Able Humber Ports Ltd
Marine Energy Park
Proposal to build a quay and associated development on the south bank of the River Humber

Planning Inspectorate Reference: TR030001

Comments on other parties' responses to The Examining Authority's 2nd Written Questions

On behalf of
The Environment Agency
Unique Reference Number: 10015552

12 October 2012

The following information is given by the Environment Agency (EA) in respect of answers given by other parties to the Examining Authority's second written questions, and to provide further clarity where possible:

Question 22:
For Natural England:
Q22. The Morris Report and Natural England's Written Representation (29 June 2012, paragraph 8.6 to 8.9) are based on a review of the managed retreat design.
A new design based on a Regulated Tidal Exchange is now being put forward by Black & Veatch (August 2012), in line with recommendations from the RSPB and the EA.
What level of certainty is there that this new design would deliver adequate compensation for –
(a) The foraging requirements and possibly roosting requirements of migratory birds?
(b) Other possible compensation requirements related to the SAC, SPA and Ramsar sites?
(c) What further assessment does Natural England consider necessary to prove or support this new proposal?

The EA would advise the Examining Authority that we have encouraged Able Humber Ports Ltd (the applicant) to explore opportunities to extend the potential time period of sustainable mudflat within the compensation site via a Regulated Tidal Exchange (RTE) scheme and we have been encouraged by the early draft concept put forward by the applicant. The EA is currently reviewing the further development of this concept in to more detailed design. The applicant needs to explore long-term management and maintenance to ensure the project will still be viable, given the complexity of the scheme. Management and maintenance arrangements will need to be secured within an appropriate Legal Agreement.

Question 23:
Paragraph 4.5.18 of Report EX44.1 in the Supplementary Environmental Information states that the DRAX Heron Renewable Energy Plant site overlaps with proposed Mitigation Site A and that if the DRAX project proceeds on its current basis then it may affect the viability of Site A. There is a suggestion in paragraph 4.5.19 that the impacts on Site A could be avoided by appropriate phasing of the AMEP project.
Please confirm:
(i) what point the discussions with DRAX have reached.

We note that the applicant has proposed an alternative option (paragraph 1.2(b)), which considers using land at Halton Marshes to provide an alternative mitigation area (temporary wet grassland). We would refer the Examining Authority to our oral representations made during the Issue Specific Hearing on Marine Matters held on 13th September 2012 and our written summary submitted on 24th September 2012 regarding this proposal, and in particular the difficulties of this being within the Able Logistics Park site and on an area that may be required for urgent flood defence works.

General Comments
Other parties have made various comments in respect of the Examining Authority's 2nd Questions, which cross-cut with issues within our remit. As the applicant is currently producing additional supplementary information in relation to these issues (as indicated in their signposting document of 24th September 2012), we do not think there is any merit in us commenting on other parties comments at this time. We are awaiting receipt of the following additional information from the applicant, which will assist us in providing more constructive comments to the Examining Authority in due course on:

- gravel modelling, including calibration and validation;
- additional ecological assessment in relation to HU082 and HU080;
- updated and amended Water Framework Directive Assessment;
- revised capital dredge and disposal assessment;
- updated interpretation of Chapter 8 in the light of all the additional modelling and assessment work undertaken;
- updated in-combination and cumulative assessment to reflect the EA's previous comments.

The submissions by Associated British Ports (Unique reference no.10015525) in June 2012 and on the 24th September 2012 include some pertinent points, raised within Mr Whitehead's evidence. Some of the issues raised by Mr Whitehead are similar in nature to those raised by the EA in its correspondence with the applicant during the last two years. Once we are in receipt of the above additional information, we should be able to provide you with more clarity on if/where there are still significant outstanding issues.

Additional information requested by the Examining Authority
The information held by the EA relating to bird data and sediment type, as mentioned in our submission of 24th September 2012, has now been received from our archives. There is limited bird data and it does not appear to be of any additional benefit to that already available, as it is not specific to the North Killingholme foreshore. We have specific information on the sediment at East Halton foreshore, which may assist the Examining Authority in understanding the nature of the mud in this area of the estuary. We have provided a Technical summary of this information (attached as Appendix A), and a map showing the location where these samples were taken (attached as Appendix B - A6148 Drawing C2) which we think will be of most use to the Examining Authority in the limited time available within the examination period. Should the Examining Authority wish to see the individual soil logs, the EA has this information available and would be pleased to provide this, but these are very large datasets. This information specifically relates to the East Halton foreshore area, but this location is the closest location to the North Killingholme foreshore for which we have detailed information available.

