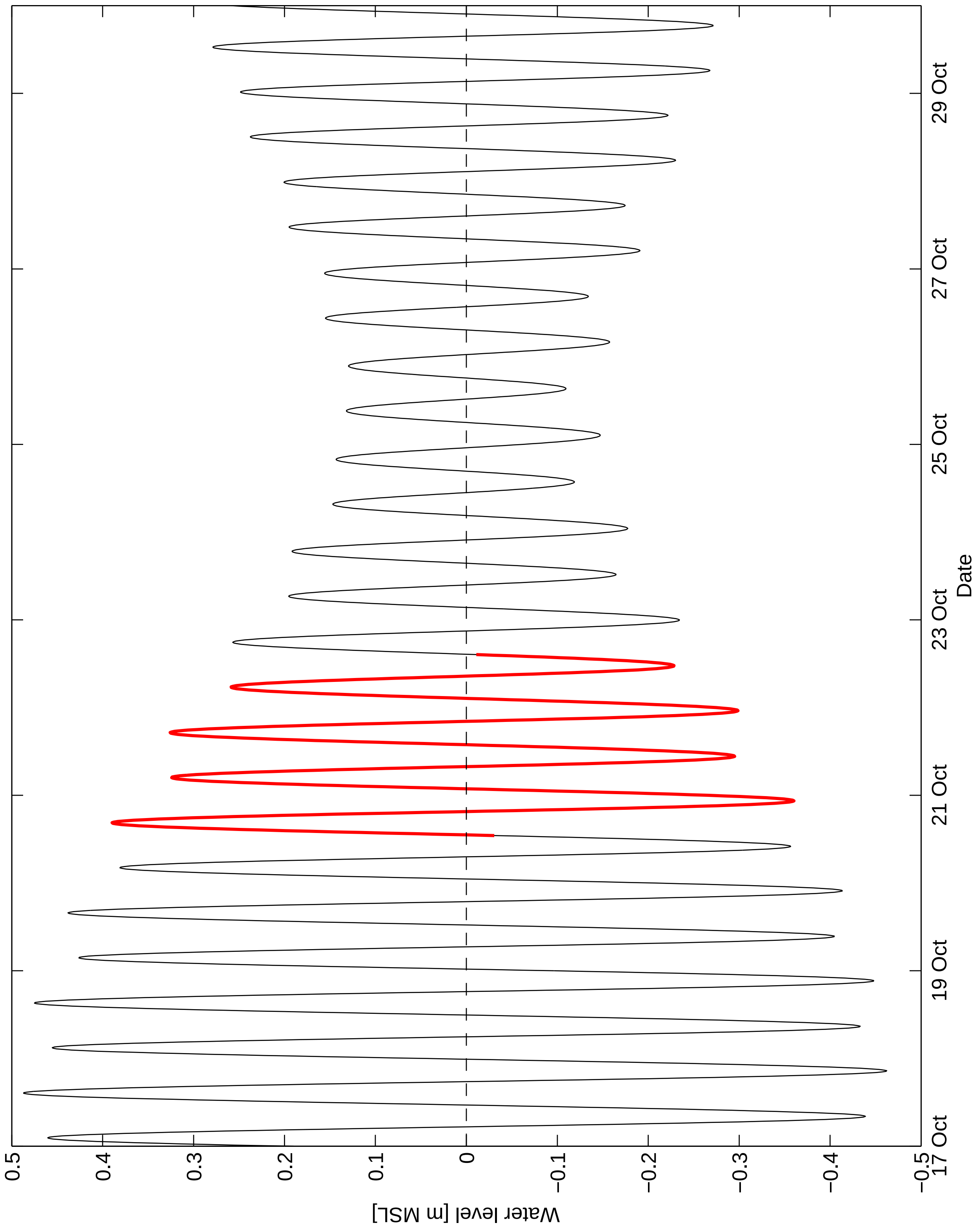
Computational grid for wave model

Delft3D − Morphology

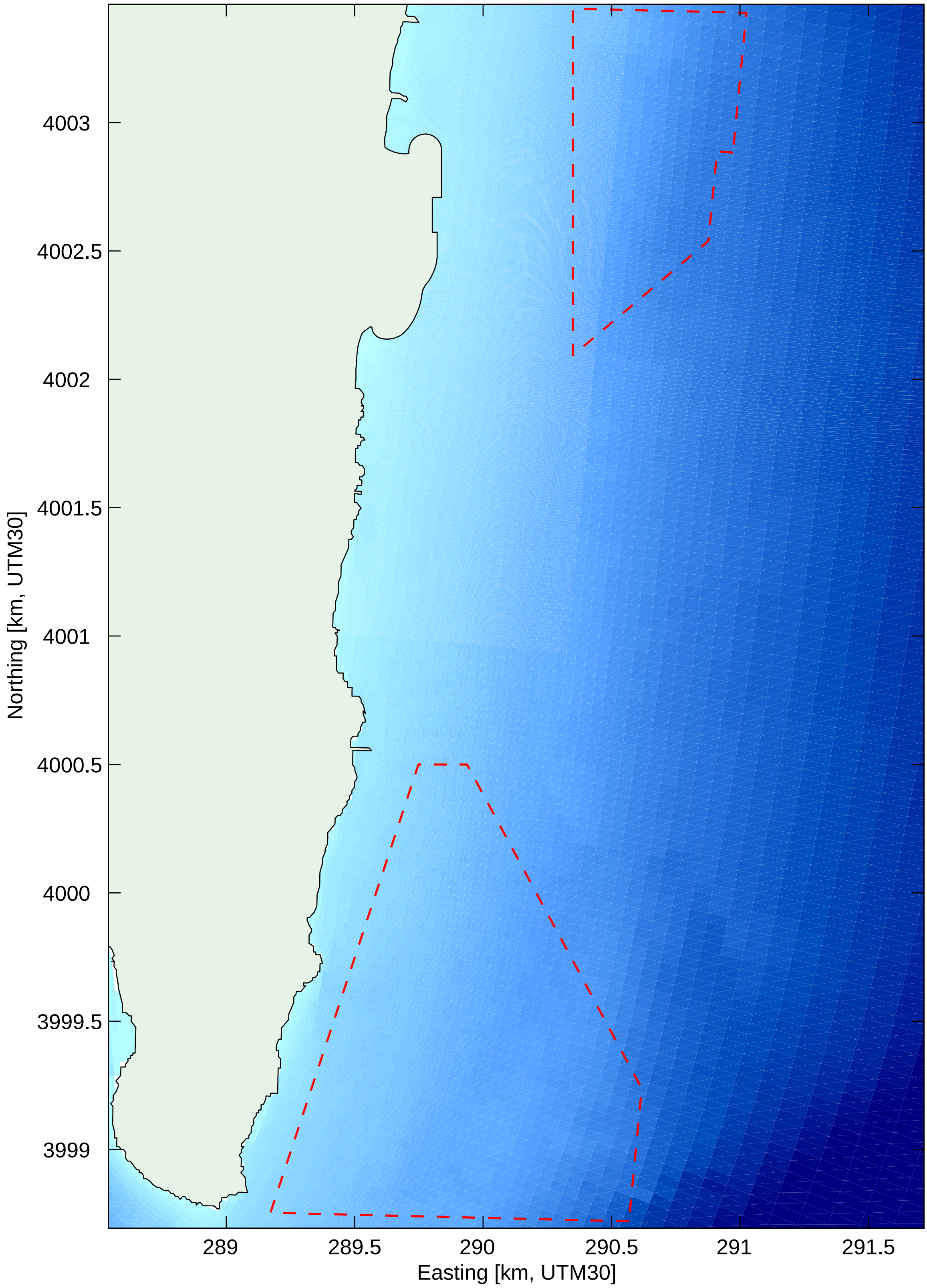
WL | DELFT HYDRAULICS

H4725

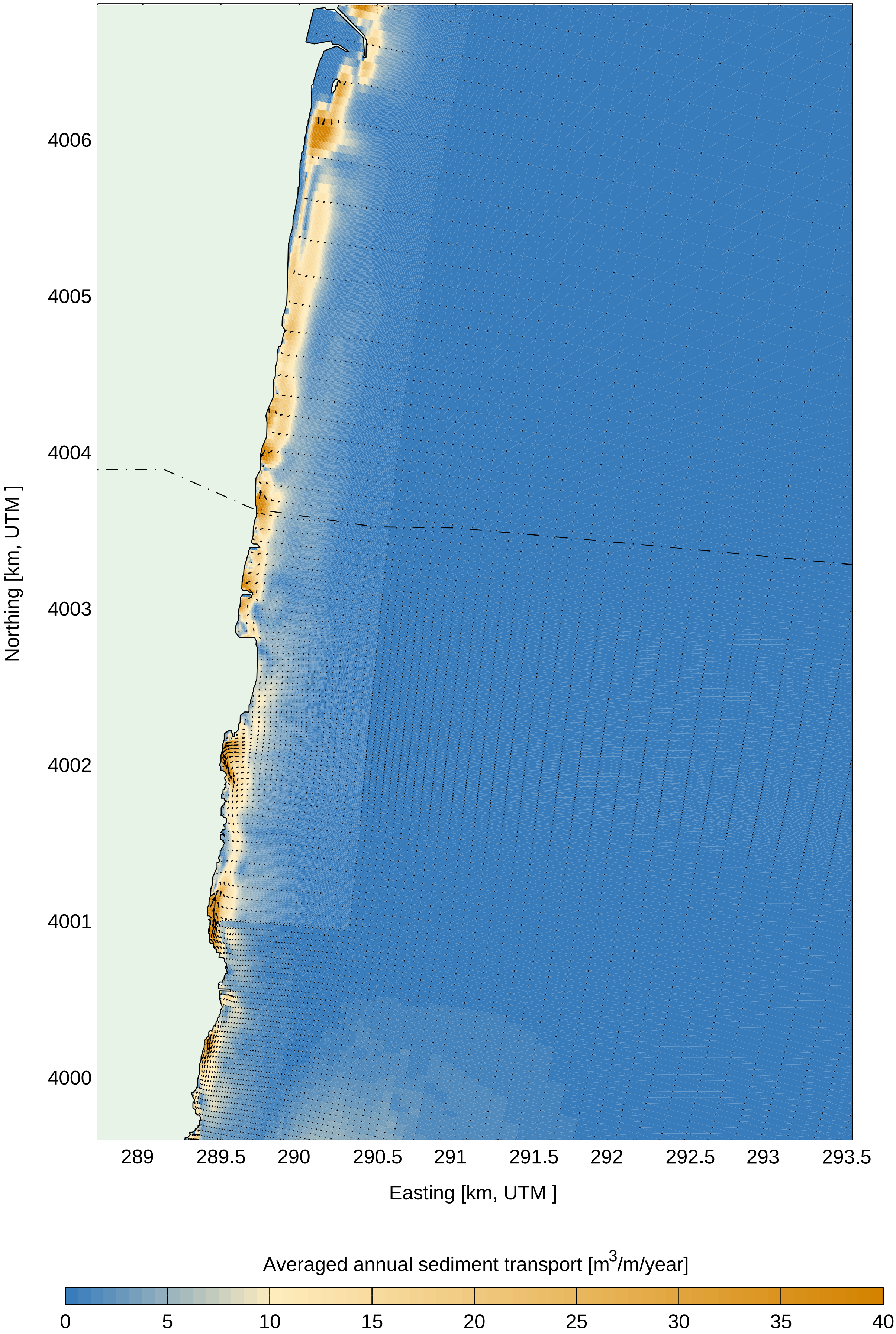
Fig. 6.3



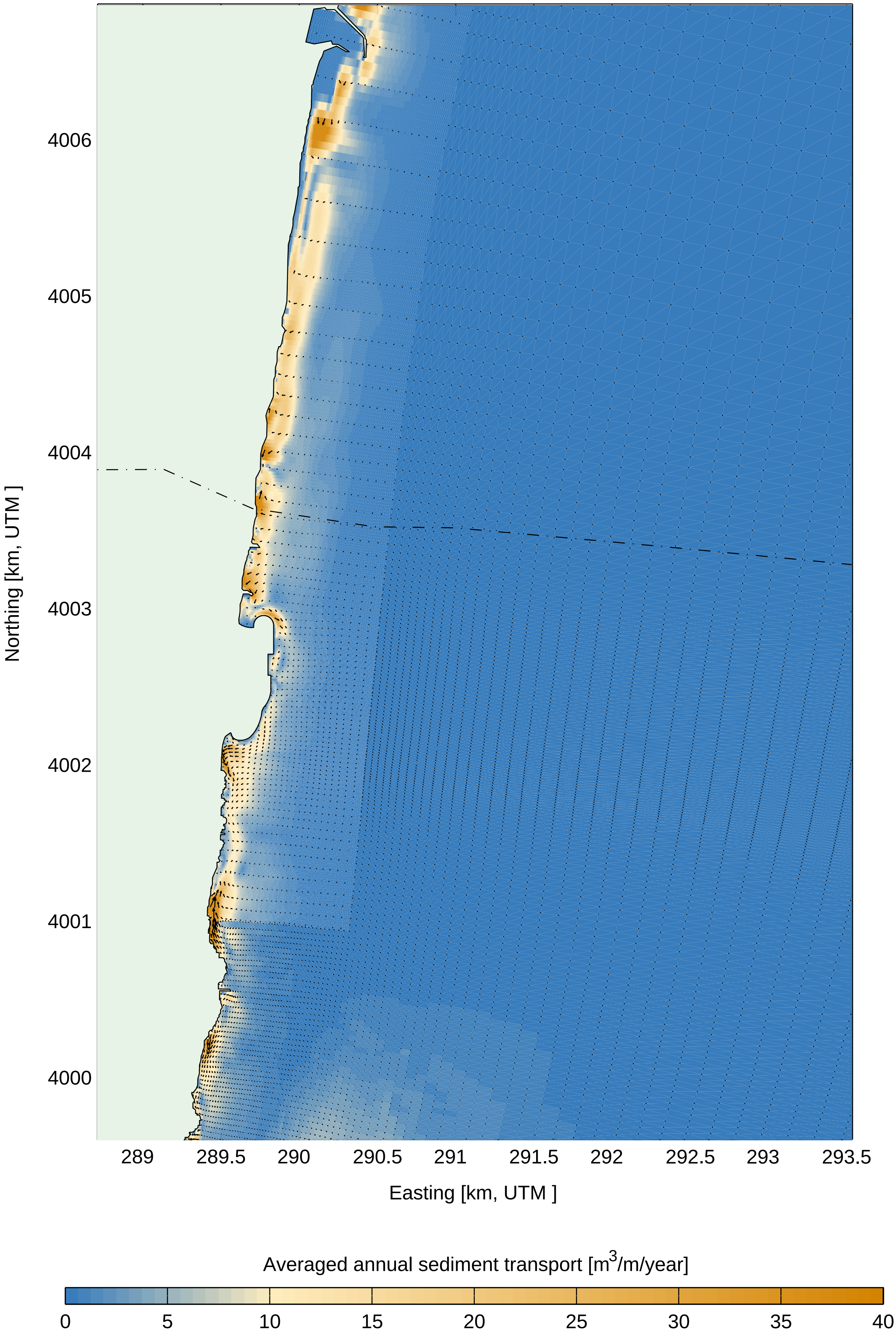
Selection of morphological tide (in red)		
	Delft3D − Morphology	
WL | Delft Hydraulics	H4725	Fig. 6.4



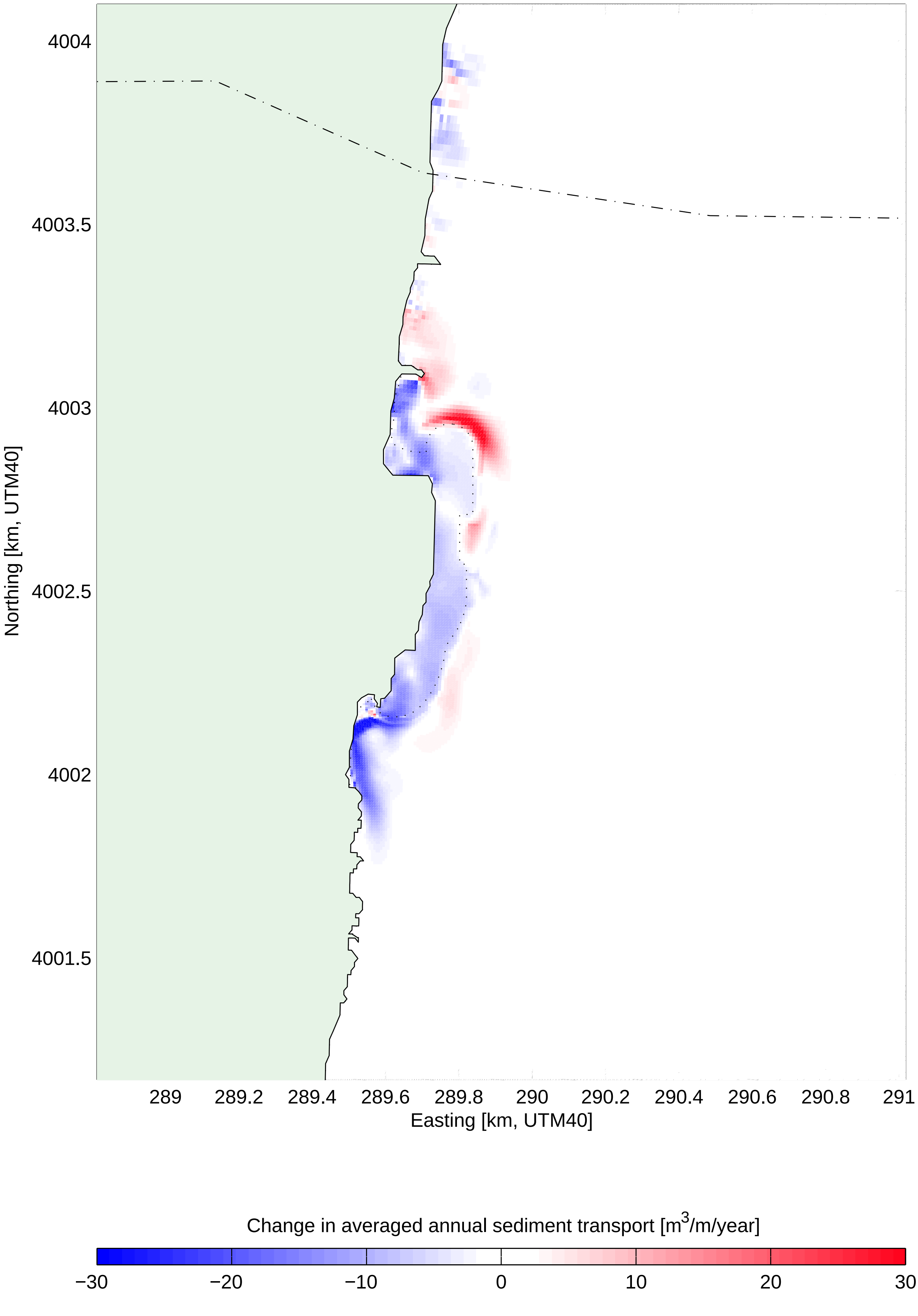
Layout 2nd scenario, borrow areas are indicated by red polygons	Delft3D − Morphology	
WL | Delft Hydraulics	H4725	Fig. 6.5



