



Rockview Farm

Flood Risk Assessment

Tobin Consulting Engineers

12th June 2007

Final Report

9S4067A0

A COMPANY OF



ROYAL HASKONING

**HASKONING UK LTD.
COASTAL & RIVERS**

19 Baggot Street Lower
Dublin 2
Ireland
+353 (0)1 634 5353 Telephone
0035316345305 Fax
info@royalhaskoning.com E-mail
www.royalhaskoning.com Internet

Document title Rockview Farm
Flood Risk Assessment
Document short title
Status Final Report
Date 12th June 2007
Project name Rockview Farm Flood Risk Assessment
Project number 9S4067A0
Client Tobin Consulting Engineers
Reference 9S4067A0/R/330004/Irel2

Drafted by Niels Minnen
Checked by Owen McManus
Date/initials check *[Signature]* 8/8/07
Approved by Hamish Hall
Date/initials approval *[Signature]* 8/8/07

SUMMARY

In order to inform the preparation of a Local Area Plan for 84.48ha lands at Rockview Farm, Dundalk, Dundalk Town Council (DTC), requested an assessment of the flood risk associated with the site and recommendations concerning management of any risk, including improved coastal defences. Accordingly, Tobin Consulting Engineers, who are part of a team of consultants engaged by DTC to prepare the Local Area Plan, commissioned Royal Haskoning to assess the flood risk and make appropriate recommendations.

At present the land is farmed and there is an existing embankment which extends along the seaside boundary offering protection from extreme tidal events. DTC have in the past commissioned a study to assess the feasibility of providing a coastal walk and cycle path along the existing embankment from Dundalk to the village of Blackrock to the south. The flood risk assessment has thus considered the proposed coastal walk in the context of ensuring that it could easily be facilitated within the provision of any improved flood defences for the site.

An important issue for consideration in the proposed development and particularly the provision of the coastal walk and cycle path is the environmental designations across Dundalk Bay, which includes a Special Protection Area (SPA), Special Area of Conservation (SAC) and a proposed Natural Heritage Area (pNHA). This issue, whilst commented on within this report, has been dealt with in detail in an ornithological assessment report which was prepared by Biosphere Environmental Services for Tobin Consulting Engineers

As part of the flood risk assessment a data collection exercise was undertaken to obtain as much relevant data as possible for the site. In particular this included tidal data to enable an assessment of extreme tidal conditions and historical aerial photographs to investigate the evolution of the saltmarsh which borders the site. In addition consultation meetings were held with both DTC and the Department of Communications Marine and Natural resources (DCMNR).

The data collection found that virtually no recorded tide data exists for Dundalk which could be used to aid in the assessment of extreme tide conditions at the site. During the consultation meeting held with DCMNR it was established that as part of their Coastal Strategy Study for Ireland extensive tidal modelling of the Irish Sea was being undertaken by RPS Consulting Engineers. In light of the limited tidal data available RPS were contacted to enquire about the availability of data in the Irish Sea with a view to verifying the tidal analysis which had already been completed. RPS subsequently provided details of astronomical tide and surge relationships between Dublin and Dundalk from their model and these were used to refine the tidal analysis and aid in the establishment of extreme tide levels at the site.

The analysis determined extreme tide levels at Dundalk associated with both the event of the 1st February 2002 and also a 200 year return period. The results indicated an extreme level on the 1st February 2002 of 3.59mODM, which falls within the anecdotal range quoted of 3.36 – 3.65mODM. The report concluded that any proposed development at the site should be protected against coastal flooding for events up to and including a 200 years event and the tidal analysis indicated this to have a present day level of 3.7mODM. Considering a design horizon until the end of this century, it was

concluded that when allowance for sea level rise is taken into account a design water level of 4.1mODM should apply for the proposed development site. In addition to extreme tide levels, an assessment was made of the likely maximum wave climate at the site which, given the extensive width of mud and sand flats and saltmarsh fronting the site, will be depth limited in nature. Based on an assessment of depth limited conditions the maximum waves that are likely to be experienced along the toe of the embankment are between 0.75 – 1.2m. As such it is recommended that the embankment should be armoured with a proprietary concrete armour system such as Armourflex.