APPENDIX A

Technical Note – Embankment Stability Analysis

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Technical note

Project	Halton Marshes	**Date**	28 August 2003
Note	Embankment Stability Analysis	**Ref**	KMLHBF/22
Author	ELC		

1 Introduction

1.1 This technical note discusses the methodology and conclusions of a stability analysis undertaken on the existing flood defence embankment at Halton Marshes on the Humber Estuary.

1.2 The embankment is approximately 4m high, with a concrete toe beam at the base and a wave wall and splashdeck at the crest.

1.3 Erosion at the toe of the embankment, exposing the toe beam structure, has led to concerns of its long-term stability.

1.4 The stability analysis is based on the results of a ground investigation and associated laboratory testing undertaken at the site between June and September 2003.

2 Site Investigation details

2.1 Eight boreholes were undertaken at the site during June and July 2003, five along the crest of the embankment and three along the foreshore at the toe of the embankment.

2.2 Samples were collected and laboratory testing undertaken during August and September 2003. The following laboratory tests were undertaken:

- ICRCL Table 3 and 4 chemical tests
- Sulphate testing (in accordance with BRE Special Digest 1, 2001)
- Moisture content
- Plasticity index
- Particle Size Distribution
- Undrained triaxial tests
- Drained triaxial tests
- Oedometer tests

2.3 Groundwater monitoring of the borehole installations was also undertaken on three dates in June and July 2003.

3 *Site Geology*

3.1 From the boreholes, the geology at the site was found to consist:

(a) Embankment Fill

The embankment was found to be constructed of a brown grey sandy gravely clay/very clayey sandy gravel, with angular to subrounded cobble sized fragments of concrete, fine to coarse sand and angular fine to coarse grained gravel of mainly concrete, with occasional mudstone, sandstone, clinker and ash. At the northern extreme of the embankment, the made ground was found to consist of a slightly sandy gravely silt with gravel of concrete, sandstone and mudstone as found along the rest of the embankment.

The fill was logged as very dense in places (BH3 and BH5 at 1 to 1.5m depth). This is probably due to concrete obstructions in the embankment and is not indicative of \ whole.

The embankment fill was found to range from around 3 to 4m in thickness along the section.

Moisture content of the fill materials was recorded at between 9 and 38%. Plasticity index values ranged from 11 to 33%, liquid limits from 37 to 58% and plastic limits from 23 to 28%. Plasticity Index results indicate the materials to have a ϕ'_{crit} of around 24 to >30 (BS 8002:1994).

Two undrained triaxial tests on fill samples recorded values of Cu between 6 and 18kPa, indicating the deposits to be very soft (BS 5930 1999). Two SPT-N values recorded in the fill materials of 10 and 7 indicate a C_u of around 35 to 50kPa (based on CIRIA Report 143), which classes the deposit as soft to firm.

(b) Clay / silt

Beneath the embankment, a grey brown very soft to soft sandy clay/silt was encountered in the boreholes along the crest of the slope. Sand was recorded as fine. An organic odour was frequently recorded.

Along the foreshore of the section, the deposit was recorded as a very soft to soft grey locally mottled brown slightly sandy slightly gravely silt. Sand was recorded as fine. Occasional gravel sized fragments of concrete and chalk were present. An organic odour was frequently recorded.

The clay deposits were recorded as very soft to soft to a maximum depth of 20.4m below the embankment crest and a maximum of 10.2m below the foreshore at the toe of the embankment. No soil geology was encountered during the site investigation.

Moisture contents recorded in these deposits ranged from 5.5 to 59%.

The plasticity index values ranged from 15 to 39%, the liquid limits ranged from 35 to 63% and the plastic limits from 21 to 28%.

Undrained triaxial tests recorded values of C_u ranging from 6 to 35kPa, indicating the deposits to be very soft to soft (BS 5930, 1999).

Two drained triaxial tests recorded values of c' of 0 and 10kPa and values of ϕ' of 40° and 22° respectively. Based on plasticity data and BS 8002 1994, values of ϕ'_{crit} range from 23 to 30°.

Sulphate testing of samples from beneath the toe of the embankment recorded sulphate levels in a 2:1 soil to water mix of 0.81 to 1.7g/l, with values of pH ranging from 7.1 to 7.8. Based of BRE Special Digest 1, 2001, this indicates that a concrete class of ACEC Class AC-1s would be required for any buried concrete structures in this area.