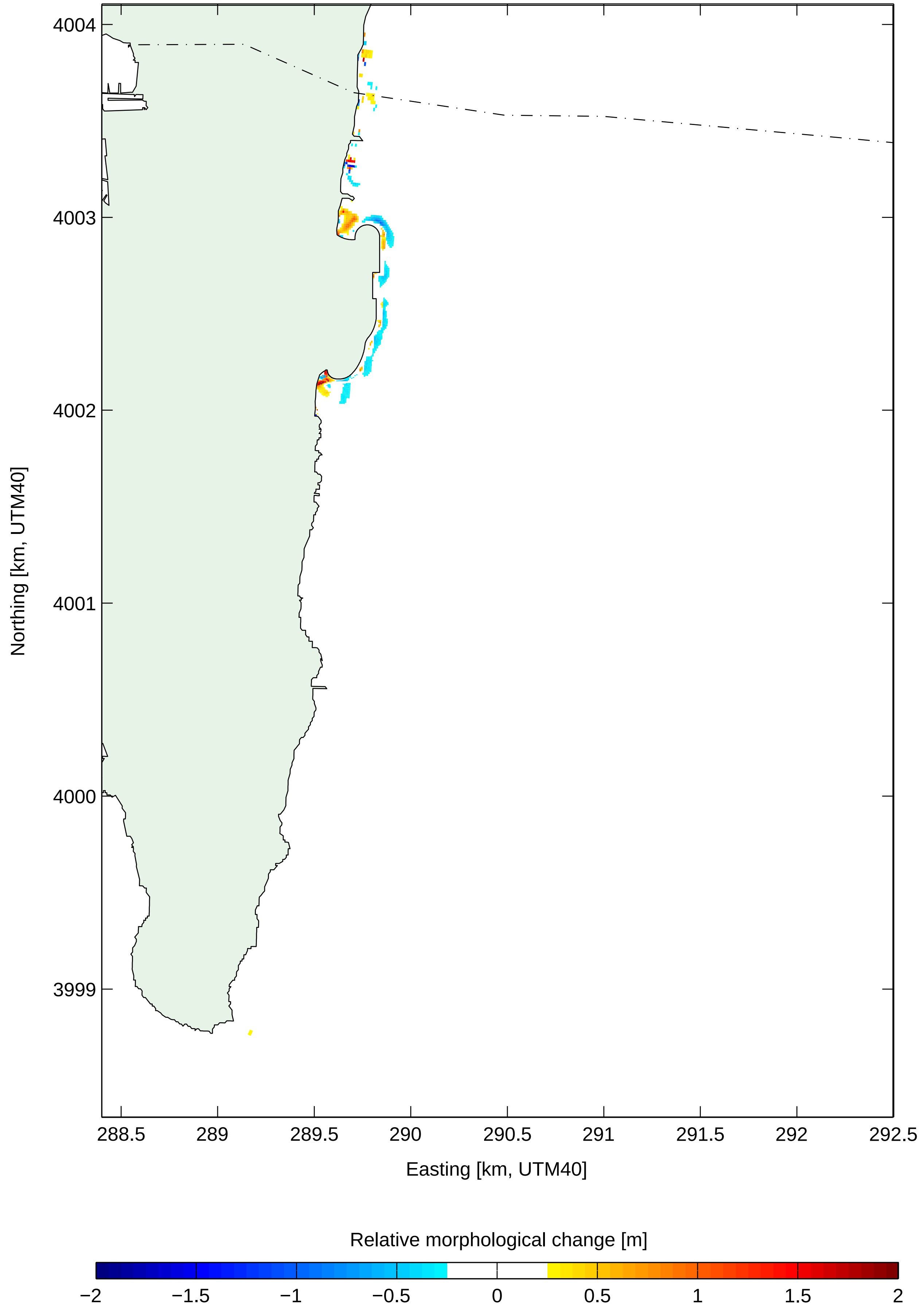
Averaged annual sediment transport Situation without development	RunID: Run p05	
	Delft3D − Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.6a



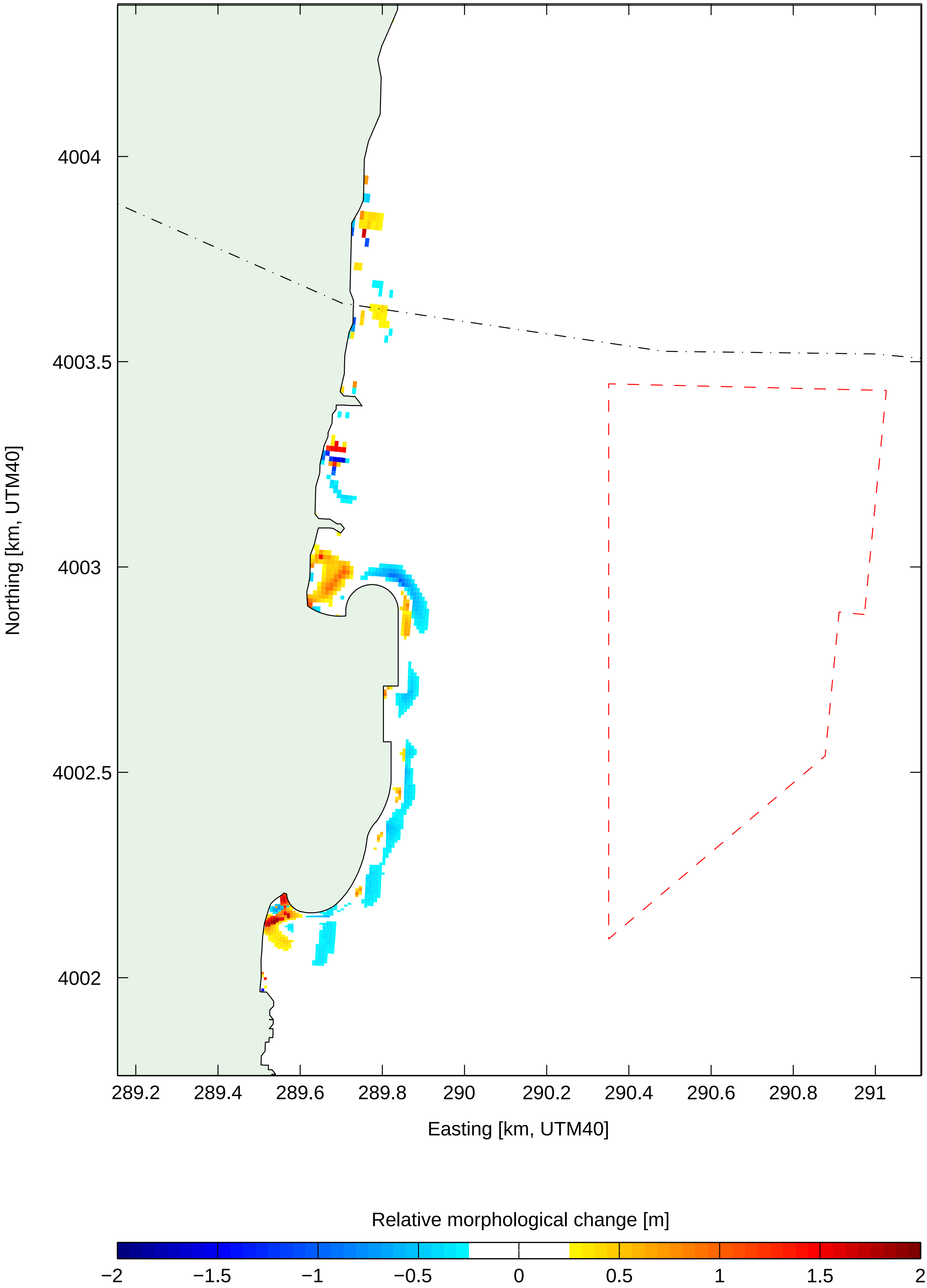
Averaged annual sediment transport Situation with development	RunID: Run p09	
	Delft3D−Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.6b



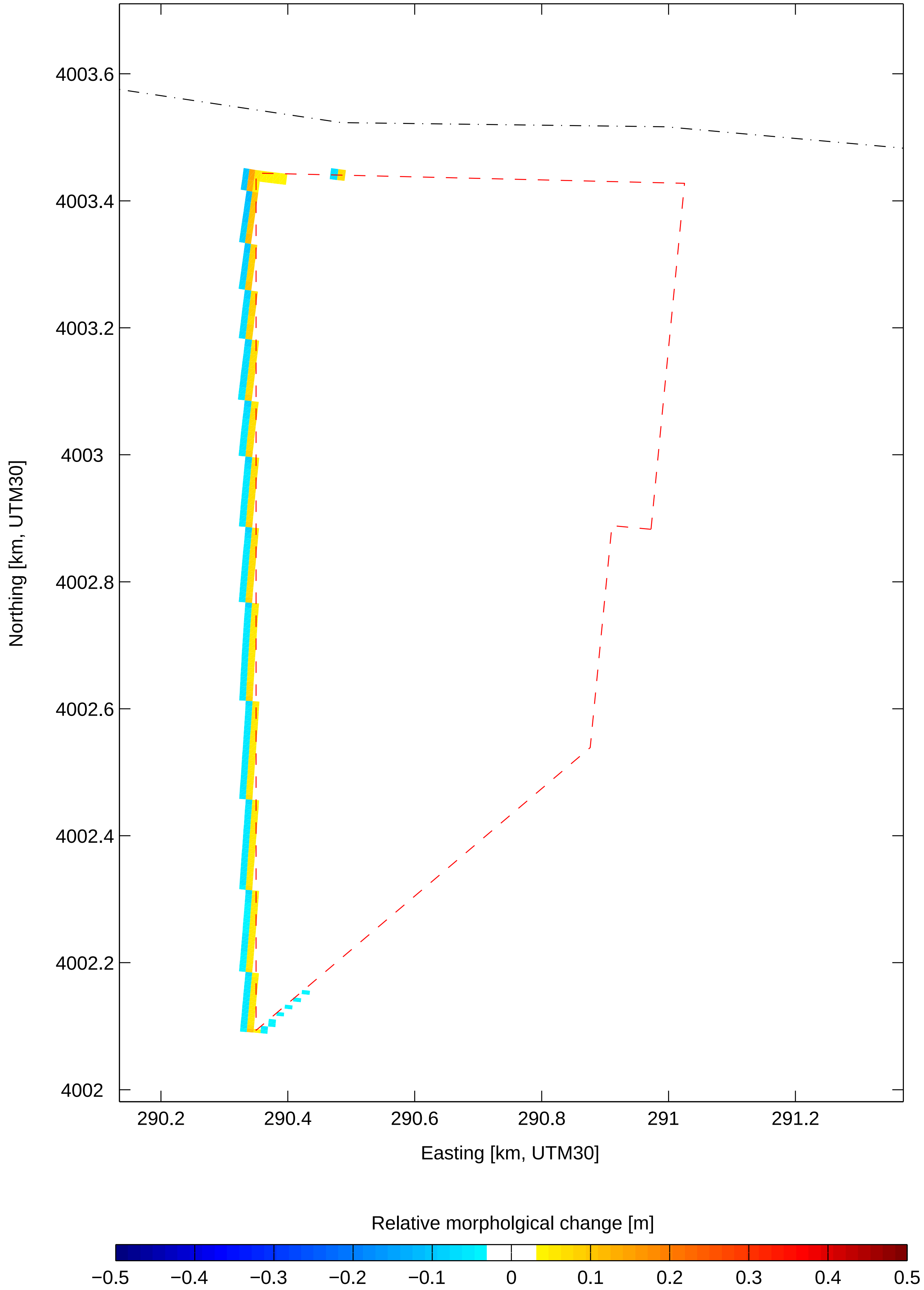
Change in averaged annual sediment transport (transports situation with development− situation without development)	RunID: p09−p05	
	Delft3D − Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.6c



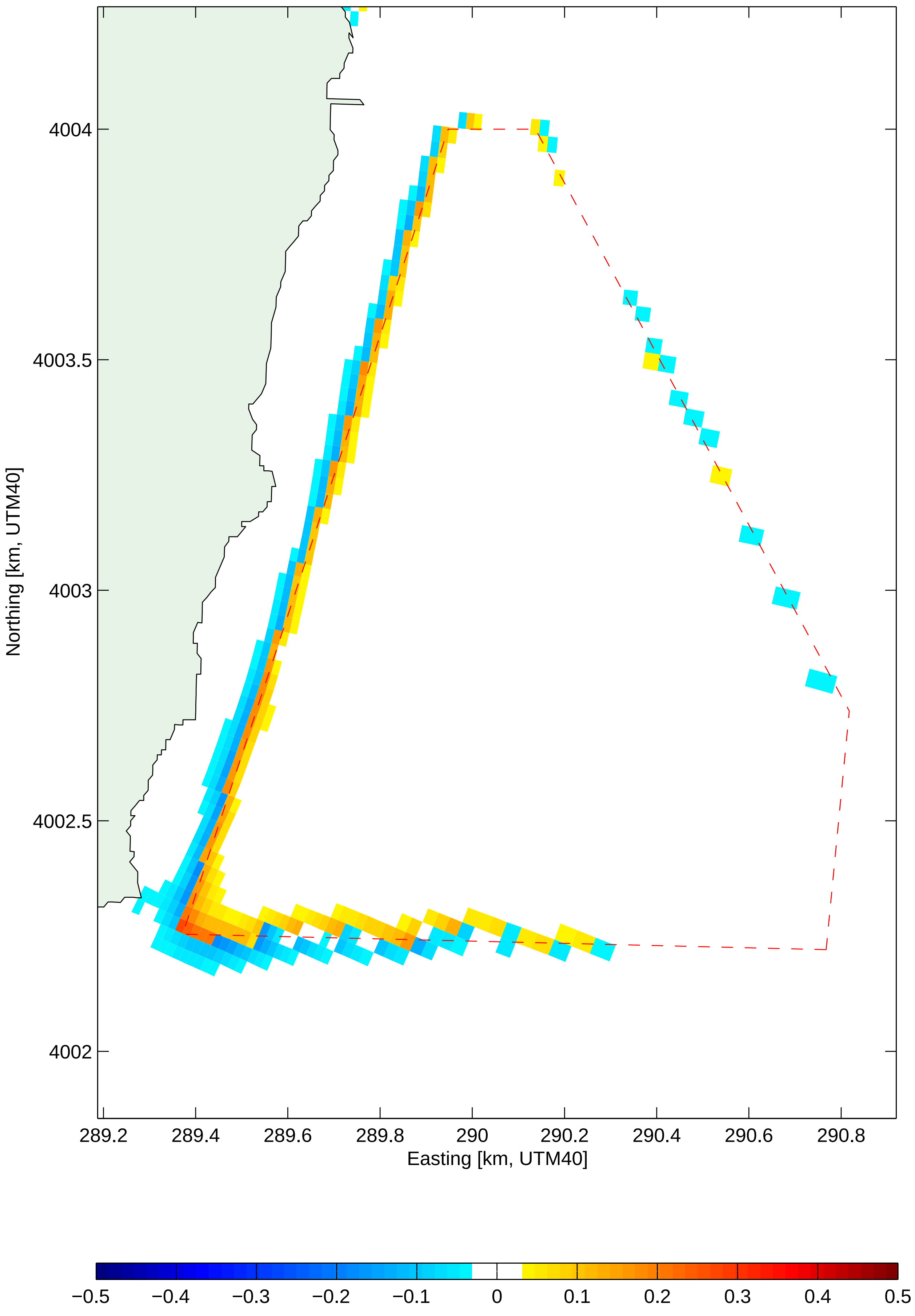
Relative morphological change after 1 year due to development and borrow pits (= bed level changes with development − situation without development)	RunID: p09r−p05	
	Delft3D−Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.7a



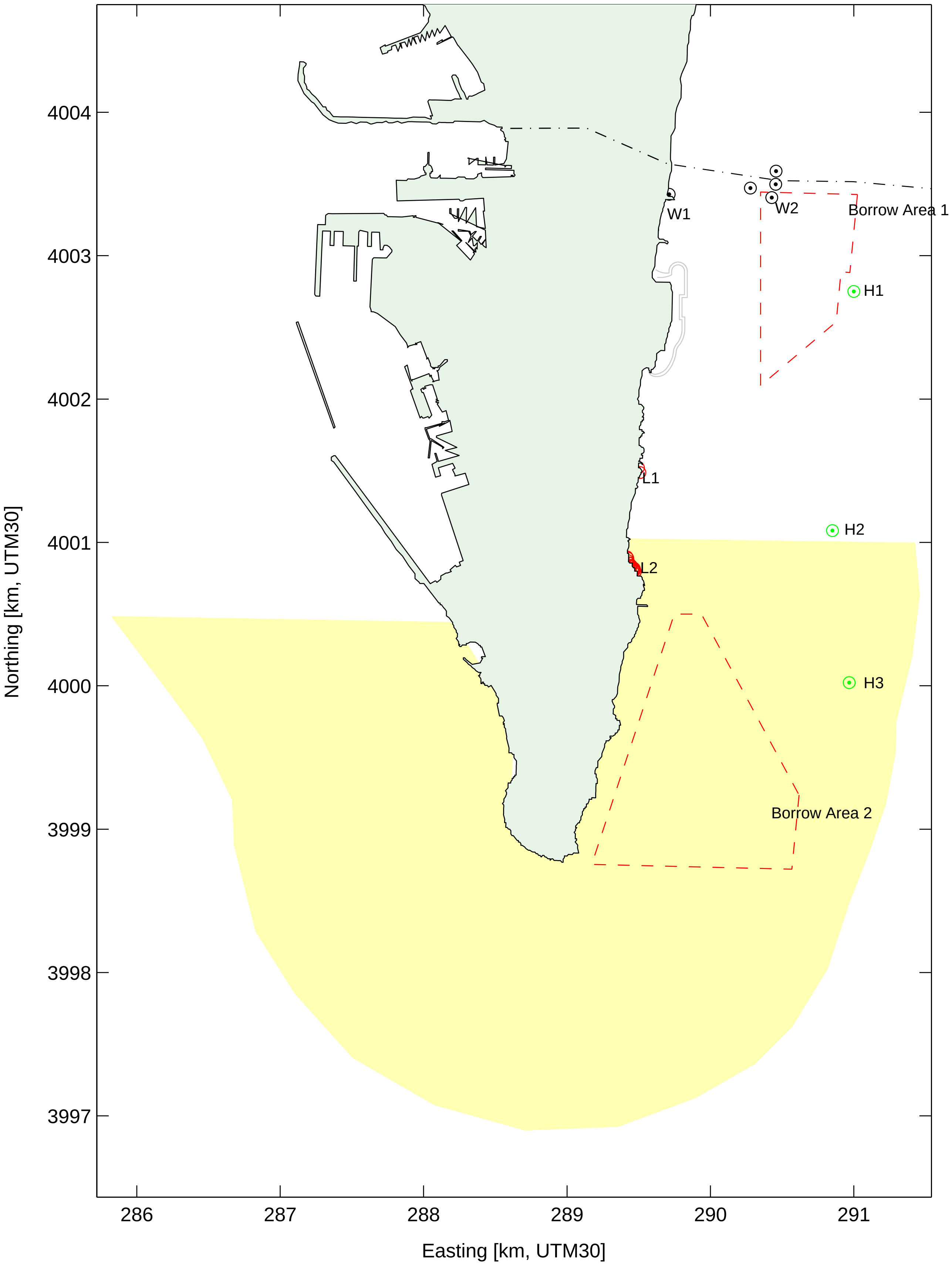
Relative morphological change after 1 year, around development and Spanish border (= bed level changes with development − situation without development)	RunID: p09r−p05	
	Delft3D−Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.7b