A detailed walk over inspection of the existing embankment was undertaken from Shore Road to the Blackrock Road in the south. The inspection concluded that whilst the embankment offers some degree of protection it is generally in a poor condition and does not provide adequate protection for the future. In particular the embankment north of the proposed development site is in a poor condition and this is reflected by the fact that several breaches occurred in it during the event of the 1st February 2002. The main coastal embankment over the site is in a better overall condition, particularly the section south of the drainage outfall, where the embankment has a wide crest. Nevertheless it is still relatively low and will not provide appropriate flood defence needs in the future. The return embankment over the southern boundary to Blackrock Road is low and in a poor condition.

Based on the results of the site inspection, topographic survey and extreme tide analysis it was concluded that the standard of protection offered by the existing main coastal embankment was in the region of 40 years return period. This would reduce for the embankment to the north of the site and over the southern boundary. Whilst this could be acceptable for low density rural or agricultural lands, it is not acceptable for a significant residential development such as proposed for the Rockview Farm site.

In view of the importance of the saltmarsh fronting the site in terms of reduced wave heights and hence coast protection for the site, an assessment of the evolution of the saltmarsh was undertaken using historic aerial photographs. Photographs for the years 1974, 1995 and 2004 were available and when compared showed that the saltmarsh has been accreting over the past 30 years. This also reflects the view of the ornithological assessment report prepared by Biosphere Environmental Services for Tobin Consulting Engineers which states that the saltmarsh has grown considerably in the 20th Century.

For the study a number of options have been developed which provide a level of protection against coastal flooding for all events up to and including a 1 in 200 year return period until the end of this century. The options have been developed using design criteria based on the latest national and international best practice for embankment flood defence structures.

Three general options for achieving the required level of protection have been considered using different site land fill and embankment levels. Within these options two embankment profile options have been considered which can facilitate the proposed coastal walk in different locations. Each of the options are summarised below.

Option 1 – Improve Main Coastal defence

This option considers improvements to the main coastal and southern return embankments and the provision of new embankments along the northern boundary of the site and along the Blackrock Road to the south. The option does not consider raising the level of the site. The new main coastal embankment would need to be raised to a level of 6.25mODM with armour placed over the front face.

Option 2 – Improve Main Coastal Defence and Raise development Lands

This option considers improvements to the main coastal defence but to a lesser extent than option 1. It also allows for the provision of improvements to the southern embankment and a new northern boundary embankment. In addition to this the option allows for the infill of the proposed development site to a level greater than the design 200 year extreme level in 2100. The new main coastal embankment would need to be raised to a level of 5.6mODM and armour placed on the front face and the crest. The proposed development lands would be raised to a level of 4.25mODM for the residential areas and 3.5mODM for the amenity areas.

Option 3 – Improve Main Coastal Embankment and Provide New Secondary Embankment.

This option considers improvements to the main coastal embankment but to a lesser extent than option 2 and the provision of a new secondary embankment at the interface between the residential and amenity areas. It also considered the provision of a new northern boundary embankment, improvements to the southern boundary embankment and the provision of a new embankment along the Blackrock Road towards Blackrock Village. The new main coastal embankment would need to be raised to a level of 5.0mODM and armoured on the front, crest and rear slope. The secondary embankment would also be placed at this level. In addition the option allows for a limited amount of infill to a level of 3.5mODM over the residential area whilst leaving the amenity lands at a level of 3mODM.