4 Chemical Testing

4.1 Chemical testing of three soil samples from beneath the toe of the embankment were tested for the following substances:

- Free cyanide
- Elemental sulphur
- Acid soluble sulphate
- Arsenic
- Cadmium
- Chromium
- Total petroleum hydrocarbons (TPH)
- Boron
- Nitrate
- Lead
- Thiocyanate
- Sulphide
- Copper
- Polyaromatic hydrocarbons (PAH)
- Mercury
- Nickel
- Selenium
- Zinc
- Total cyanide

4.2 With regard to CLEA (Contaminated Land Environmental Assessment) Soil Guideline Values for the end-use of commercial/industrial, and where these values are not available, with regard to Dutch Target Values, all three samples were found to contain elevated levels of PAH (up to 421mg/kg) and one sample was found to contain elevated levels of TPH (59mg/kg).

4.3 Therefore any construction works around the toe of the embankment would need to assess the risk posed by these levels.

5 Soil Parameters

5.1 Based on the ground investigation results and the resultant laboratory testing, soil parameters were assigned for each soil type in order to carry out the stability analysis.

5.2 Parameters for the concrete structures and embankment revetment are assumed based on the recorded construction materials.

5.3 Undrained triaxial test data indicates an increase in undrained shear strength with depth in the clay/silt deposits across the site. Beneath the embankment (approximately 5 to 6m depth from the crest), values of C_u were recorded at 23-29, indicating that consolidation of the deposits directly beneath the embankment has increased their strength. This is reflected in the stability model, by the incremental increase in the C_u parameter with depth based on the undrained triaxial testing (as shown in Table 1) on both the material beneath the embankment itself and the material beneath the toe.

5.4 Table 1 below details the range of parameters and their source.

Table 1 Summary of soil parameters used in stability analysis

Soil Type	Soil Parameter (unit)	Range	Value used in stability analysis	Source
Concrete structures (wave wall/splash back and toe beam)	γd (Mg/m^3)	-	25	Assumed
	C (kPa)	-	500	Assumed
	ϕ (°)	-	45	Assumed
Revetment	γd (Mg/m^3)	-	19	Assumed
	C (kPa)	-	0	Assumed
	ϕ (°)	38-42	40	Assumed
Made Ground/ Embankment fill	γd (Mg/m^3)	1.76	1.76	Lab data
	C_u (kPa)	6-50	25	Undrained triaxial testing SPT data
	ϕ'_{crit} (°)	24->30	30	Plasticity results
	C' (kPa)	0-10	5	Assumed based on PSD gradings, density and descriptions.
Clay/Silt	γd (Mg/m^3)	1.68 to 1.91	1.79	Lab data
	ϕ'_{crit} (°)	23-30	26	Plasticity results
	Cu at 0 to 2m depth* (kPa)	6 - 9	8	Undrained triaxial testing
	Cu at 2 to 5m depth* (kPa)	11 - 18	15	Undrained triaxial testing
	Cu at 5 to 10m depth* (kPa)	18 - 20	20	Undrained triaxial testing
	Cu at 10 to 20m depth* (kPa)	-	50	Assumed
	Cu under embankment (kPa)	24 - 29	26	Undrained triaxial testing
	ϕ' (drained) (°)	22-40	30	Drained triaxial testing and correlations with plasticity (BS8002 1994)
	C' (drained) (kPa)	0-10	0	Drained triaxial testing

* Depth taken from the base of the embankment

6 Groundwater

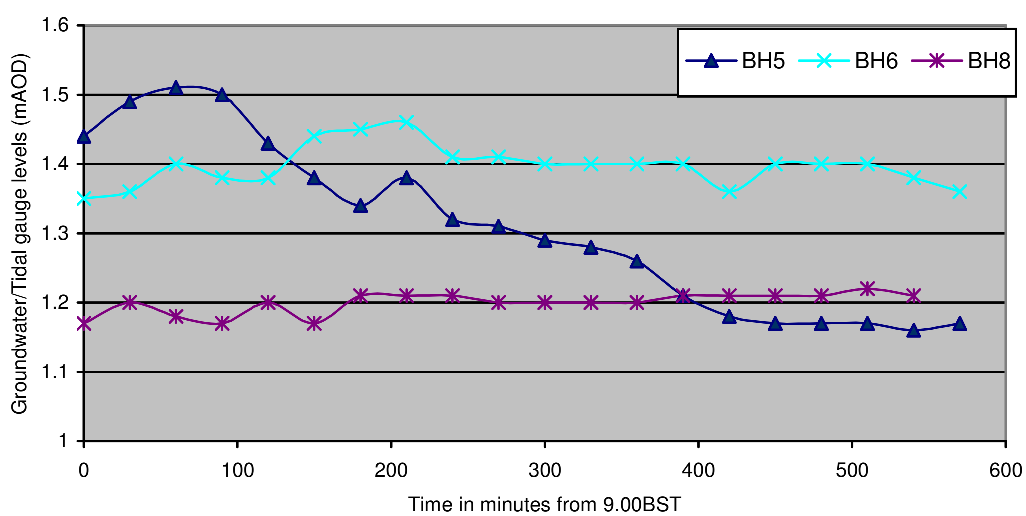
6.1 Groundwater monitoring was carried out in standpipe piezometers installed in boreholes during the ground investigation. Readings were taken on the 27th of June the 30th of June and the 3rd of July 2003, with a set of 20 readings undertaken in each borehole throughout the day (between 9.30am and 6.30pm at half our intervals) on the 3rd of July 2003. Therefore, only summer groundwater levels have been recorded on these dates and levels would be expected to be higher than this when spring high tides coincide with high groundwater levels.