Relative morphological change after 1 year, around northern borrow area (= bed level changes with development − situation without development)	RunID: p08−p05	
	Delft3D−Morphology	
WL | Delft Hydraulics	H4725	Fig. 6.7c

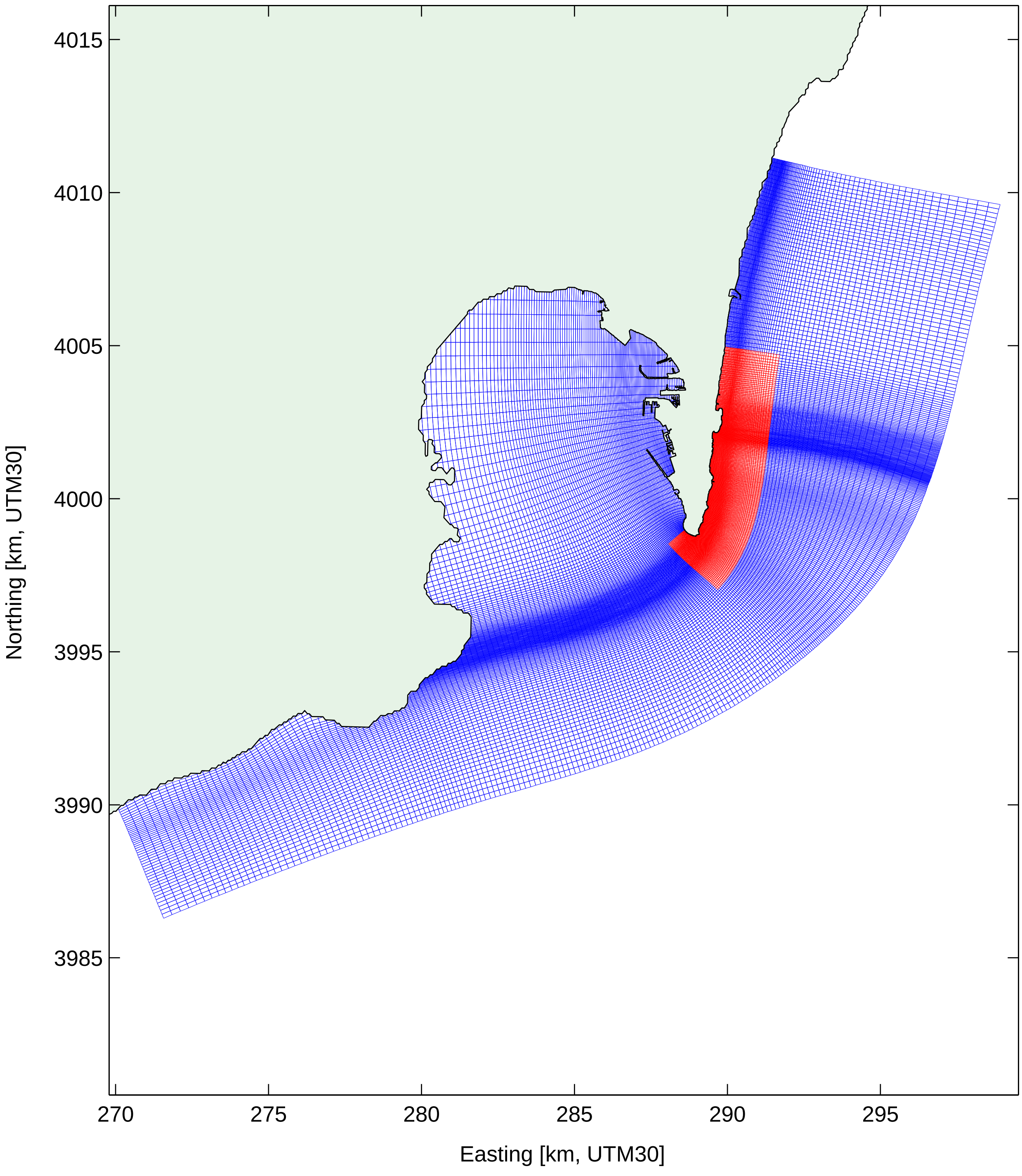


Relative morphological change after 1 year, around southern borrow area (= bed level changes with development − situation without development)	RunID: p09r−p05	
	Delft3D−Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 6.7d



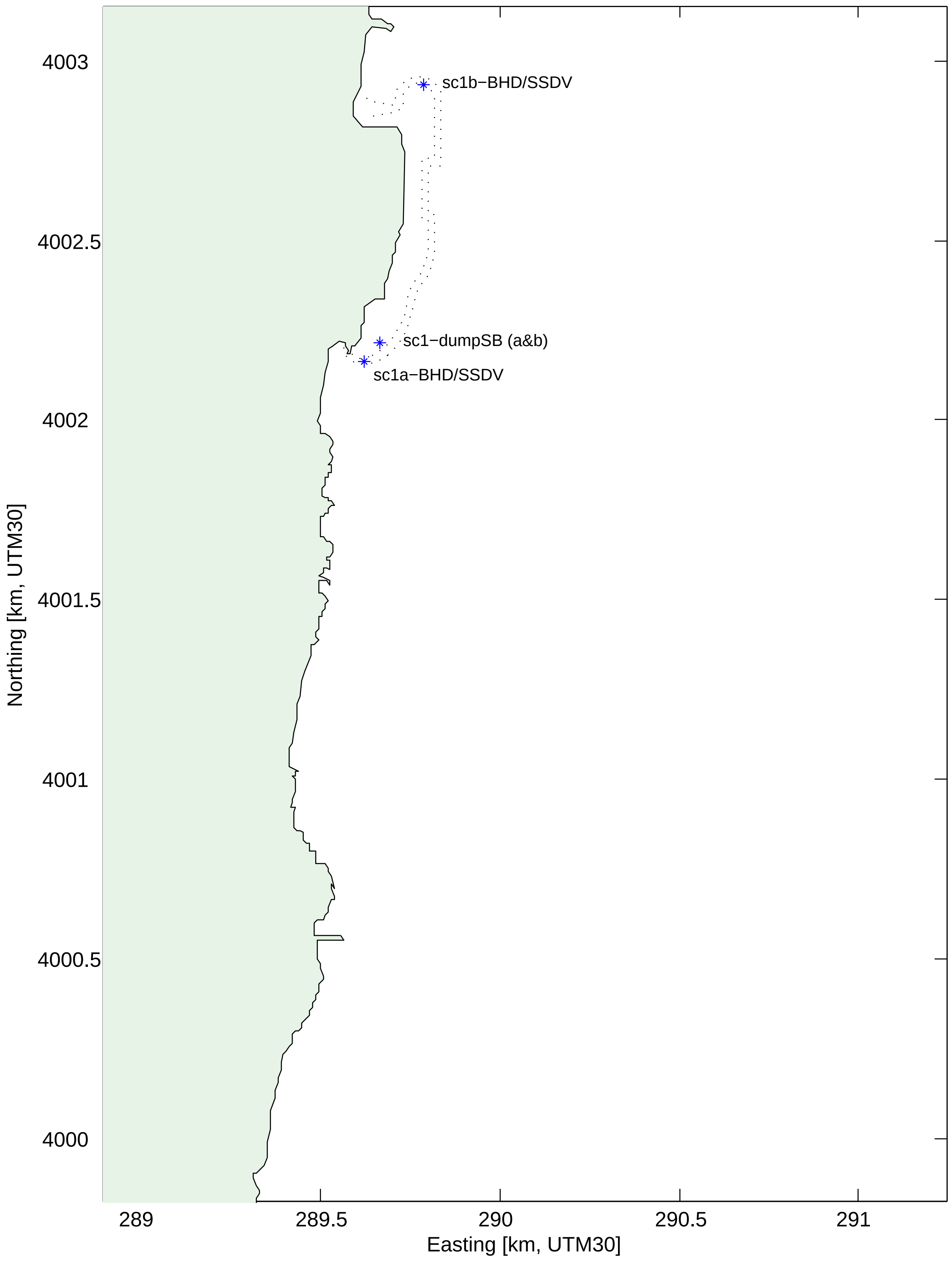
The red, green and black spots indicate respectively the limpets (L), the habitats (H) and the wreck sites (W), the yellow shading indicates the Gibraltar Marina Nature Reserve. Red dashed area represent the borrow areas.

WL | DELFT HYDRAULICS	H4725	Fig. 7.1



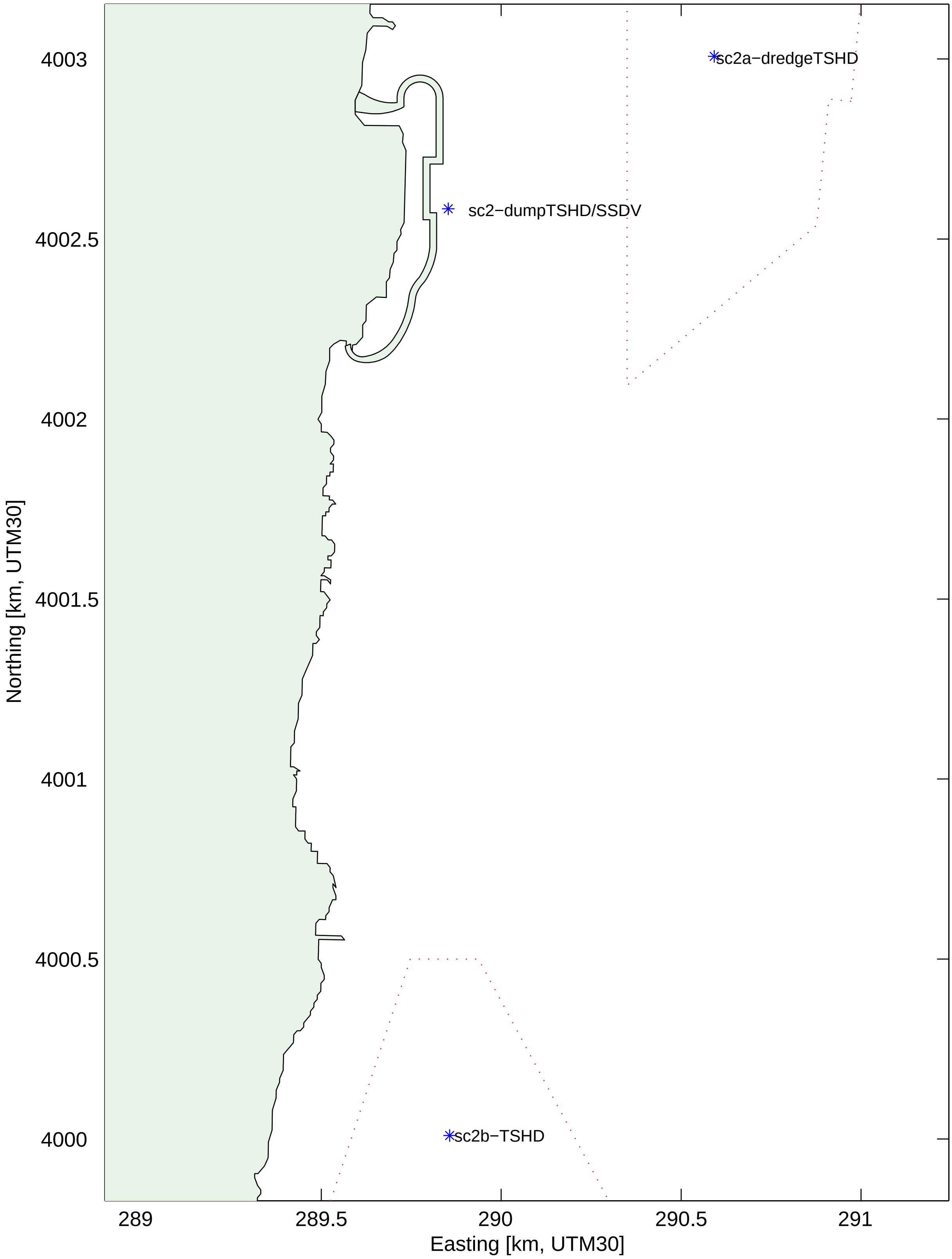
Computational grids for plume dispersion modelling
Blue: overall model grid (2DH), red: 3D grid

	Delft3D	
WL | DELFT HYDRAULICS	H4725	Fig. 7.2



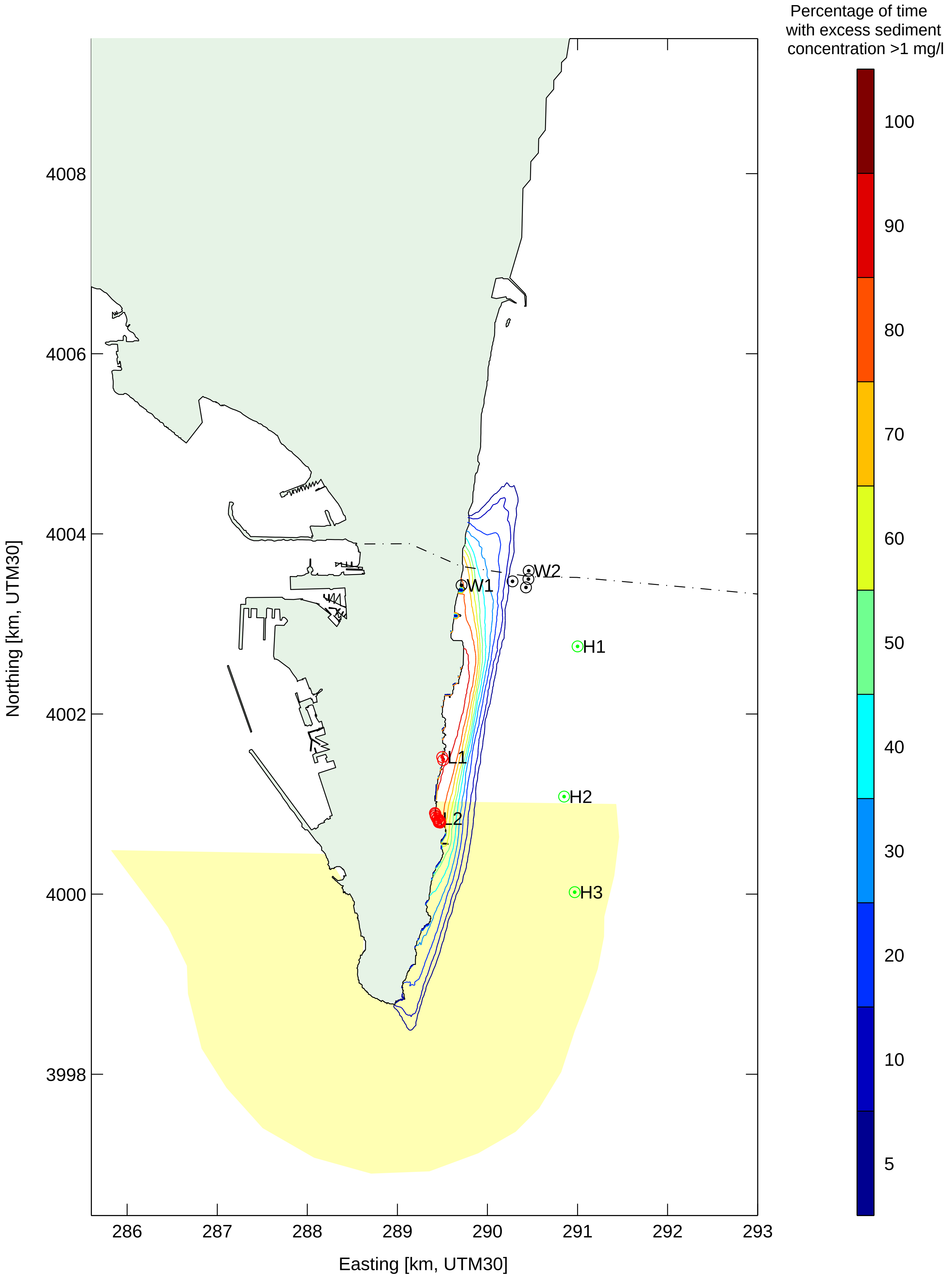
Layout and spill locations for Scenario 1 (a&b)
Areas under construction are indicated by dotted lines.