Within each of the above options two embankment profiles were considered. Embankment 1 consists of a typical trapezoidal profile with a 4m wide crest and berm along the rear toe. Within this embankment profile the proposed coastal walk could be placed on the crest or along the rear toe. The second profile, Embankment 2, consists of a bermed embankment with a 4m wide berm placed on the rear slope. This berm could facilitate the proposed coastal walk and would be placed at a level sufficient to minimise disturbance to the waterfowl using the SAP while enabling users to enjoy views of the surrounding area.

Of the options presented it is considered that option 2 incorporating embankment 2 will provide the most sustainable means of managing flood risk for the proposed development site. It provides adequate protection to the proposed development lands through raised ground levels and also caters for the longer term coast protection needs of the area and protection of the amenity lands through the provision of improvements to the existing embankment. Embankment 2 is also considered to provide a more robust flood defence structure and provides the best foundation for the provision of the proposed coastal walk by balancing the need for minimising disturbance to the SPA against maintaining the interaction of the potential users with the surrounding area.

CONTENTS

	Page
1 INTRODUCTION	1
1.1 General	1
1.2 Description of the site	2
1.2.1 Location	2
1.2.2 Description of proposed development	3
1.2.3 Environmental Designations	4
1.2.4 Historic Flood risk	5
2 CONSULTATION, DATA COLLECTION AND REVIEW	7
2.1 Consultation	7
2.1.1 General	7
2.1.2 Dundalk Town Council	7
2.1.3 Department of Communications, Marine and Natural Resources	8
2.2 Data Collection and Review	9
2.2.1 Topographical data	9
2.2.2 Tide data	9
2.2.3 Reports	11
2.2.4 Aerial Photographs	11
3 SITE INSPECTIONS	12
3.1 General	12
3.2 Embankment Inspection	12
3.2.1 Section 1: Shore Road to Start of Firing Range (Ch 0 to 653)	12
3.2.2 Section 2: Firing range to drainage ditch outfalls (chainage 653 to 1275)	14
3.2.3 Section 3: Drainage ditch outfalls to end of wide embankment (chainage 1275 to 1700)	16
3.2.4 Section 4: End of wide embankment to end of main coastal embankment (chainage 1700 to 2050)	18
3.2.5 Section 5: Embankment Perpendicular to Main Embankment extending back to the Blackrock Road (chainage 2050 to 2475)	19
3.3 Other Site Features	21
3.3.1 Drainage Ditches	21
3.3.2 Drainage Ditch Outfall	22
3.3.3 Northern boundary	24
3.3.4 R172 south of the site	25
4 ASSESSMENT OF COASTAL CONDITIONS	26
4.1 Assessment of Extreme Tide Levels	26
4.1.1 Methodology	26
4.1.2 Method 1: Jerzy Graff Relationship	26
4.1.3 Method 2: Astronomical Tide and Surge Transfer Relationship	28
4.1.4 Coastal Strategy Study for Ireland – Tidal Modelling	31
4.1.5 Final Extreme Water Levels at Dundalk	34
4.2 Sea Level Rise	35
4.3 Assessment of Extreme Wave Conditions	36

5	ASSESSMENT OF FLOOD HAZARD	37
5.1	General	37
5.2	Comparison of Embankment Levels and Extreme Tide Levels	37
5.3	Existing Standard of Protection	38
5.4	Saltmarsh Evolution	39
6	OUTLINE FLOOD ALLEVIATION OPTIONS	42
6.1	General	42
6.2	Design criteria	42
6.2.1	Standard of Protection	42
6.2.2	General shape of the embankment	44
6.2.3	Crest level of the embankment	45
6.2.4	Armouring of the embankment	46
6.3	General options	46
6.3.1	Option 1 – Improve Main Coastal Defence	46
6.3.2	Option 2 – Improve Main Coastal Defence and Raise Development Lands	48
6.3.3	Option 3 – Improve Main Coastal Embankment and Provide New Secondary Embankment.	50
6.4	Embankment options	51
6.5	Southern Boundary	54
6.5.1	Proposed option south	54
6.5.2	Alternative south	54
6.6	Costs of the options	54
6.7	Evaluation of options	55
7	CONCLUSIONS AND RECOMMENDATIONS	56
7.1	Conclusions	56
7.2	Recommendations	57