6.2 Tide levels recorded by the Environment Agency indicate that high tide levels on the 27th and 30th of June and the 3rd of July were between 2.6 and 3.2mAOD.

6.3 The lowest tide levels were not recorded by the Environment Agency in these data sets as the levels dropped below the level of the measuring gauge. As with the groundwater monitoring, this data is only relevant to high tide levels during a specific period of the year and during high spring tides the levels would be expected be higher.

6.4 A summary of the groundwater and recorded tidal data is shown in Figure 1 below:

Figure 1: Groundwater levels recorded in boreholes on the 3rd of July 2003



6.5 Figure 1 indicates that high groundwater levels in the summer months could be expected to be around 1.2 to 1.5mAOD across the site.

6.6 Groundwater monitoring undertaken on the 27th and 30th of June coincided with reducing tide levels, not with the time of day at which the lowest tidal point occurred. Therefore the groundwater levels would be expected to reach lower levels than those measured during the monitoring period.

6.7 The groundwater levels recorded in boreholes BH5 and BH6 show a time lag response to the tidal levels recorded by the Environment Agency of approximately one hour at borehole BH5 and three and a half hours at BH6.

6.8 Borehole BH8 did not record such a marked peak groundwater level in response to the tidal fluctuation as boreholes BH5 and BH6.

6.9 The groundwater levels recorded in BH2, however, recorded a level of 0m below ground level throughout the monitoring on the 3rd of July, indicating that the borehole was possibly flooded during this day. Levels recorded in this borehole on the 27th and 30th of June recorded levels of 3.8 and 3.68m below ground level respectively.

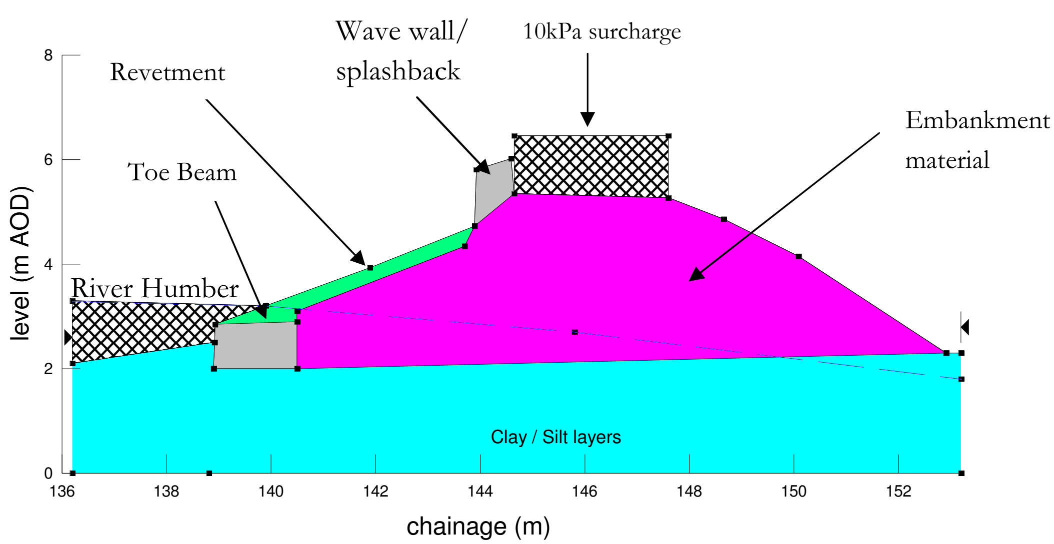
7 Slope Stability Analysis

7.1 Based on the geometry of Section 5 (drawing HG 3900-2) and the soil parameters in Table 1, an analysis was carried out using SLOPE/W software.