WL | DELFT HYDRAULICS	H4725	Fig. 7.3a

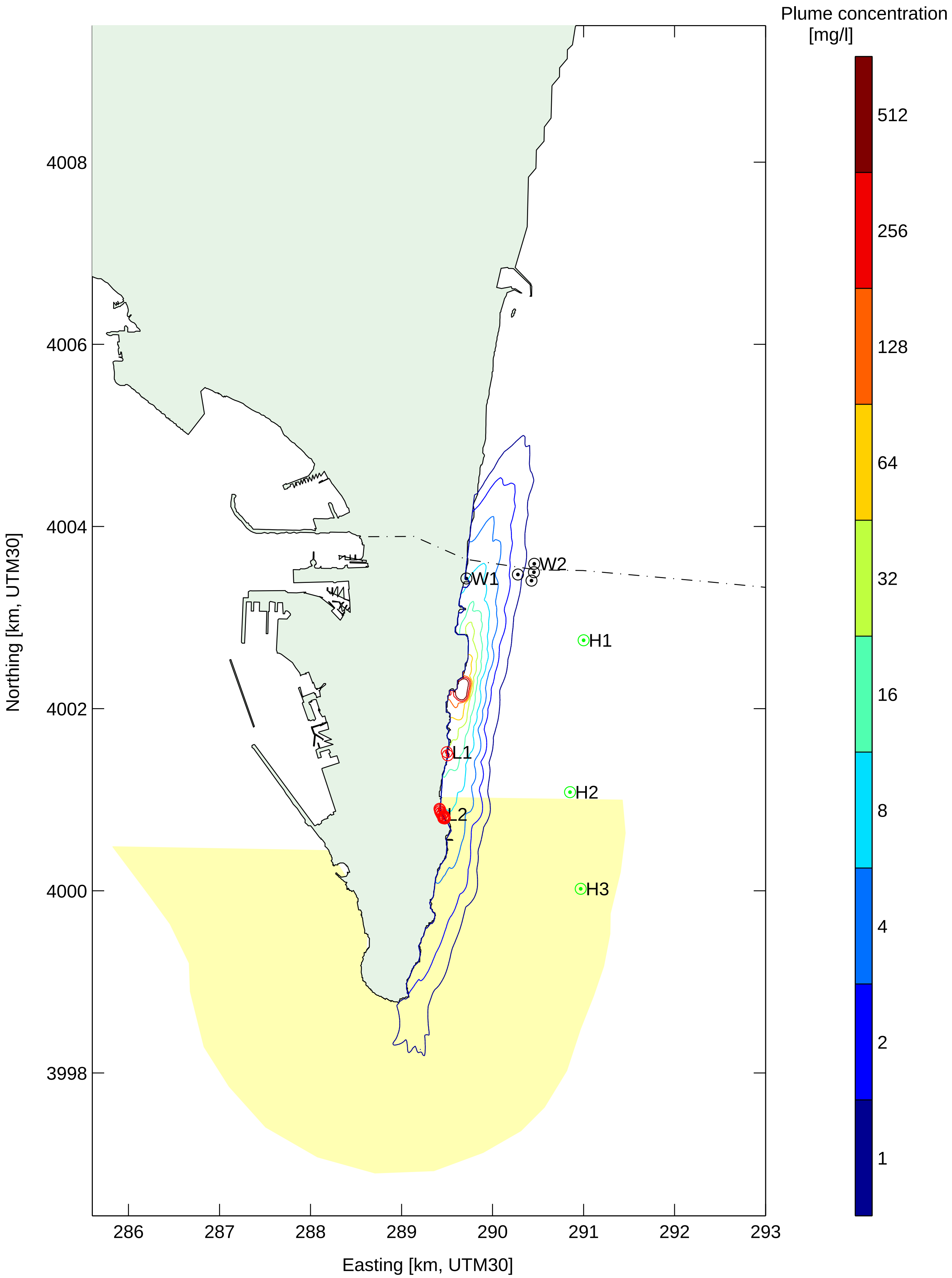


Layout and spill locations for Scenario 2 (a&b)
Borrow areas are indicated by red dotted lines.

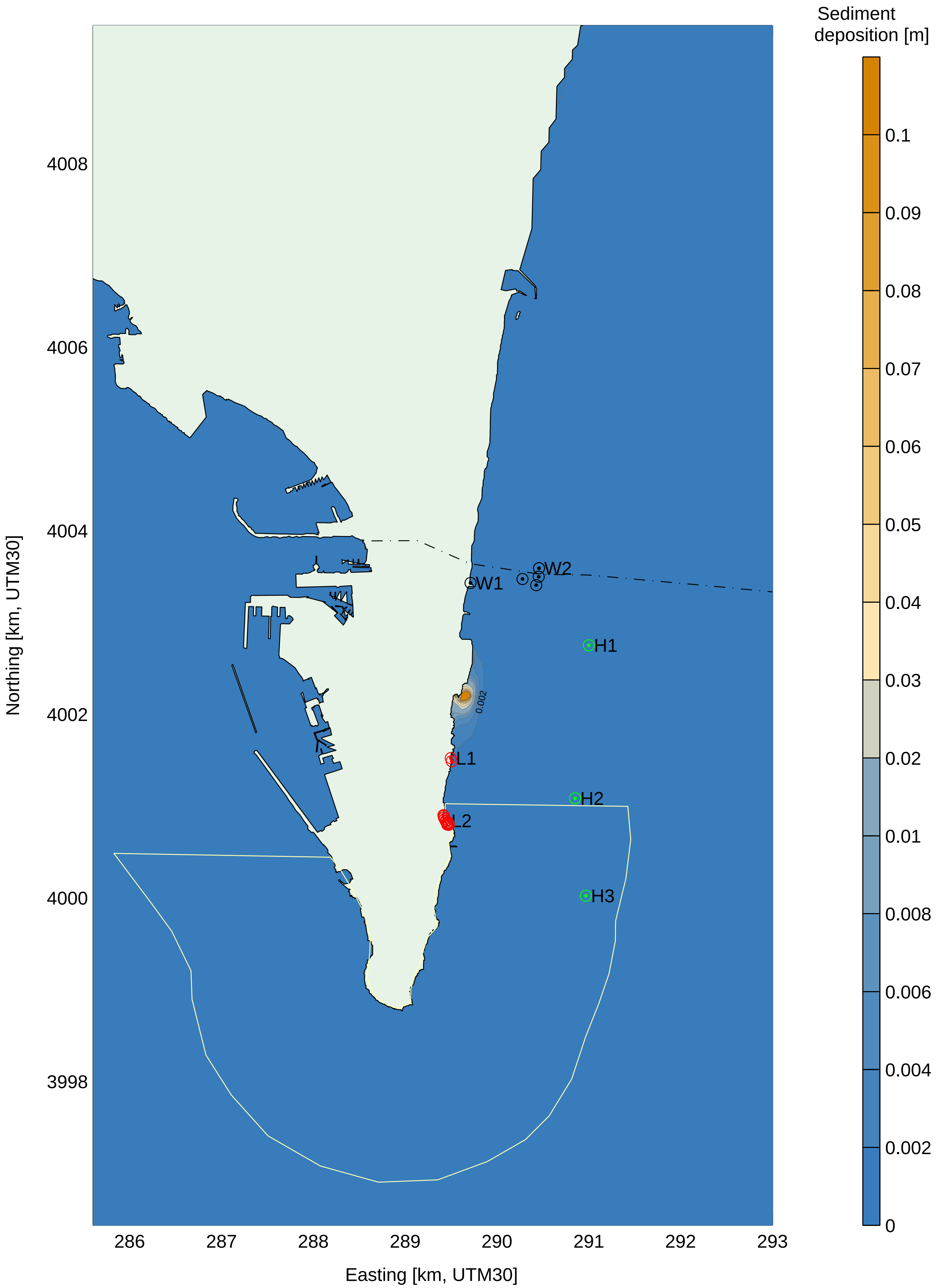
WL | DELFT HYDRAULICS	H4725	Fig. 7.3b



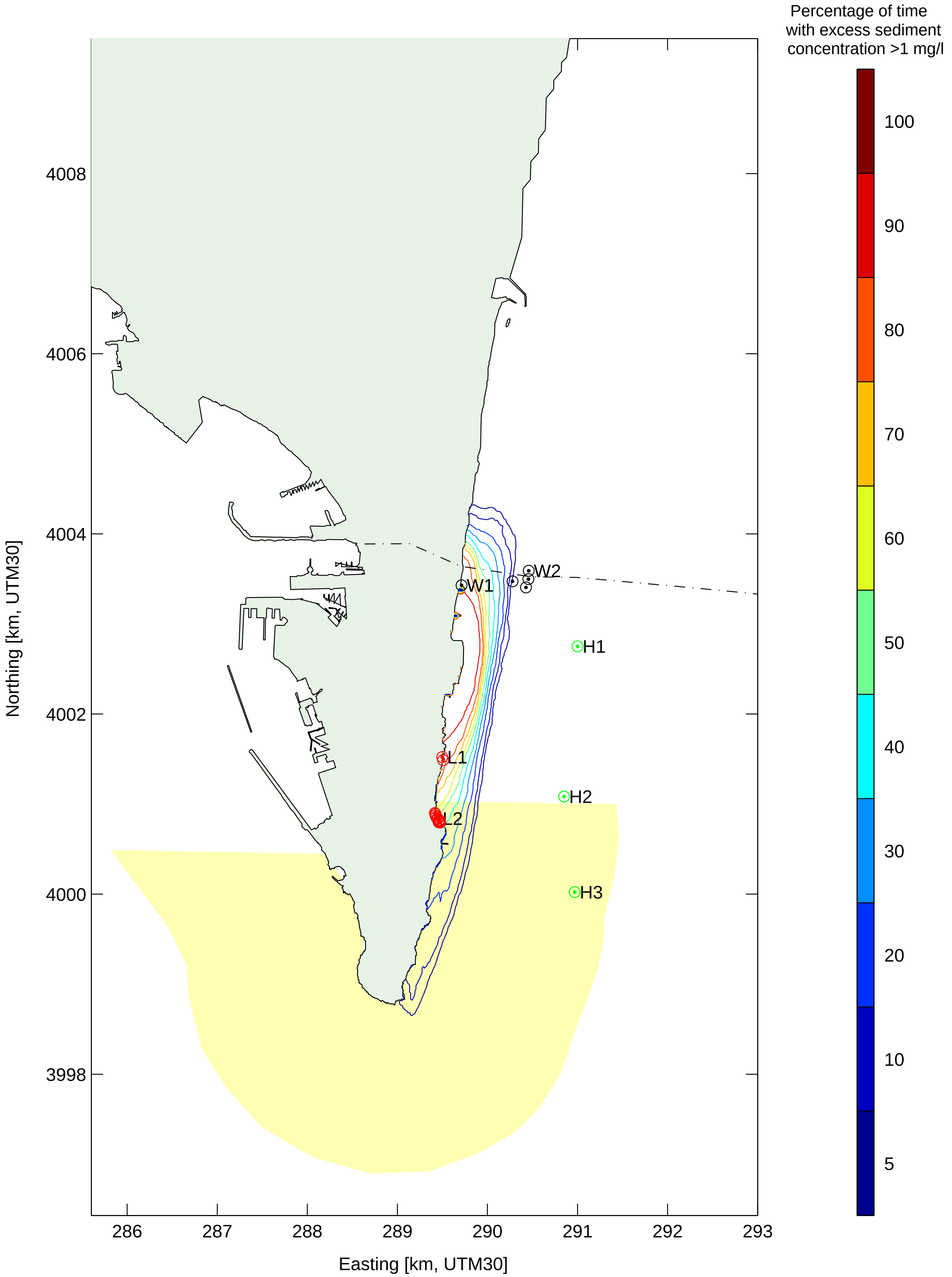
Percentage of time that the excess sediment concentration > 1 mg/l, scenario 1a Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Spring tide , construction of southern part of sea defence	ID: sc1a−s	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.4c



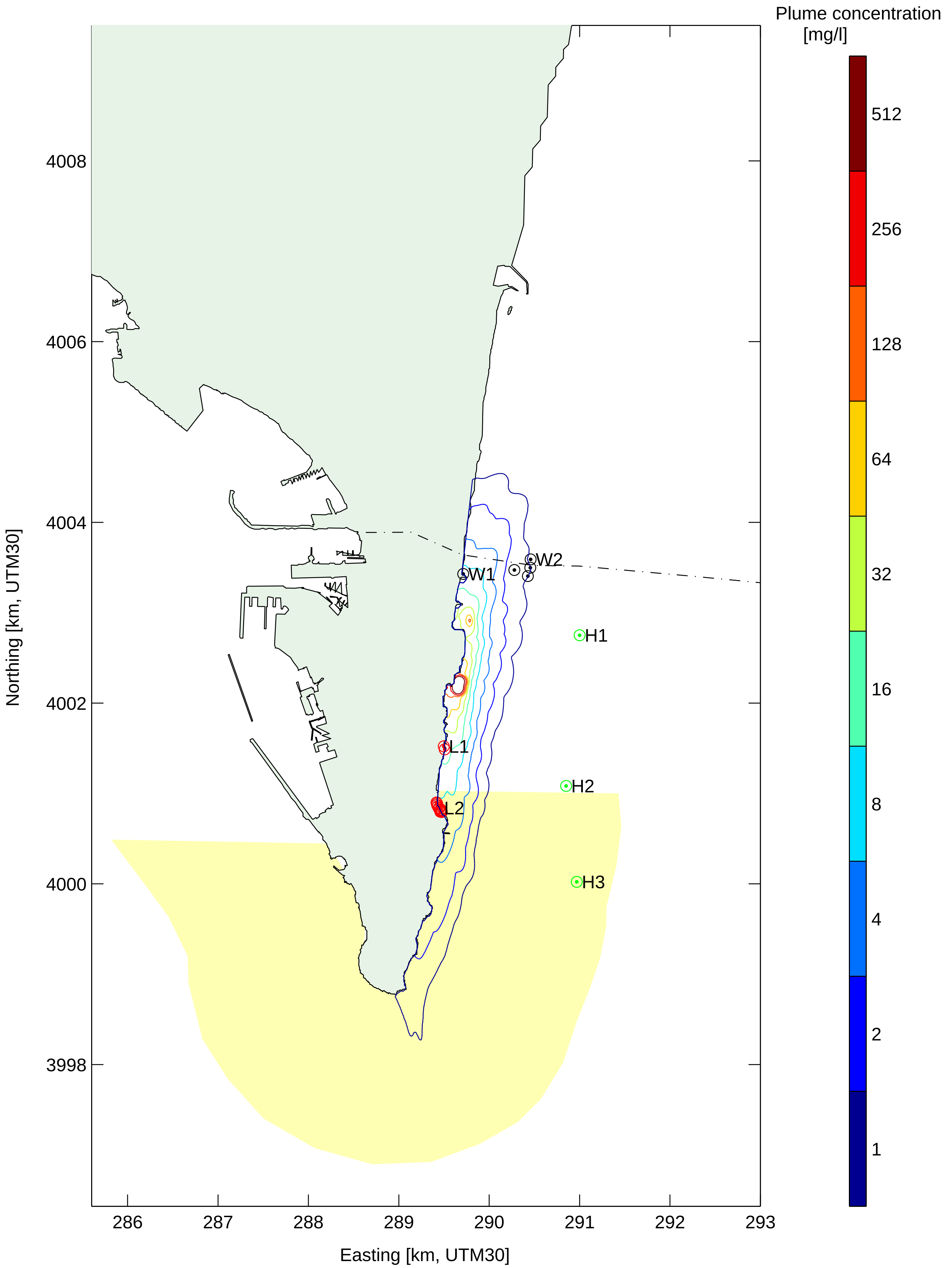
Max. expected excess concentrations of plume extent, scenario 1a Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Spring tide , construction of southern part of sea defence	ID: sc1a−s	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.6c



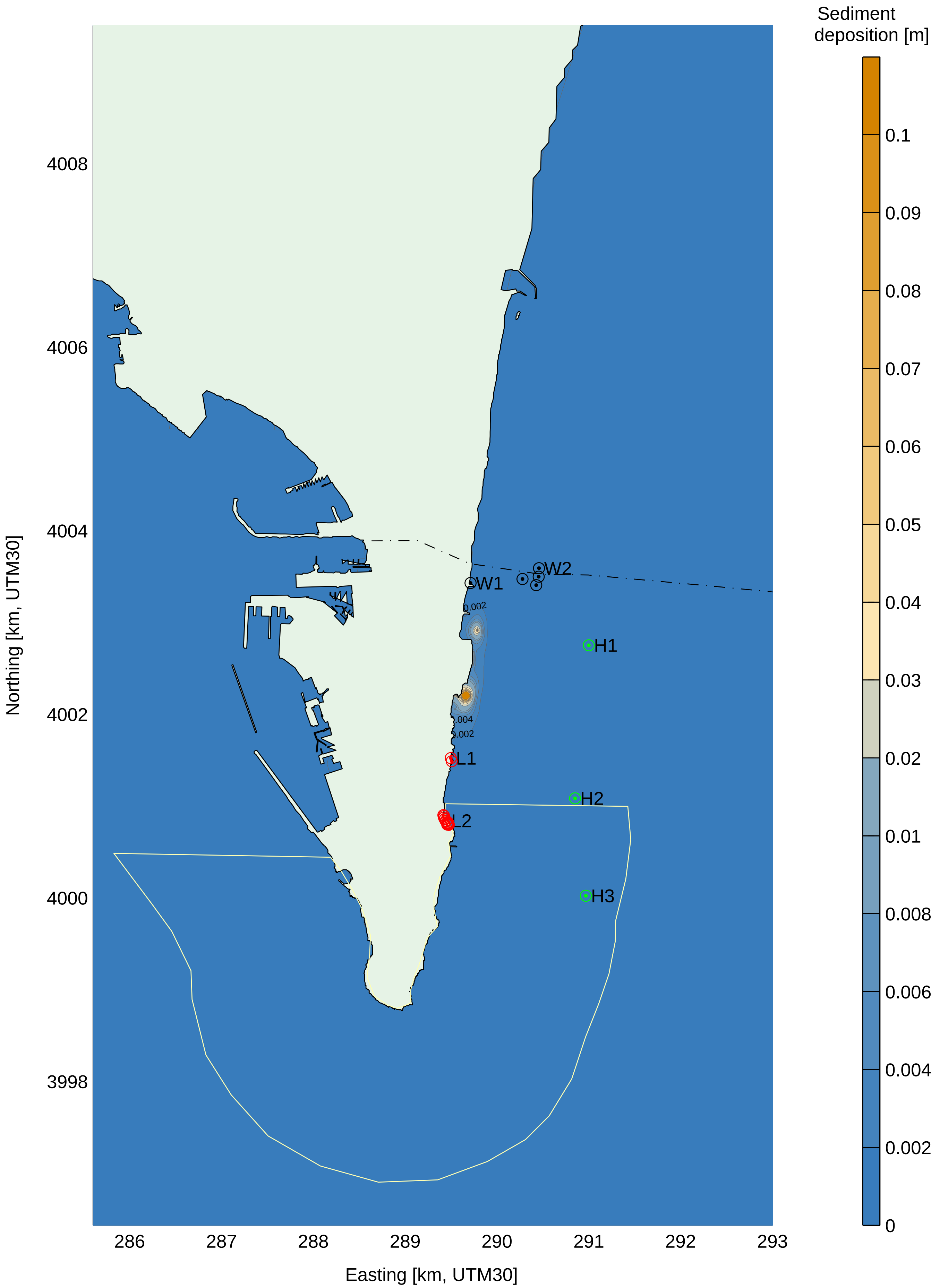
Deposited sediment thickness due to scenario 1a Red, green and black spots are resp. limpets locations, habitats and wreck sites Yellow outlined: Marine Nature Reserve	RunID: sc1a	
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.8