APPENDICES

Appendix A	Site Plans
Appendix B	Minutes of Consultation Meetings
Appendix C	Embankment Inspection Photographs and Sections
Appendix D	Tidal Analyses
Appendix E	Saltmarsh Evolution
Appendix F	Option Drawings
Appendix G	Product Information Armourflex

1 INTRODUCTION

1.1 General

In order to inform the preparation of a Local Area Plan for 84.48ha lands at Rockview Farm, Dundalk, Dundalk Town Council (DTC), requested an assessment of the flood risk associated with the site and recommendations concerning management of any risk, including improved coastal defences. Accordingly, Tobin Consulting Engineers, who are part of a team of consultants engaged by DTC to prepare the Local Area Plan, commissioned Royal Haskoning to assess the flood risk and make appropriate recommendations.

The flood risk assessment deals with protection against coastal flooding and coastal erosion. The initial flood alleviation option discussed with DTC centred on raising the ground levels over the site, particularly over the residential areas, to ensure that the ground floor levels of the dwellings are higher than the threshold value currently requested by the council. However, the council have considered this in isolation to be inadequate and have requested that some consideration be given to the improvement of the existing coastal defences to provide better protection to the amenity lands which form an integral part of the overall development.

At present the land is farmed and there is an existing embankment which extends along the seaside boundary offering protection from extreme tidal events. A more detailed description of the site and the existing defence is provided in sections 1.2 and 3.

The council have in the past commissioned a study to assess the feasibility of providing a coastal walk and cycle path from Dundalk to the village of Blackrock to the south. That study considered the coastal walk to be provided along the existing sea defence embankment bordering the site. As such the flood risk assessment was required to consider the coastal walk in the context of ensuring that it could easily be facilitated within the provision of any improved flood defences for the site. An important issue for consideration in the development and particularly the provision of the coastal walk and cycle path is the extensive number of environmental designations across the area. These have been discussed briefly in section 1.2.4 and are dealt with in more detail by the Local Area Plan (LAP) and Environmental Impact Statement (EIS) which are being prepared for the development of the site by Tobin Consulting Engineers and a team of consultants.

Tobin Consulting Engineers, on behalf of their client, have been requested to provide a robust approach to managing flood risk at the site and have been asked to provide both raised ground levels and improved flood defences. As such they have requested that Royal Haskoning undertake an assessment of the flood risk issues at the site with a view to determining the best approach to managing the flood risk for the future.

This report addresses these issues and sets out the work that has been undertaken to arrive at the preferred flood mitigation option for the site. Section 2 sets out the extensive data collection exercise which was undertaken, including consultation with relevant authorities. Section 3 provides a detailed description of the existing coastal flood embankment and the site in general and in particular considers its condition and appropriateness as a flood defence structure. Section 4 sets out details of the coastal conditions at the site including extreme tidal analysis, wave climate and climate change.

Section 5 considers the findings of the site inspection and relates it to the extreme tide and wave analysis to consider the current flood risk to the site. Appropriate flood mitigation options are presented in section 6, each of which would provide adequate protection for the proposed development site against coastal flooding and erosion up to the required indicative levels currently being recommended as the national standards for Ireland. This section also considers in outline the options for the incorporation of the walkway within the new flood defence. This includes its position and elevation but does not include any detail of the walkway itself or the associated costs.

1.2 Description of the site

1.2.1 Location

The Rockview Farm site lies on the southeast of the town of Dundalk, as indicated on Figure 1-1. It is around 85 hectares in area and is currently protected from the sea by an earth embankment fronted by a wide expanse of saltmarsh. The site is bordered by the wayleave to the firing range and the firing range itself on the north and by Blackrock Road (R172) on the west. On the south side the embankment connects with the R172, thus forming the boundary of the site. The site boundaries are shown by the red line presented in Figure 1-2 below and detailed plans of the site are shown in Appendix A.