7.2 An acceptable factor of safety for the embankment was taken as 1.3.

7.3 The basic geometry of the slope (as used in the analysis) is as follows:

Figure 1: Basic Embankment Geometry



7.4 A surcharge of 10kPa was applied to the crest of the embankment to allow for any vehicular movement across it.

7.5 Three groundwater conditions were modelled, namely:

- Low tide
- High tide
- Rapid draw-down conditions following a very high tide and/or flood conditions (critical groundwater case)

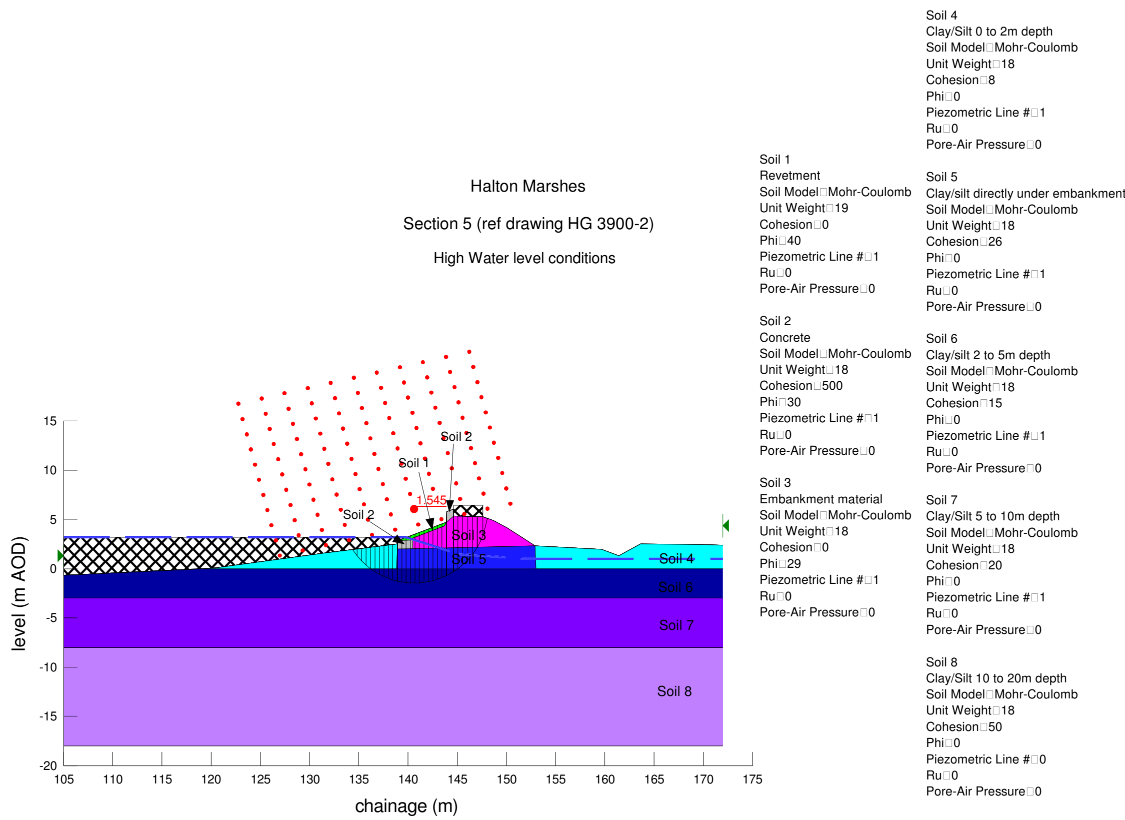
7.6 For each groundwater condition, the embankment was analysed for the following failure modes:

- Rotational sliding of the embankment as a whole
- Block failure of the toe beam for different levels of foreshore erosion.