Percentage of time that the excess sediment concentration > 1 mg/l, scenario 1b Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide , construction of northern part of sea defence	ID: sc1b−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.10c



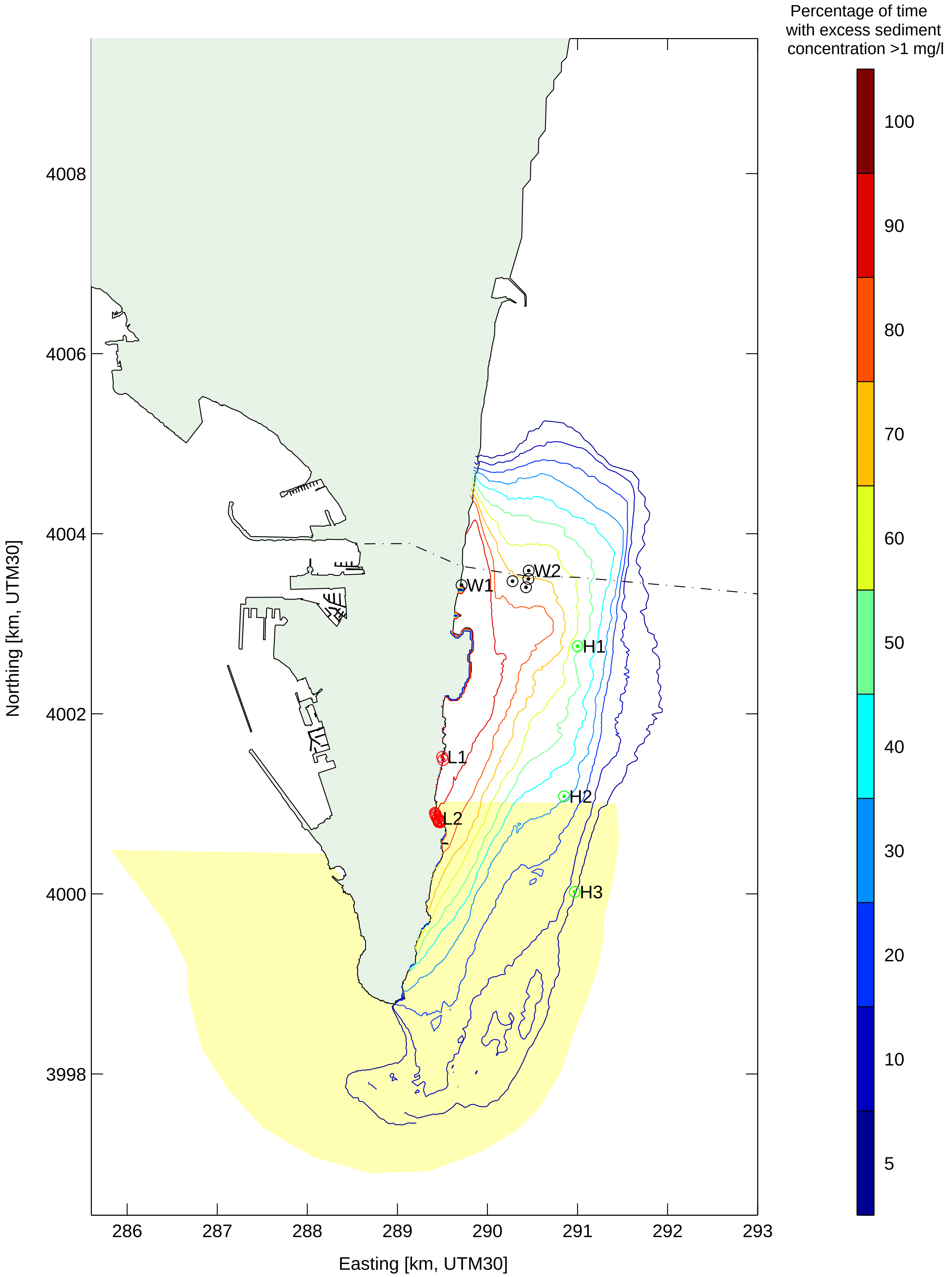
Max. expected excess concentrations of plume extent, scenario 1b Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide , construction of northern part of sea defence	ID: sc1b−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.12c



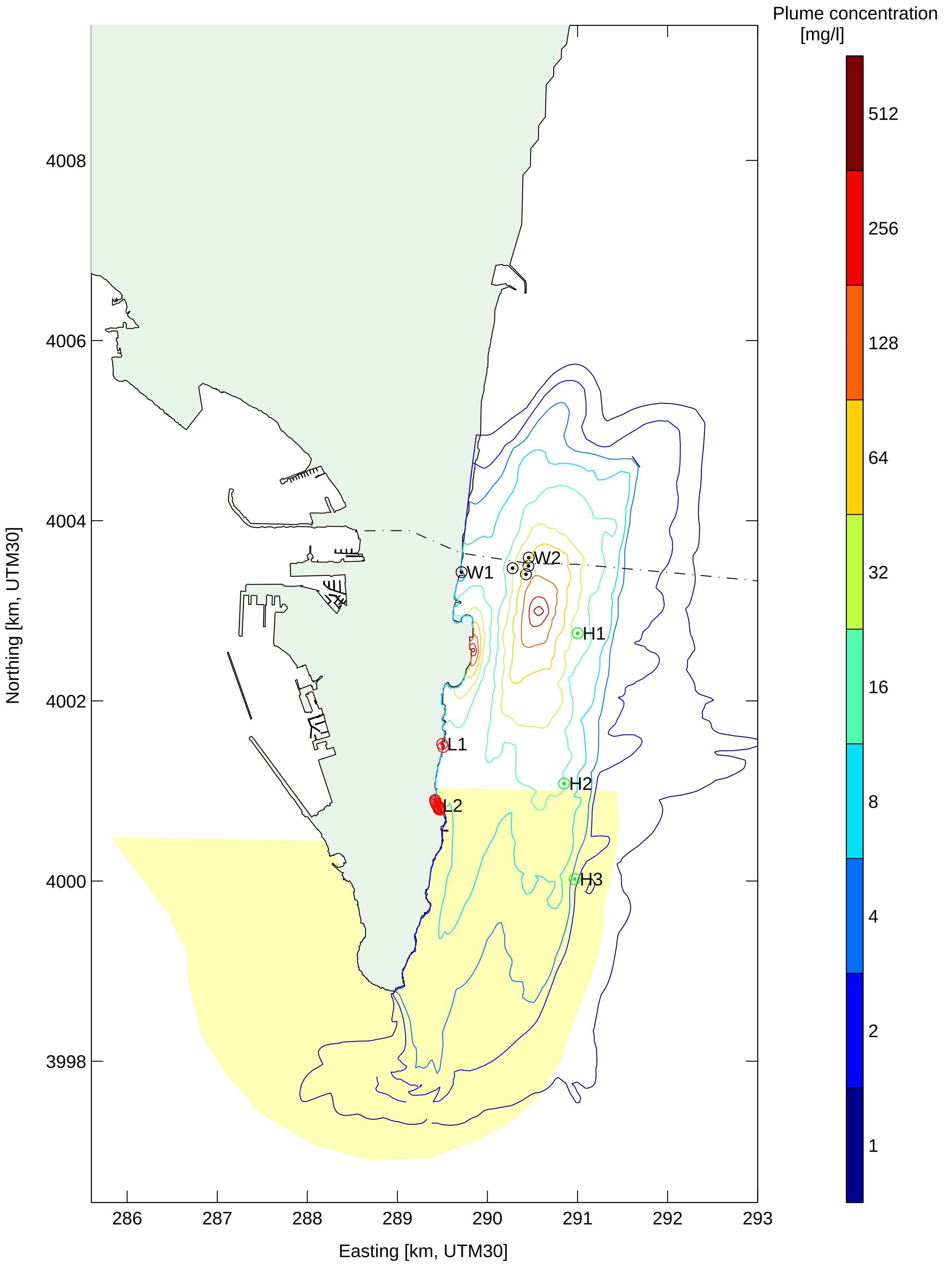
Deposited sediment thickness due to scenario 1b
Red, green and black spots are resp. limpets locations, habitats and wreck sites
Yellow outlined: Marine Nature Reserve

RunID: sc1b	
Delft3D−Online Morphology	

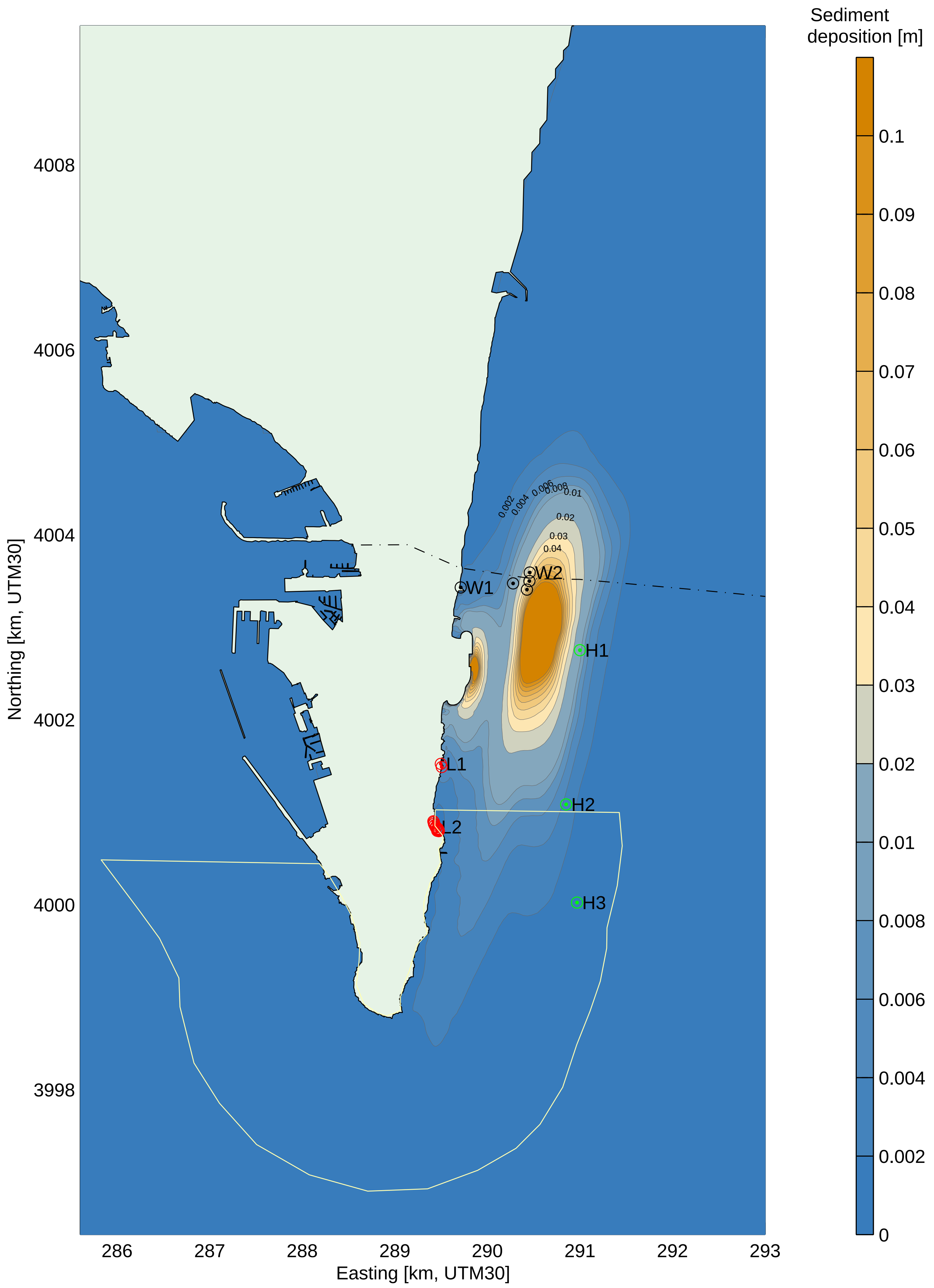
WL | DELFT HYDRAULICS | H4725 | Fig. 7.13



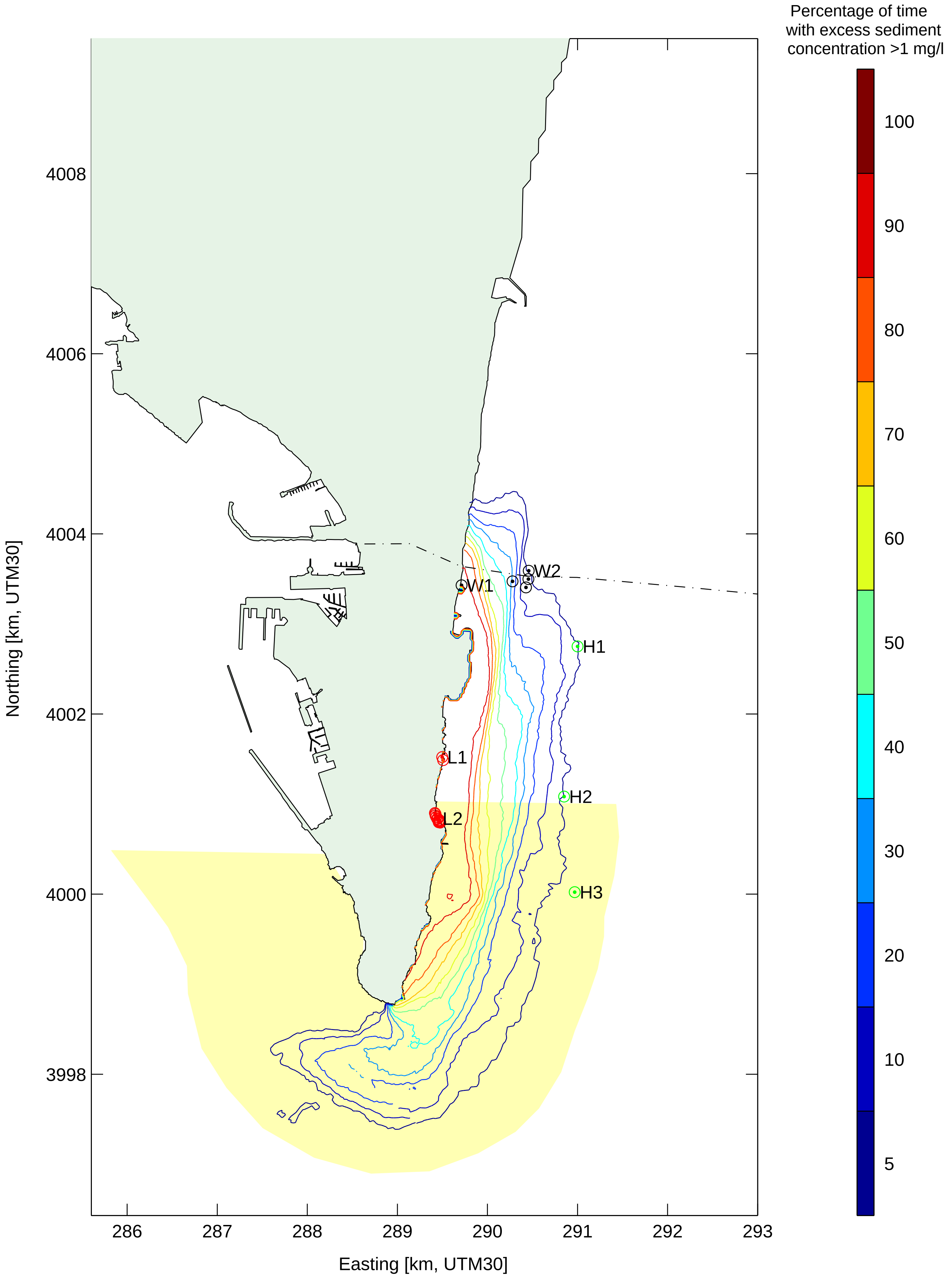
Percentage of time that the excess sediment concentration > 1 mg/l, scenario 2a Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide , with dredging from northern borrow area	ID: sc2a−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.15c



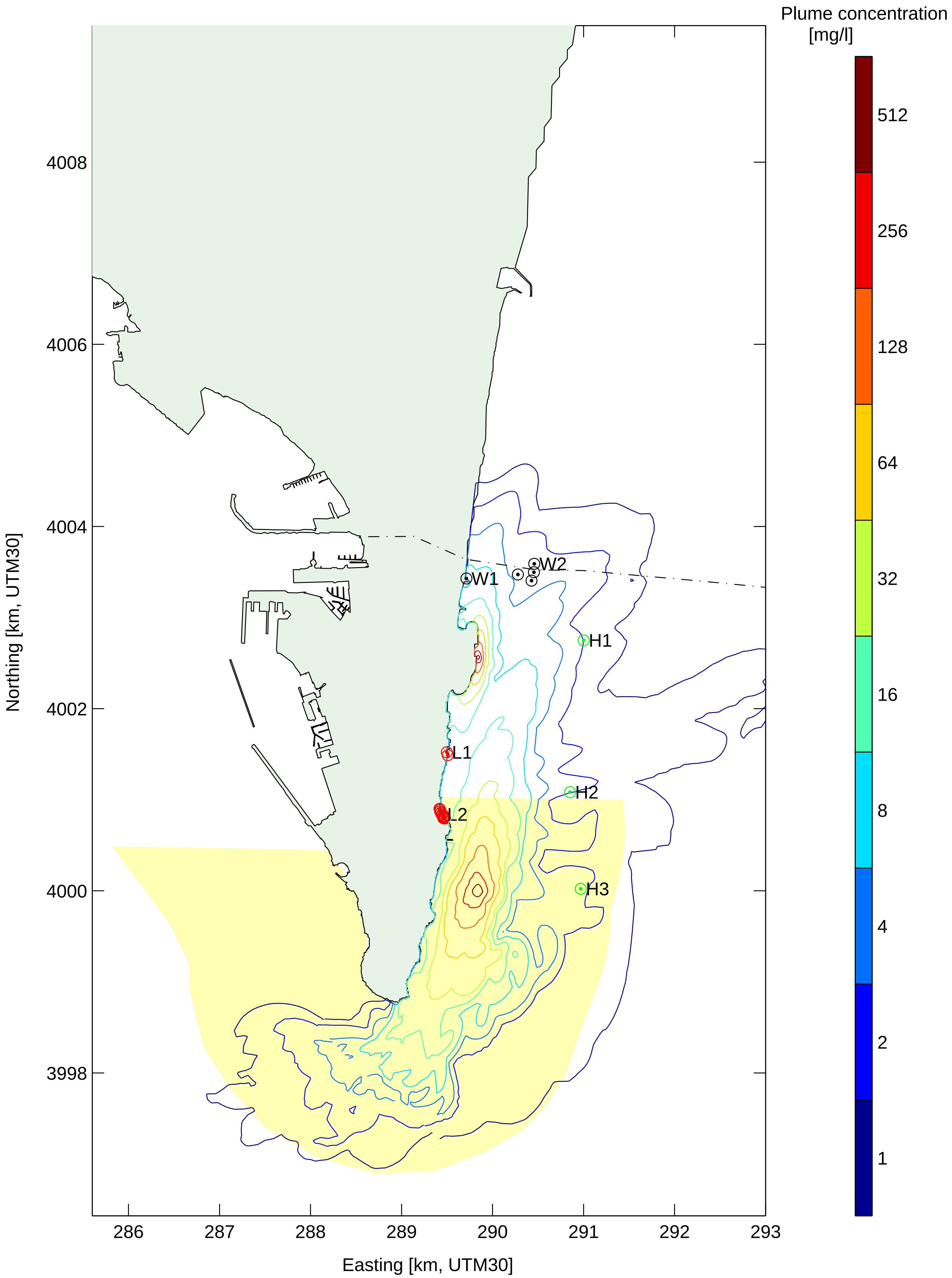
Max. expected excess concentrations of plume extent, scenario 2a Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide, with dredging from northern borrow area	ID: sc2a−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.17c