Figure 1-1 Location of Rockview Farm site

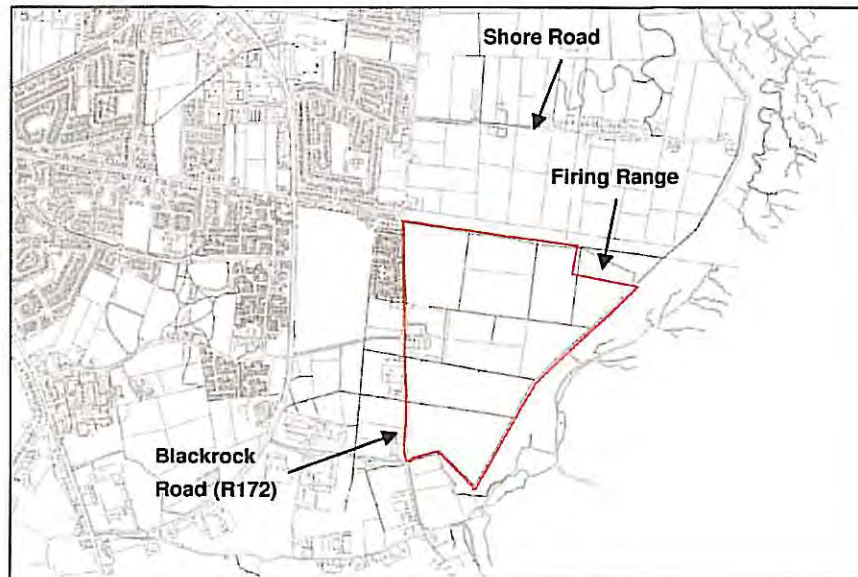


Figure 1-2 Boundaries of Rockview Farm site

1.2.2 Description of proposed development

The site and general overview of the proposed development is shown in Figure 1-3 below. All of the area shown hatched including the saltmarsh but excluding the firing range is in ownership of Topfloor Properties. Only the green and yellow hatched areas are proposed for development. The developments proposed roughly entail amenity areas in the strip (green hatched) adjacent to the existing embankment and residential areas (yellow hatched) on the rest of the lands. The firing range and the saltmarsh will be maintained as they are. More details of the specific development can be found within the LAP.

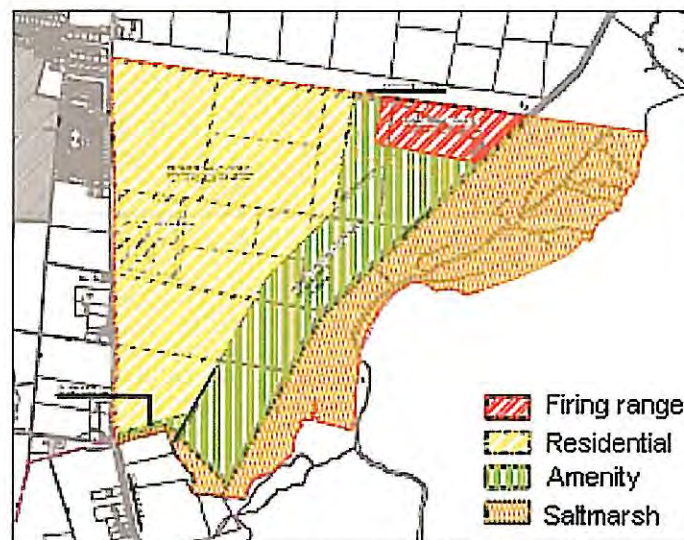


Figure 1-3 – Overview of Proposed Site Development

The development area and a large extent of the lands behind it, believed to extend back as far as the Dublin Road, drains via open drainage ditches from the west to the east

through the development lands into the open drain behind the embankment. This drain then discharges via two 900mm pipes controlled by Tideflex non return valves through the embankment into a natural channel on the saltmarsh.