8 ***Results of Stability Analysis***

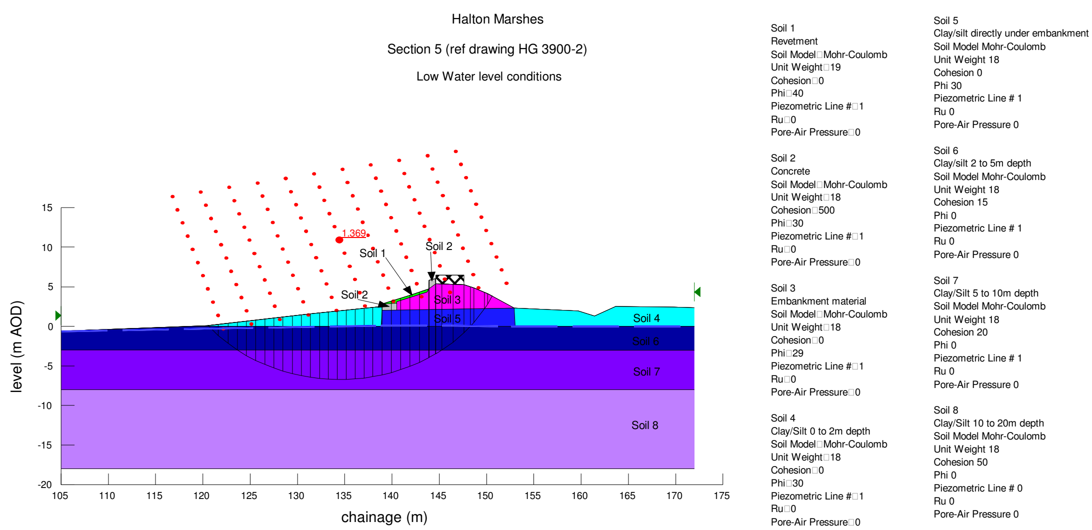
(a) Rotational failures

For high tide water levels (based on Environment Agency recorded levels for June and July), the minimum factor of safety recorded was 1.55, which is above the recommended factor of 1.3, indicating the slope to be stable in its current condition (as shown below).

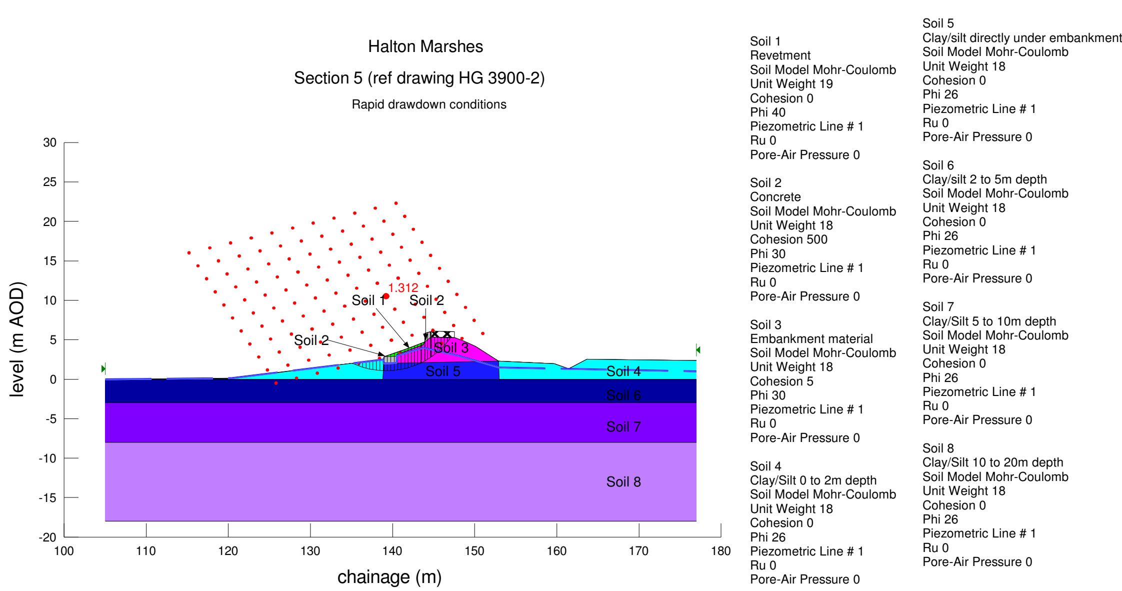


River levels could be expected to reach higher levels than those illustrated above, however this would only be expected to increase the stability due to the increase toe weighting from the water.

For low tide levels, the minimum factor of safety recorded for the embankment was 1.37 (as detailed below), also above the recommended factor of 1.3, indicating the embankment to be stable at low tides under current conditions.



For the critical case conditions of rapid draw-down (allowing for pore-water pressure effects), the minimum factor of safety recorded was 1.31 (as detailed below), indicating the slope to be on the limits of stable conditions.



The highest groundwater levels as part of the rapid draw-down scenario have been estimated due to a lack of groundwater monitoring.

(b) Block Failure

An analysis was carried to ascertain the stability of the embankment with regard to block failure of the toe beam (the toe beam structure is indicated in the diagram in Section 7.3). The analysis modelled the effects of erosion occurring at the base of the toe beam over time.

The analysis was carried out based on the critical conditions of rapid drawdown, which could occur following high tides and/or flooding conditions.

The figure below shows an example of the minimum factor of safety achieved if 0.8m of erosion were to occur at the site. As shown the minimum factor of safety is 1.14, indicating that the slope would become unstable if such erosion were to occur. Similar analyses were carried out for stages of 0.2m of erosion at the foreshore face of the toe beam in order to ascertain the level of erosion at which the beam would become unstable (i.e. the factor of safety drops below 1.3). The results are detailed in Table 2 below.