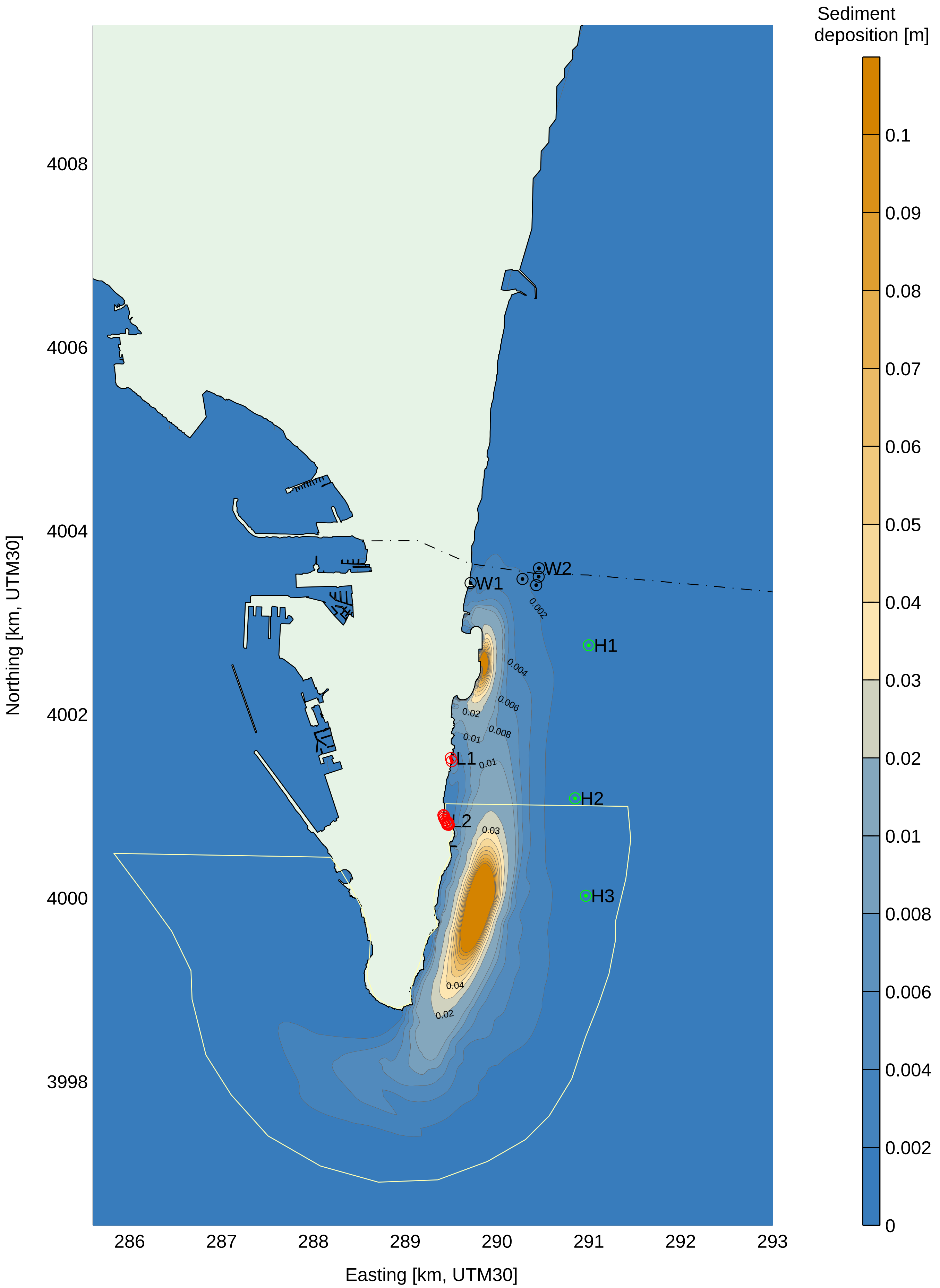
Deposited sediment thickness due to scenario 2a Red, green and black spots are resp. limpets locations, habitats and wreck sites Yellow outlined: Marine Nature Reserve	RunID: sc2a	
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.18



Percentage of time that the excess sediment concentration > 1 mg/l, scenario 2b Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide , with dredging from southern borrow area	ID: sc2b−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.20c



Max. expected excess concentrations of plume extent, scenario 2b Red, green and black spots are resp. limpets locations, habitats and wreck sites yellow shaded area: Marine Nature Reserve Neap tide, with dredging from southern borrow area	ID: sc2b−n	Bottom layer (5)
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.22c



Deposited sediment thickness due to scenario 2b Red, green and black spots are resp. limpets locations, habitats and wreck sites Yellow outlined: Marine Nature Reserve	RunID: sc2b	
	Delft3D−Online Morphology	
WL | DELFT HYDRAULICS	H4725	Fig. 7.23

Appendix A

Model descriptions

Unibest-CL+

DELFT3D-SED-ONLINE

Kamphuis Diffraction Formulae

computer program

Unibest-CL+

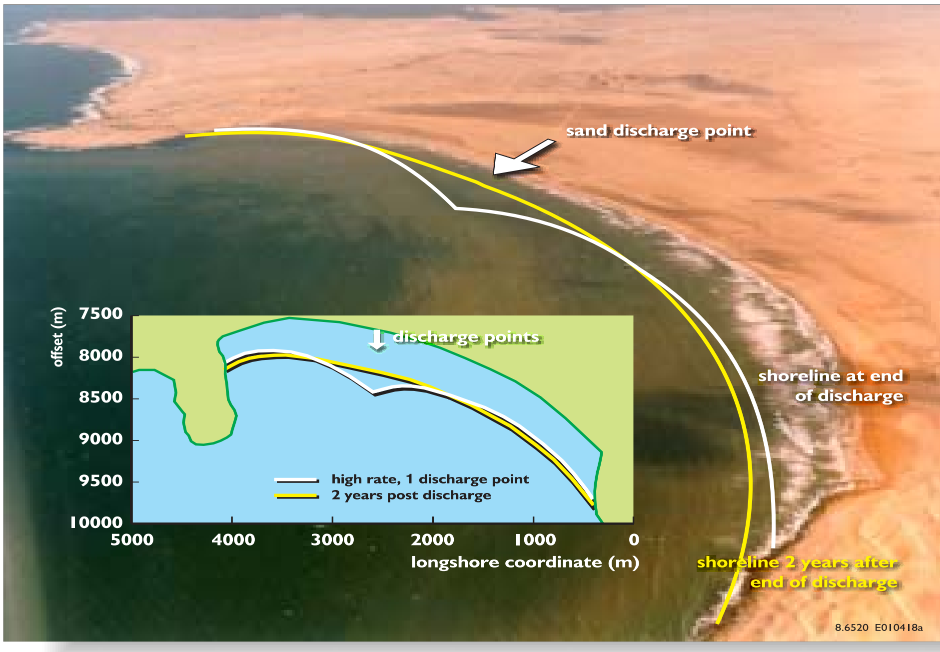
applications

UNIBEST-CL+ is a powerful tool to model longshore sediment transports and morphodynamics of coastlines. During its application on coasts world-wide it has proven to be a flexible and easy-to-access tool capable of simulating a large variety of coastal problems. UNIBEST-CL+ is a sediment balance model with which longshore transports computed at specific locations along the coast can be translated to shoreline migration. If required the effect of cross-shore phenomena can be included by importing the results of the UNIBEST modules UNIBEST-TC and / or UNIBEST-DE.

The shoreline model UNIBEST-CL+ can be used for a wide range of coastal engineering projects. A typical application is the analysis of the large scale morphology of coastal systems to provide insight in the causes of coastal erosion or to predict the impact of planned coastal infrastructure (such as a port) on the coast. But the model can also be used for considerations on a smaller scale, like the evaluation of the shoreline evolution around coastal protection works (groynes, revetments, river mouth training works and to some extent detached breakwaters). Sediment sources and sinks can be defined at any location to simulate river sediment supplies, the effect of land subsidence or sea level rise, offshore sediment loss, artificial sand bypass and beach mining. These features make it a suitable tool for the functional design of coastal defence schemes and the prediction of their impact on the coast, in the feasibility stage and in many cases also in the detailed design stage of projects.

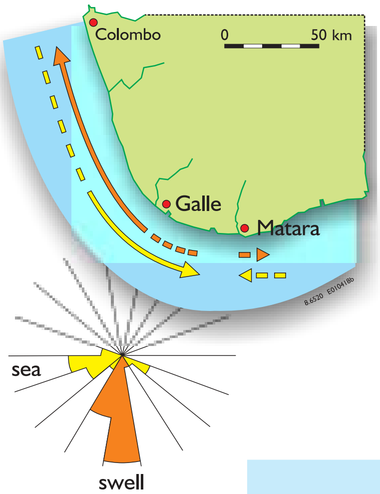
The shoreline is defined relative to a user-defined reference line which may be curved. This enables the modelling of complex coastal areas such as deltas, bays, circular shaped beaches and even complete islands. Shoreline evolution computations can be made over a period of decades. In these computations changes in the longshore transports with time due to re-orientation of the shoreline are taken into account. Since the runs are very time-efficient, with a properly set-up model various scenarios can be evaluated and sensitivity analyses can be carried out.

The release of UNIBEST-CL+ marks a significant enhancement on its very successful predecessors UNIBEST-LT and UNIBEST-CL, and combines these two models in a windows environment. The improvements relate to the possibility of applying a curvilinear reference line, the definition of different scenarios, and the graphical output.



Example of prediction of re-distribution of discharged sand along the coast.
(courtesy of CSIR, South Africa and NAMDEB)

WL | delft hydraulics

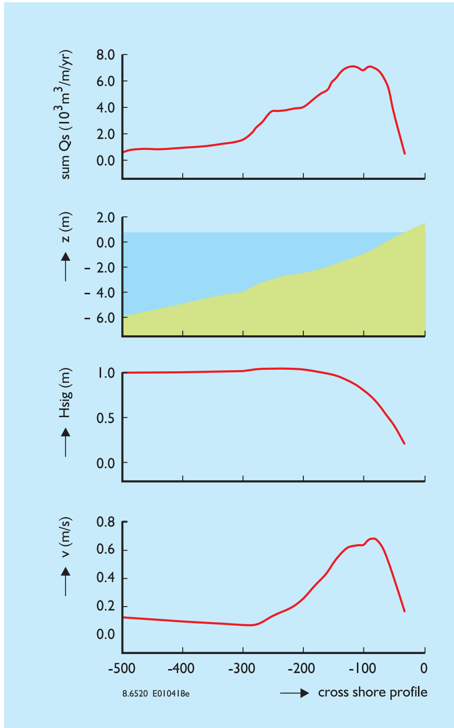


processes

Longshore currents and sediment transports generated by tidal currents and obliquely incoming waves are computed. The gradients in the (time-dependent) transports are used as input for the shoreline model.

The model includes a wave propagation module to transform the offshore wave climate to the surf zone (assuming fairly uniform depth contours) and to compute the surfzone dynamics, according to the Battjes-Stive model for wave propagation and wave decay. Principal processes are accounted for, such as changes in wave energy as a result of bottom refraction, as well as shoaling and dissipation induced by wave breaking and bottom friction.

The distribution of the longshore current along the coastal profile is derived from the depth-averaged momentum equation alongshore, where bottom friction, the gradient of the radiation stress and the tidal surface slope alongshore are accounted for.



Examples of computed distributions of longshore transport, wave height and longshore current along the profile

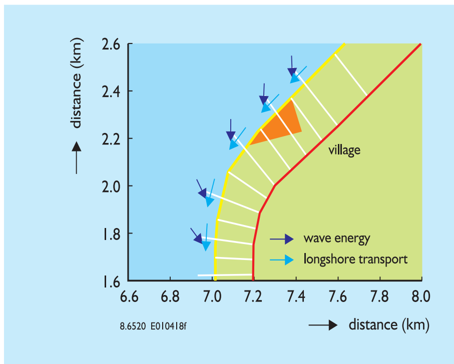
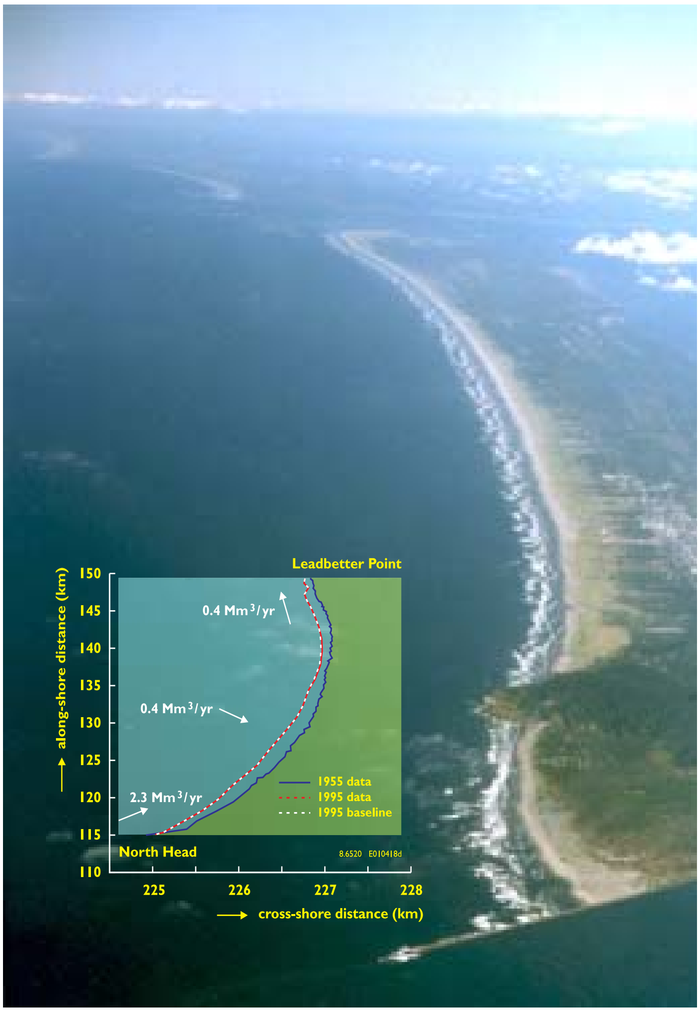


Illustration of curvilinear grid



Long Beach Peninsula, Washington, U.S.A. (Courtesy of the Washington State Department of Ecology, SW Washington coastal erosion study).

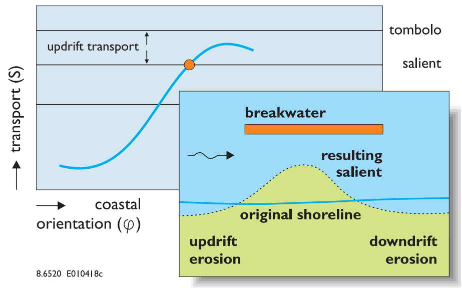
Longshore transport and its distribution along the coastal profile can be evaluated according to several total-load sediment transport formulae for sand (such as Bijker, van Rijn) or gravel (Van der Meer & Pilarczyk). The transports respond to local wave- and current conditions in an instantaneous, quasi-steady way. The net longshore transport can be computed on the basis of hundreds of combinations of wave- and tidal conditions, and the gross contributions in both directions can easily be derived.

An essential aspect in shoreline modelling is the strong relationship between the orientation of the coast and the longshore transport. In UNIBEST-CL+ this relationship is computed and presented in a so-called S-ϕ-curve. This curve forms the basis for the shoreline modelling since it provides information on transport gradients caused by a curvature of the coastline, as well as on the time-dependent response of the longshore transport on changes of the coast-orientation with time.

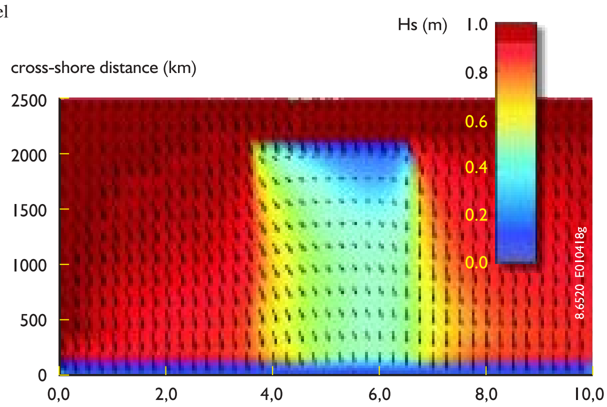
In the shoreline model the coast is represented by a single line. Given an active profile height and computed longshore transports along the coast the shoreline movement is computed on the basis of a continuity equation for sediment. Various initial and boundary conditions may be introduced so as to represent a variety of coastal situations. A non-uniform offshore or nearshore wave climate along the coast (e.g. due to structures) and resulting gradients in the longshore transport can be modelled. Different shapes of the coastal profiles can be defined along the coast and seasonal variations in the wave climate can be simulated.