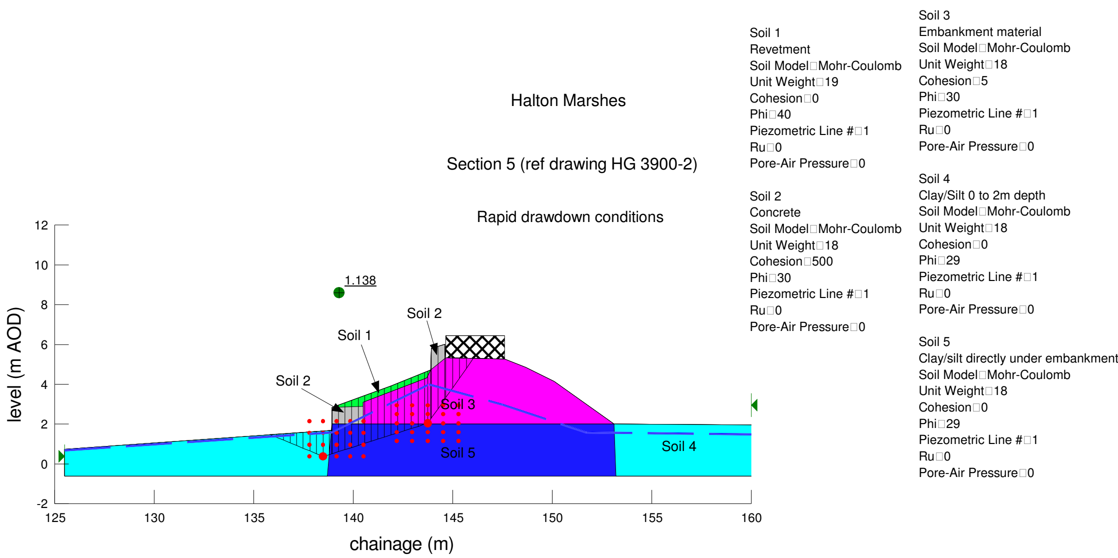


Table 2 Results of block failure analyses

Amount of erosion at the foreshore	Minimum factor of safety for block failure
No erosion	1.3
0.2m	1.27
0.4m	1.21
0.6m*	1.16
0.8m	1.14
1m	1.07

* point at which foreshore drops to a level below the base of the toe beam (i.e. toe beam is completely eroded out)

8.2 Erosion along the face of the toe beam would also be expected to affect the global rotational stability of the embankment, as weight would be removed from the toe of any potential slip surfaces.

9 Conclusions

9.1 The global stability analysis has indicated that the embankment exceeds a factor of safety of 1.3 for the three groundwater conditions modelled, indicating that the embankment is stable in it's current condition for the climatic conditions modelled.

9.2 For conditions of rapid draw-down, the factor of safety achieved is close to 1.3, indicating that the slope is on the limit of stability in it's current state and any changes in geometry (i.e. erosion, loading at the crest, extreme climatic conditions) could result in a reduction in this stability.

9.3 The erosion analysis indicates that if erosion of material from the face of the toe beam structure continues at the site, the toe beam is at risk of sliding failure where if the foreshore is eroded even by relatively small amounts (approximately 0.2m).

9.4 Erosion along the toe of the embankment would also be expected to reduce the rotational stability of the embankment as a whole to less than 1.3, as erosion would reduce the stabilising toe weight.

9.5 Limitations of this stability analysis include:

- Groundwater conditions within the embankment were based on limited monitoring data taken in June and July only, therefore there is some uncertainty over the accuracy of the high water levels modelled.
- Due to the low strength of the clay soils at the base of the embankment, any construction disturbance in the area could potentially reduce the material strength, consequently reducing the stability of the embankment.

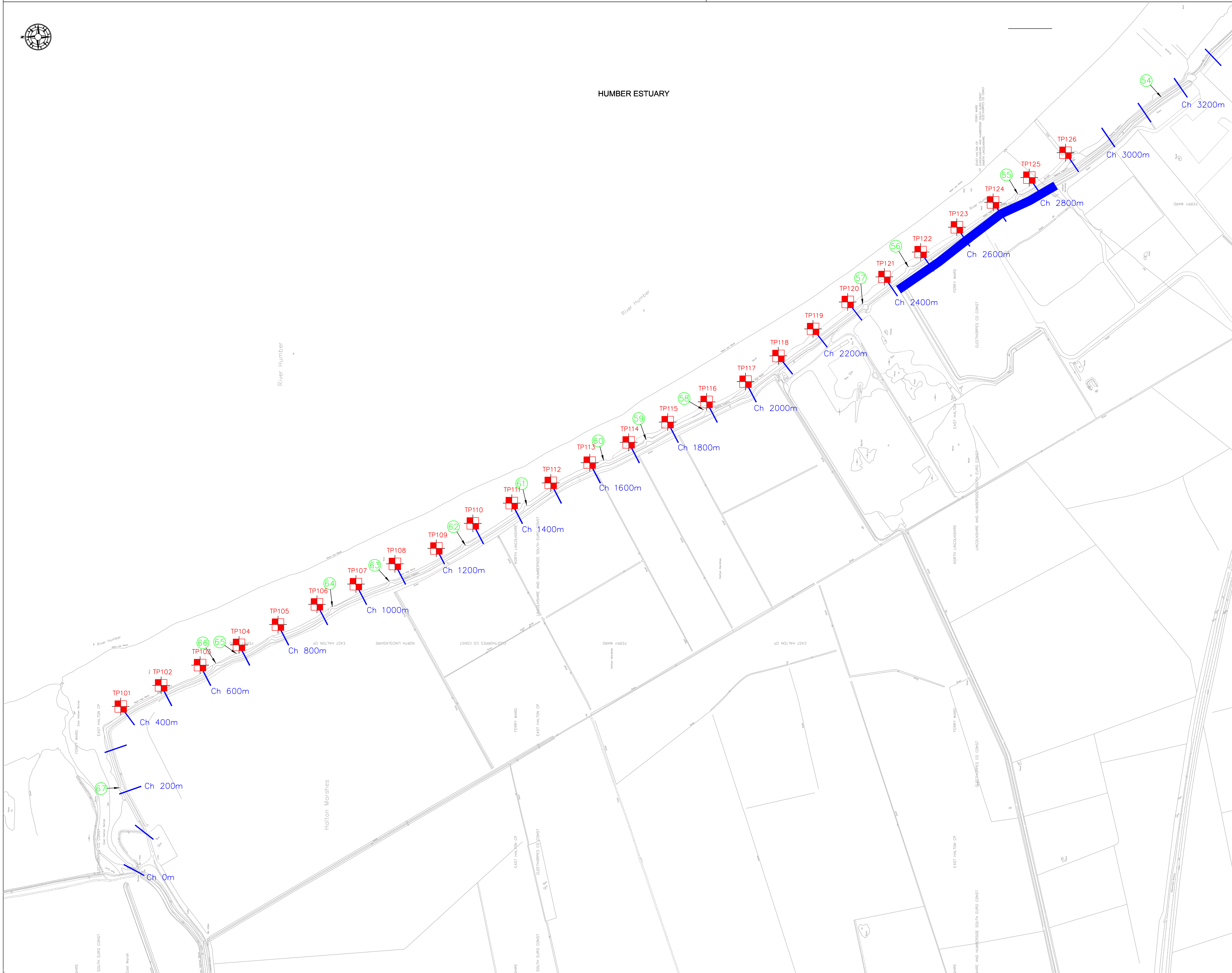
- Lateral variation in the nature of the embankment fill and consequently soil parameters meaning that sections of the embankment may be more stable than the modelled condition (where conservative parameters were used).

9.6 Chemical testing on samples from the toe of the embankment has indicated the presence of elevated polyaromatic hydrocarbons and total petroleum hydrocarbons with regard to Dutch Target Values (in the absence of CLEA Soil Guideline Values), therefore, any excavation of materials in this area will require disposal to an appropriately licensed landfill site.

9.7 Therefore based on this analysis, measures to reduce the risk of continued erosion at the toe of the embankment would be required in order to maintain its stability in the long term as erosion of the foreshore below the base of the toe beam will potentially lead to slope failure by either block or rotational modes.

APPENDIX B

A6148 Drawing C2 – Sample locations



GENERAL NOTES

LEGEND TO SYMBOLS

Trial Pit Location

Scale: 1:4000

0 40m 80 160 240 320

Rev	Drawn	Date	Approv.	Date	Modification Details
A	NH	14.06.05	MH	14.06.05	OS Coords

AMENDMENTS

Title

SITE PLAN

Project

HALTON MARSHES FLOOD DEFENCE IMPROVEMENTS

Client

Environment Agency

Soil Mechanics

Date	Drawn By	Approv. By
06.11.06	AW	RVH

Sheet Size	Scale	Project No
A1	1:4000	A6148

Drawing No	Rev
C2	0