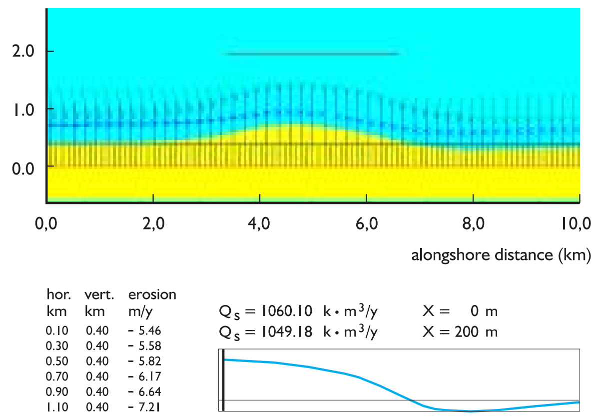
The bypass of sand along structures is modelled, taking into account the cross-shore distribution of the transport and the time-dependent shoreline migration, including the effect of shoreline re-orientation updrift of the structures.

S-ϕ curve, providing insight in salient or tombolo formation



A wave field around offshore breakwater, computed with DELFT3D-WAVE model





Computed salient behind offshore breakwater, computed with UNIBEST-CL+

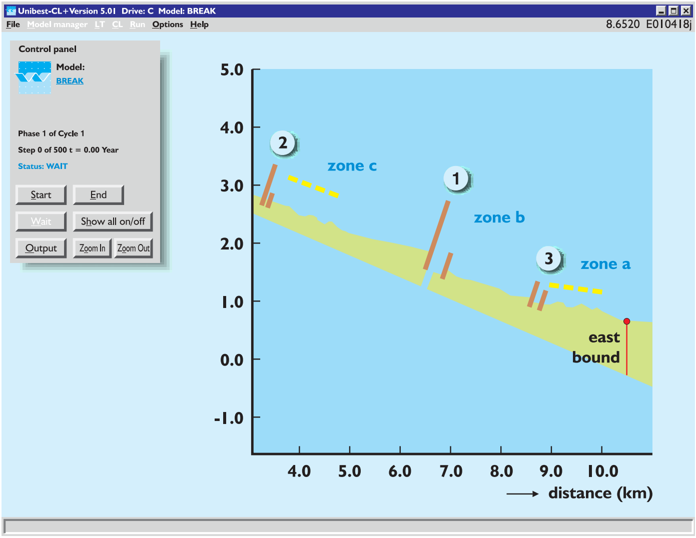
link with other models

In most cases the nearshore coastal dynamics are largely determined by the waves and the wave-induced currents. A proper representation of the wave climate just seaward of the active profile is therefore essential, and translation of the offshore wave climate to nearshore locations is required. For many cases the wave propagation can be computed with the wave module of UNIBEST-CL+. However, for a complex bathymetry or for a situation with large structures the model is often applied in combination with 2DH wave propagation models. The new version of UNIBEST-CL+ offers the option to improve this procedure, by exchanging information with the sophisticated 2DH wave propagation model DELFT3D-WAVE in an efficient and user-friendly way. An interface helps the user to extract wave climates at nearshore points from computed 2DH wave fields in an interactive manner. This option is available for users who also own the model DELFT3D-WAVE. This model includes the state-of-the-art wave propagation models SWAN and HISWA. With this option the development of the shoreline in and adjacent to the shadow zones of offshore islands, harbour moles and (large) detached breakwaters can be computed on the basis of detailed wave modelling in a time-efficient and flexible way. This option is especially valuable for the impact assessment or the functional design of structures, and for cases with a complex offshore bathymetry (shoals, channels, bays, non-uniform depth contours).

UNIBEST-CL+ generates output suitable for use in Geographical Information Systems (GIS) for the analysis of the effects of coastal erosion and coastal defence measures on values in the coastal zone.

future developments

UNIBEST-CL+ is regularly upgraded so as to increase its scope. For the next release the graphical options for the definition of the coastline model will be improved for more accuracy and user-friendliness. On the basis of scanned maps of the area with this planned option the reference and the coastline, as well as the positions of the local wave climates and structures can be defined graphically.



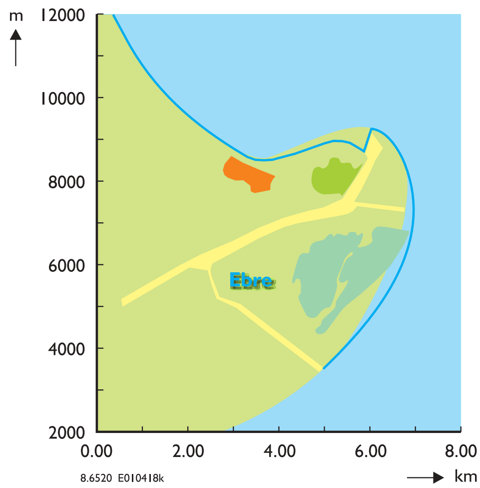
with coastal protection schemes

model organisation

UNIBEST-CL+ runs under Windows-based systems and is controlled by a user-friendly interface in which input files can be created and edited, simulations be made and model results be graphically inspected. High-quality colour graphics can be sent to different hard copy output devices. Detailed output can be written to ASCII files. For the longshore transport computations a batch facility is provided to execute a large number of consecutive (sensitivity) runs. The various input fields are stored in different ASCII files, enabling the user to store and link different input combinations to the runs.

The main input consists of:
- the wave climate : combinations of significant wave height, wave period, wave direction and the percentage of occurrence of these combinations
- the tidal regime : combinations of current velocities and water levels and the percentage of occurrence of these combinations
- the coastal profile : any coastal profile and the active zone can be defined
- the parameters for wave propagation and sediment transport computations
- the sediment: characteristics of sand or gravel (non-cohesive sediment)
- the coastline
- the boundary conditions
- active profile heights
- grids, in the coastal profile and along the coastline
- structures (groynes, offshore breakwaters, revetments, sources/sinks)
- scenarios for combinations of above input, defining the runs

The main output (all to be graphically inspected and in ASCII files) consists of:
- the wave characteristics along the coastal profile
- the cross-shore distribution of the longshore currents (wave-induced and tidal)
- longshore transport for various coast-orientations if requested
- the cross-shore distribution of the longshore transport
- longshore transports in all profiles along the coast
- the coastline position, migration and orientation between grid points
- coastline migration rates between grid points



Example of application with strongty curved coastline

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computer program

3D Sediment Transport within Delft3D-FLOW

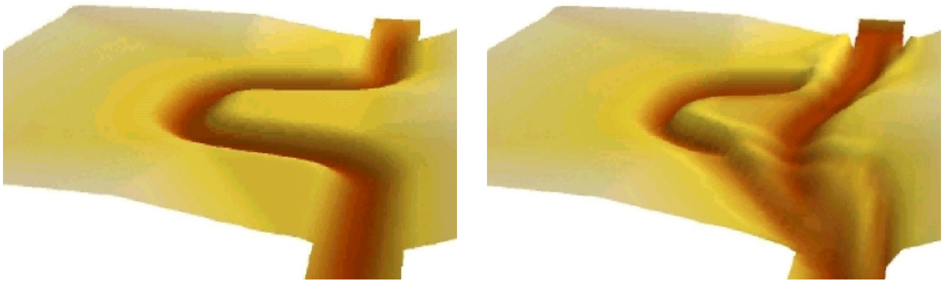
introduction

The full 3D suspended sediment transport, the initial bed level changes and the interaction between sediments and water quality parameters can be studied using Delft3D-SED. Neither the bed level changes nor the density effects of the sediment have feed back on the hydrodynamics.

Long-term morphodynamic changes, taking into account the influence of short crested wind generated waves can be handled with Delft3D-MOR. The applications are depth-averaged and the interaction between hydrodynamics, sediment transport and bed level changes is off-line, i.e. hydrodynamics, transport and bed level changes are computed sequentially one after the other.

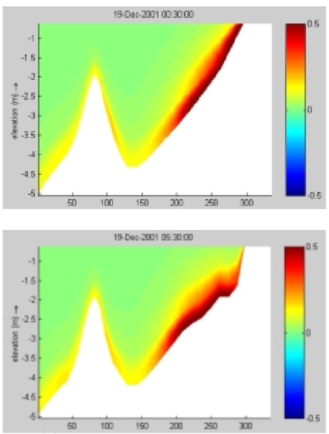
In situations where the effects of 3D flow are significant, or when the sediment concentrations substantial affect the density distribution, the 3D sediment transport option in Delft3D-FLOW is available. The effects of the sediments and bed level changes on the flow are now automatically taken into account.

Typical time and length scales of the applications are similar as for the hydrodynamic applications, i.e. from hours to days or a full spring-neap cycles in tidal areas. Typical length scales can range from near-field morphology such as local scour near the head of breakwaters to tidal inlets and estuaries and coastal areas. The on-line sediment transport formulation allows the combined use of cohesive and non-cohesive sediments.

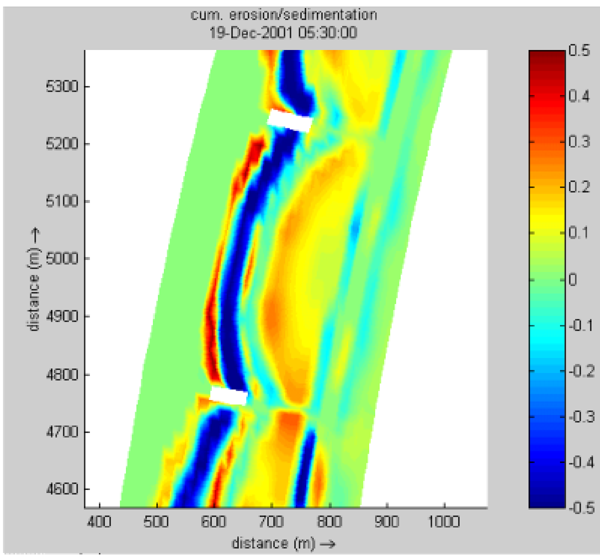


Example river bend cut-off; initial (left) and final (right) bathymetry.

Morphological impact of low-crested structures



Sediment concentration at begin (upper) and after 1 month (lower)



Bed level changes after 1 month

keywords:

cohesive and non-cohesive sediment, erosion, deposition, hindered settling, flocculation,

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suspended sediment transport

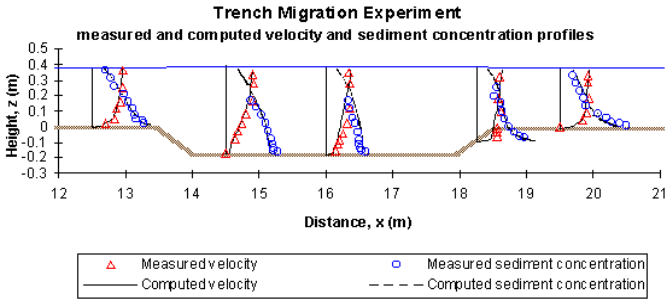
Three-dimensional transport of suspended sediment is calculated by solving the advection-diffusion equation for (up to 5) sediment fractions.
The fractions can be cohesive (mud) and non-cohesive (sand) sediments, each with its own characteristics.

cohesive sediments

The settling velocity accounts for flocculation due to salinity. Erosion and deposition rates follow the well-known Partheniades-Krone formulations.

non-cohesive sediments

The settling velocity and erosion and deposition rates follow van Rijn's formulations. The vertical sediment mixing is effected by waves and the choice of turbulence model. The equilibrium profile can be used as open boundary condition.

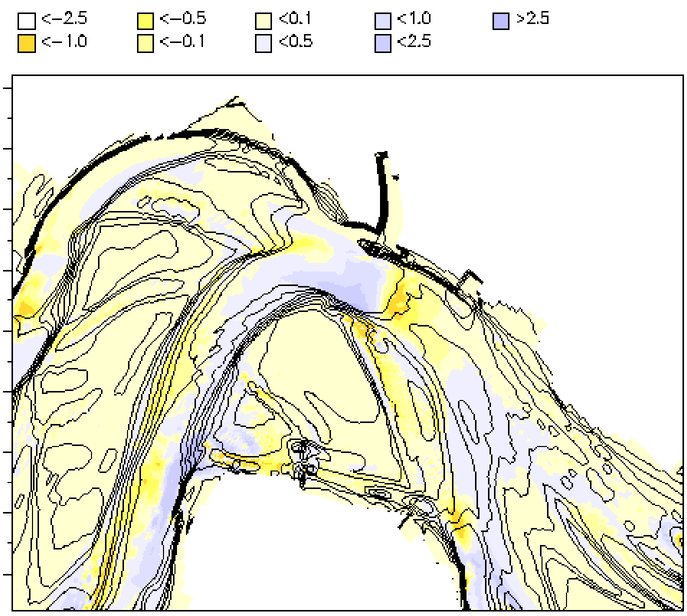


bed-load cohesive sediment transport

The bed-load transport of cohesive sediment is included, generally following van Rijn. Spatial gradients in bed-load transport will cause erosion and accretion of the bed. The transport vector includes the effect of longitudinal and transverse bed slopes.

morphological updating

The bed level is updated dynamically during the flow computation, taking into account the exchange with the suspended sediment vertical and the gradient of the bed load transport.



Example Western Scheldt: bed level changes in m after 1 morphological month.

additional features

The effect of the sediments on the fluid density can be included.

Hindered settling is accounted for.

A morphological delay can be applied to allow the hydrodynamics to reach a dynamic equilibrium from the initial conditions.

As morphological developments take place on a time scale larger then hydrodynamic time scales, a morphological time scale factor can be applied.

The effect of non-erodable layers can be taken into account; the vertical exchange term and the bed load transports are gradually reduced to zero as the sand layer thickness approaches zero.

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Diffraction Kamphuis (1992)

Wave diffraction is expressed in the diffraction coefficient K_d which is defined as:

$$K_d = H_d / H_i \qquad \text{(E.1)}$$

with

K_d = diffraction coefficient
H_d = diffracted wave height
H_i = incident wave height

In this section the schematisation of wave diffraction in the PONTOS model is decribed. The applied formulations for diffraction around a structure have been derived by Kamphuis (1992). The equations of Kamphuis are based on data presented by Goda et.al. (1978) and Goda (1985) for random waves, and include the effects of directional spreading.

A definition sketch is presented in Figure E.1. The equations presented by Kamphuis are:

$$K_d = 0.69 + 0.008\,\theta \qquad \text{for} \quad 0 \geq \theta > -90° \qquad \text{(E.2)}$$
$$K_d = 0.71 + 0.37\,\sin\theta \qquad \text{for} \quad 40 \geq \theta > 0° \qquad \text{(E.3)}$$
$$K_d = 0.83 + 0.17\,\sin\theta \qquad \text{for} \quad 90 \geq \theta > 40° \qquad \text{(E.4)}$$

with

θ = the angle between the straight line between the point of interest and the diffraction point (α_d) and the incident wave direction (α_i) and, see Figure E.1.

In Steetzel et.al.(1998) a preliminary formulation was proposed for implementation in the pilot version of the PONTOS model . This formula was based on the data of Wiegel (1962) for monochromatic waves. In Table E.1 the results of the formulae of Kamphuis are compared with data of Wiegel. The most striking differences between the data of Goda (Kamphuis formula) and the data of Wiegel are:

- The data of Goda indicate a diffraction coefficient of K_d= 0.7 at the edge of the shadow zone instead of K_d= 0.55 for the data of Wiegel.
- The data of Goda indicate that the diffraction coefficient at a certain point is fairly independent of the number of wave lengths from the diffraction point, while the data of Wiegel suggest that this is a relevant parameter, though only in the shadow zone, see Figure E.2 (and Table E.1).

The data of Goda for irregular short crested waves are considered to be more realistis than the data of Wiegel for monochromatic waves. Therefore in the PONTOS model the wave height in a diffraction zone is determined with the formulas of Kamphuis (equations E.2 to E.4). For the wave direction in the diffraction zone the angle α_d is taken. The wave period in the diffraction zone remains unchanged.

It should be noted that the formulae of Kamphuis seem to be based mainly on the data for considerable directional spreading (the left diagram of Goda in Figure E.2). For the situation with only little directional spreading (the right diagram of Goda in Figure E.2) the values in the shadow zone tend to be somewhat smaller than computed with the Kamphuis formulae. It can therefore be expected that the equations E.2 to E.4 are more reliable for sea waves than for swell waves. In this stage no fine-tuning of the formulae for the degree of directional spreading has been applied.

References

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Computation of coastal morphology
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Volume 4 - Background document

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Diffraction of waves by a semi-infinite breakwater,
Journal of the Hydraulic Division, Vol. 88, No. HY1, pp. 27-44

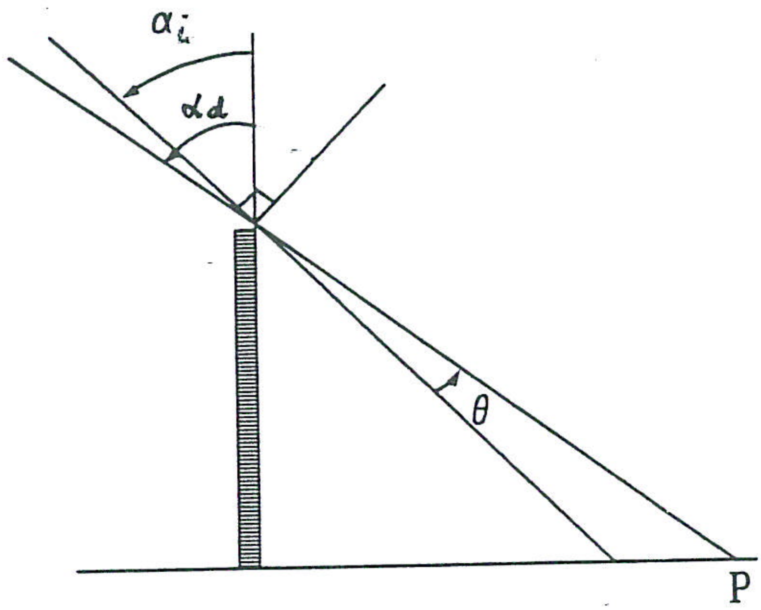
θ (°)	Kamphuis (1992)	Wiegel (1962)	
		r/L = 1	r/L = 5
-60	0.21	0.24	0.11
-30	0.45	0.35	0.17
0	0.69	0.56	0.54
+30	0.90	0.96	1.04
+60	0.98	1	1

θ = the angle between the straight line between the point of interest and the diffraction point (α_d) and the incident wave direction (α_i) and, see Figure E.1.

r = distance between point of interest and diffraction point

L = wave length

Table E.1 Comparison of Kamphuis formula with Wiegel data



Definition sketch for schematisation diffraction
(after Kamphuis, 1992)

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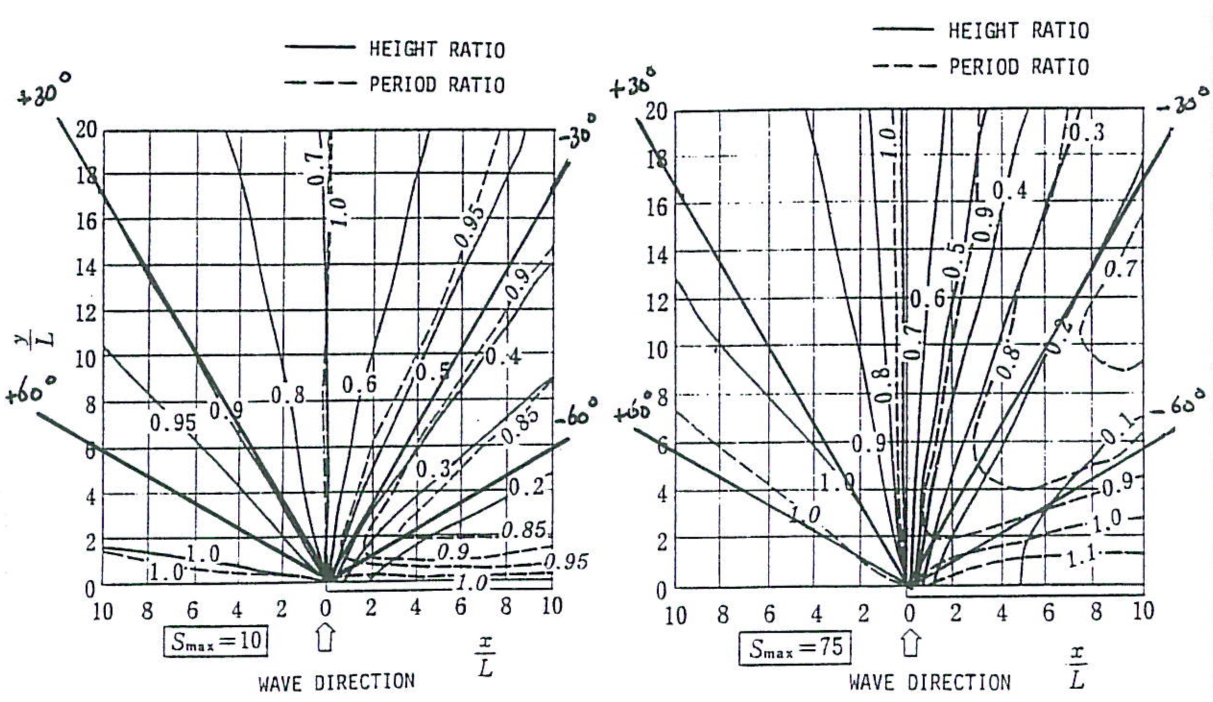
FIGUUR E.1

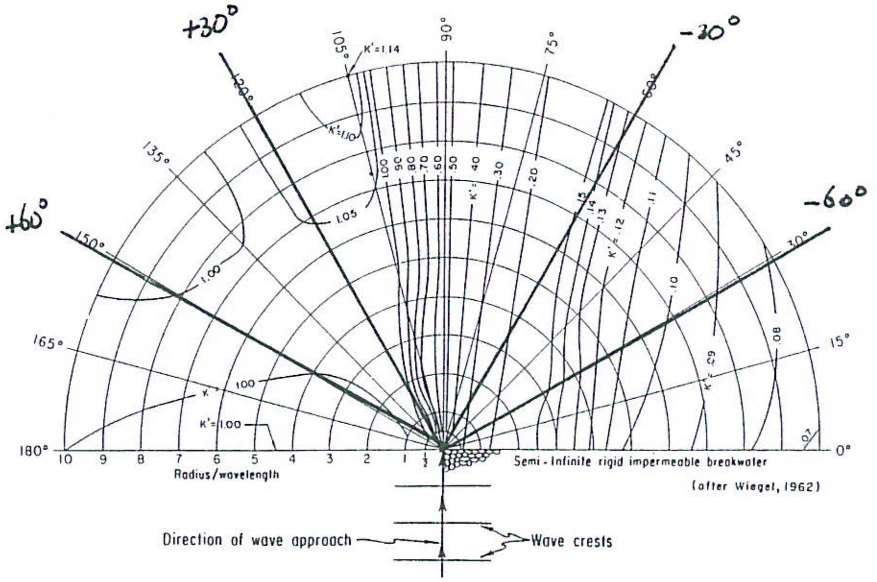


(a) S_{max} = 10

(b) S_{max} = 75

Goda (1978)

Diffraction Diagrams of a Semi-Infinite Breakwater for Directional Random Waves of Normal Incidence



Wave diffraction diagram--90° wave angle.

Wiegel (1962)

Data of Goda (1978) and data of Wiegel (1962)	**H3446**	
ALKYON / WL | DELFT HYDRAULICS	**FIGUUR E.2**	

Appendix B

EBG-TN-300 EIA Modelling: Sediment Plume Input based on Work Method

MCB (Gibraltar) Ltd

PROJECT NR: 30.3126

EASTSIDE DEVELOPMENT,
GIBRALTAR

EBG *Marine Engineering & Construction*

DOCUMENT TITLE:

EIA Modelling: Sediment Plume
Input based on Work Method

01	EFU	04-04-2007	Project information corrected	GSP	MLI
00	EFU	29-03-2007	For information – based on Masterplan March 2007	GSP	MLI
Rev	Author	Date	Description / Reason for Issue	Checked	Approved

Document Number: **EBG - TN - 300**

Technical Note

EBG

Project:	**Eastside Gibraltar**	Page:	1 of 5
	30.3126	Date:	29/03/2007
		Rev. No.:	00
Title:	**EIA Modelling: Sediment Plume Input based on Work Method**	Prep.:	EFU
		Chkd./App.:	GSP/MLI

1 INTRODUCTION

WL | Delft Hydraulics is preparing a hydrodynamic model for the Environmental Impact Assessment (EIA) of the Eastside Gibraltar project. This Technical Note presents an indication of the losses during dredging as expected by the Contractor. The loss estimates will be used by WL | Delft Hydraulics in the Delft3D modelling of sediment dispersion.

2 INFORMATION

2.1 PROJECT INFORMATION

Roughly three types of main marine works can be distinguished: dredging of trenches for the breakwaters (~35,000m^3), secondly the rockworks, approximately 750,000 ton rock (exclusive Accropods and concrete blocks) will be placed by various equipment and finally the material supply for the reclamation works (~800,000m^3). The sand dredged from the trenches will be used in the reclamation.

During the execution of the project various types of equipment will be used, a global time scheme for the main works is presented in Figure 1. The duration of each activity is presented in weeks. A Backhoe Dredger will start dredging at the beginning of the project. It will dredge the trenches for the rockworks which will take about 3 weeks. The dredged material (~35,000m^3) will be used in the reclamation.

At the same time the rock works will start, a Side Stone Dumping Vessel (SSDV) and land-based equipment will carry out these works. The installation of all rock works will take about 45 weeks.

When the breakwaters are partly completed, a Trailing Suction Hopper Dredger (TSHD) will carry out the sand supply for the reclamation. Part of the rubble tip can also be used for the reclamation. The sediment dispersion calculations will be based on the assumption that the total volume of suitable material from the rubble tip is approximately 200,000m^3. The remaining part for the required reclamation volume will be sand, i.e. the TSHD will dredge a total of 600,000m^3.

The complete project (including stockpiling of rock, land works, etc.) will take about 18 to 24 months.

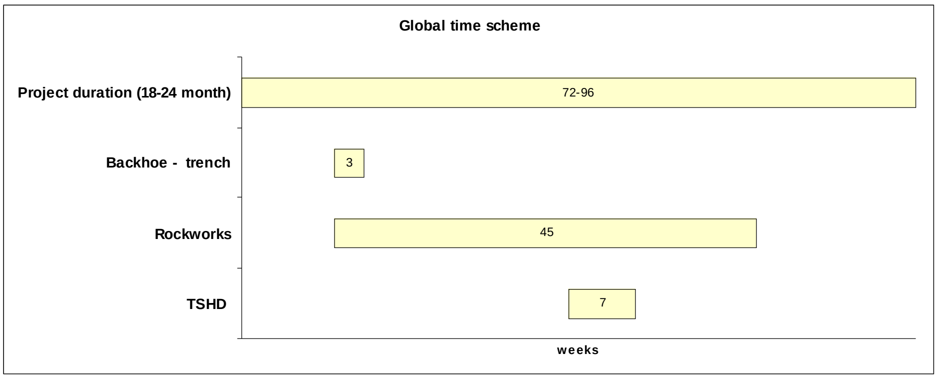


Figure 1 Global time scheme of main works

2.2 SOIL DATA

A geotechnical investigation has been performed. Preliminary grain size distributions are available for the EIA modelling. The average grains size distribution for the Northern Borrow Area is presented in Appendix I.

The in-situ density is estimated to be 1,950kg/m^3. The grain size in the trench is expected to be about the same as the Northern Borrow Area, the in situ density is 1,900kg/m^3.

	Northern Borrow Area	Southern Borrow Area
D(90) [mm]	0.946	0.964
D(80) [mm]	0.330	0.497
D(70) [mm]	0.280	0.390
D(60) [mm]	0.258	0.339
D(50) [mm]	0.238	0.296
D(40) [mm]	0.220	0.271
D(30) [mm]	0.195	0.248
D(20) [mm]	0.166	0.227
D(10) [mm]	0.110	0.195

Table 1 Grain size distribution borrow areas

2.3 EQUIPMENT

Three peaces of main equipment are proposed for the project, a Backhoe Dredger Razende Bol with Split Hopper Barge HAM 586 (750m^3), Trailing Suction Hopper Dredger type Ham 311 (2,175m^3) and Side Stone Dumping Vessel like HAM 601 (1,000 ton) or equivalent.

3 INITIAL SPILL

3.1 TRENCH DREDGING

The Backhoe Dredger will perform the trench dredging. The grain size distribution of the spill from the Backhoe Dredger will be equal to the in-situ grain size distribution of the trench. As the grain size distribution of the trench is only known from previous sampling yet, it seems reasonable to assume that the grain size distribution is equal to the Northern Borrow Area.

Disposal of sediment dredged from the trench by the Backhoe Dredger is done by a Split Hopper Barge (SHB ~750m^3).

Loading	240 min (175m^3/OH)
Sailing / positioning	20 min
Dumping	5 min
Sailing empty	5 min
	270 min

Spill BHD during loading	**5% ≡ 4.7kg/s** in-situ grain size (10% < 100µm)
Spill SHB during dumping	**5% ≡ 222kg/s** < 100µm

The grain size distribution of the spill during dumping is expected to be 100% < 100µm and 40% < 63µm.

3.2 BORROW AREA

TSHD cycle:

dredging	45 min
sailing loaded	30 min
connecting	20 min
pumping ashore	50 min
sailing empty	20 min
	165 min

Production per week: 106,000m^3/wk

Spill during dredging — Northern Borrow Area:
10% ≡ 2,175/(45*60)*1,950kg/m^3*0.10 **= 157kg/s**

Spill during dredging — Southern Borrow Area:
7% ≡ 2,175/(45*60)*1,950kg/m^3*0.07 **= 110kg/s**

The Southern Borrow Area contains slightly less fines, for this reason the overflow losses in the Southern Borrow Area will be slightly less as well.

The grain size distribution of the spill will be the difference between the in-situ grain size distribution and the calculated grain size distribution in the hopper, as presented in Appendix I. The grain size distributions of spill are approximately the same for both borrow areas. The grain size distribution of spill has been calculated with data from the Northern Borrow Area. The grain size distribution of the spill can be summarized as follows:

Grain size [µm]	% min	% max
200-150	**10**	20
150-63	**30**	40
63-30	30	**40**
<30	10*	**20***

* assumed

Table 2 Grain size distribution

It is proposed to do the EIA modelling with the most conservative grain size distribution, which is the grain size distribution with the highest percentage of fines (bold printed values in Table 2).

3.3 ROCK WORKS

The rock works (construction of breakwaters and coastal defence) will be carried out using different equipment. As the works can not yet be divided between land based equipment and Side Stone Dumping Vessel (SSDV), calculations should be performed with the initial loss for the SSDV for all rockworks, as this is the most conservative scheme.

SSDV cycle:

loading	2 hr
sailing loaded	1 hr
positioning/dumping	1 hr
sailing empty	1 hr
	5 hr

Production per week	27,552ton/wk
Spill during dumping	**1% ≡ 2.8kg/s**

The spill is expected to contain 50% sand (~150µm) and 50% of fines (<63µm).

3.4 RECLAMATION WORKS

The reclamation works will be an open reclamation, losses are difficult to predict for these situations but theoretically all sediments with a grain size larger than 63µm will settle within 100 meter and all fines (<63µm) will be suspended. This will be **0.5%** of all sediment brought into the reclamation area or beach nourishment. During heavy sea states, losses may be greater temporarily. This will be reduced as much as possible by construction of the breakwaters in an early stage.

Spill during disposal: **0.5% ≡** 2,175 m^3/(50*60)*1,950kg/m^3*0.005 **= 7kg/s**

4 DEEPENING OF BORROW AREAS

The average deepening of the borrow areas can be calculated by dividing the borrow area surface area by the total dredged quantity.

Assumptions

- It is assumed that the total quantity is dredged from one borrow area and the dredging is equally distributed over the complete area.
- The total quantity of sand supply which is needed the works is expected to be 600,000m^3.

The borrow area's are displayed at drawing EBG-DR-10.352 (Appendix II).

Calculation

Southern Borrow Area (1,600,000m^2)

Expected deepening: $$\frac{0.6 \cdot 10^6 m^3}{1.6 \cdot 10^6 m^2} = 0.4m$$

Northern Borrow Area (660,000m^2)

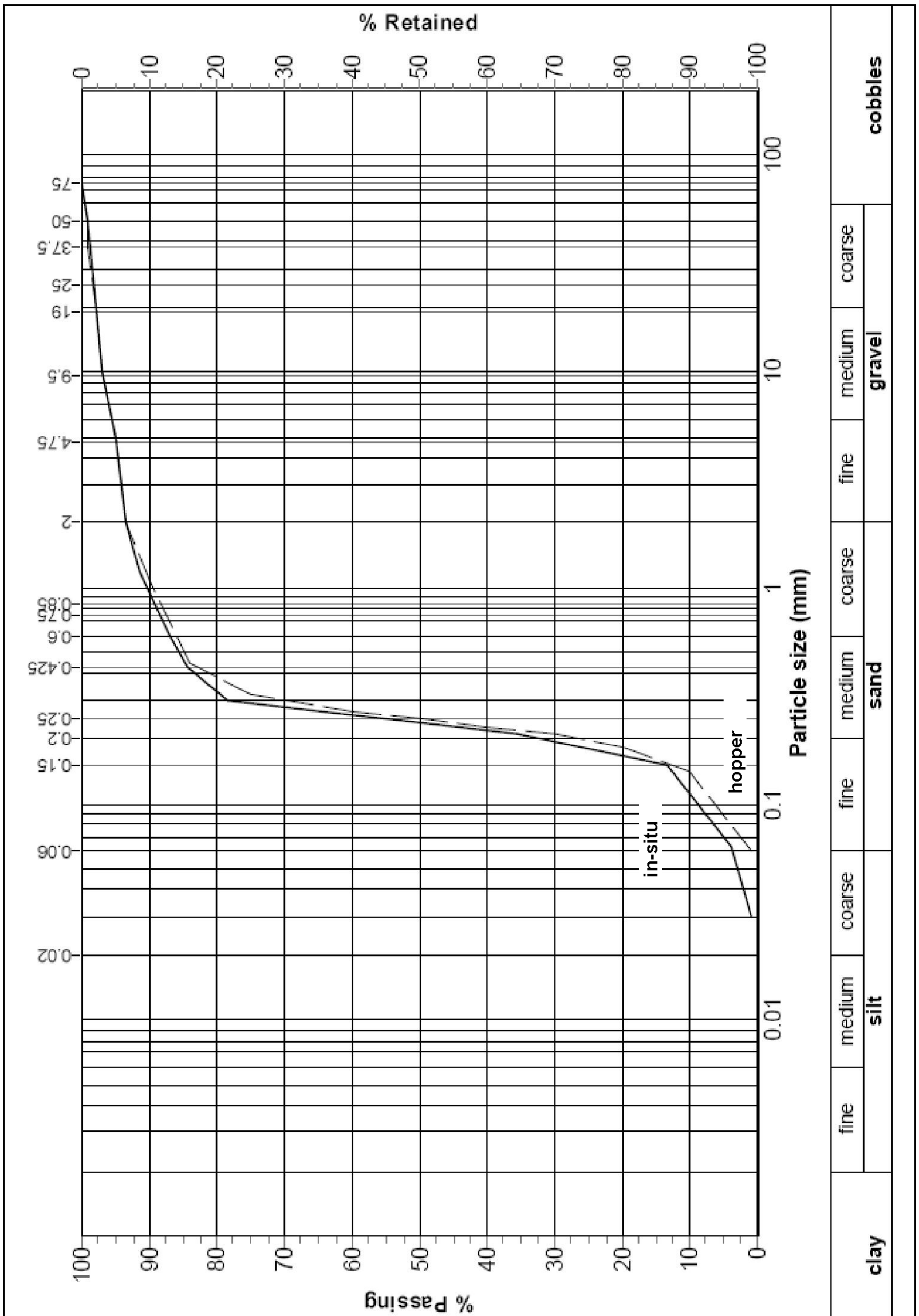
Expected deepening: $$\frac{0.6 \cdot 10^6 m^3}{0.66 \cdot 10^6 m^2} = 0.9m$$

5 DISCUSSION AND CONCLUSION

The presented fluxes (printed in bold in this document) are proposed for sources in the hydrodynamic modelling of the dredging plume. It should be noted that they only occur during the actual dredging/dumping works, these periods are mentioned for each source. Production of the proposed equipment and execution times are all based on preliminary information, for this reason conservative assumptions haven been made.

EBG

Appendix I



Particle Size Distribution	Pr. no. 30.3126
Eastside Development, Gibraltar	Calc by CLD
	Date 27/07/06
Northern Borrow Area	
Grain Size Distribution In-situ and in Hopper	

Appendix II Drawing Proposed Borrow Areas: EBG-DR-10.352.pdf