Hydrological aspects of the site and the wider catchment have been evaluated as part of the overall assessment of the site by Tobin and have been discussed in the EIS which they have prepared for the site.

1.2.3 Environmental Designations

On the sea side the development lands border Dundalk Bay, which is a Special Protection Area (SPA), Special Area of Conservation (SAC) and a proposed Natural Heritage Area (pNHA), see Figure 1.4 below. Furthermore Dundalk Bay is an internationally important location for overwintering waterfowl.

Careful consideration will need to be given to the impact which the development and in particular the improved flood defence and new coastal walk could have on these designated areas. In particular disturbance impacts could exist both during construction and operation of the new flood defence and walkway. The impacts would be of short duration during the construction and mitigation measures could be put in place during this stage to minimise the impact. However, disturbance during the operation phase of the coastal walkway will need careful consideration, in particular the location of the new walkway and segregation of users (and their pets) from the designated area. The main concern over the area is the roosting sites for birds that can in particular be found at the northern and southern extent of the saltmarsh.

Whilst a full environmental assessment of the impacts of the development, flood defence and walkway is not provided in this report, issues in respect of disturbance in the context of the flood defence and walkway have been discussed in the option section of the report (Section 6). A fuller evaluation of environmental issues for the development can be found in the EIS and LAP for the site.



Figure 1-4 Environmental Designations Surrounding the Site

1.2.4 Historic Flood risk

The site is currently protected by an earth embankment that extends from the south of the site to the northern boundary at the firing range. The embankment extends further to the north and has been assessed within this study up to the Shore Road, see Figure 1-2. The general level of the main coastal embankment varies from a lowest level of 3.42m Ordnance Datum Malin Head (mODM) to a highest level of 4.59mODM and is generally around 3.9 to 4.1mODM over much of its length. The condition of the embankment has been assessed through detailed site inspections, the results of which have been presented in Section 3.2.

Historically there are records of coastal flooding having occurred at locations adjacent to the Rockview Farm site. However, it is believed that coastal flooding of the Rockview Farm site has never occurred. There is no evidence of breaches having occurred in the embankment at the site. The only recorded breaches in the embankment occurred during the February 2002 event and these were all located to the north of the site. The location of the breaches was recorded by a



Figure 1-5 – Flooding on 1st February 2002 as seen from Shore Road

topographic survey completed for a coastal walk study in December 2002. Where these breaches occurred the adjacent farm land was flooded extensively to depths of between 0.5 – 1 metre. Figure 1-5 shows a photograph of the flooding that occurred adjacent to the Shore Road during the 1st February 2002 event.

The level to which the extreme tide rose on the 1st February 2002 at Dundalk has been reported as 3.65mODM by DTC during the consultation meeting held on 14th March 2007. This level was not determined as a result of tide gauge measurement but is believed to result from a level taken on a building near the Dundalk Harbour Commissioners offices following the receding tide. Discussion with representatives of Louth County Council indicated similar levels (circa 3.6mODM) although other reports reviewed for the development and the coastal walk tend to indicate other slightly lower historic levels. Previous reports for the development of Rockview Farm have quoted a maximum historic water level of 3.36mODM, although it is not known if this relates to the February 2002 event. Previous coastal walk reports¹, whilst not quoting a specific value, have suggested a return period associated with the February 2002 event of around 50 yrs which based on the tidal analysis quoted at that time would relate to a water level of around 3.28mODM. This value is lower than all other quoted values and as it was taken

¹ Dundalk to Blackrock Coastal Walkway, Feasibility Study, January 2003, RPS Kirk McClure Morton. Two page extract from report dated August 2005, RPS Kirk McClure Morton.

from a 2 page extract from the report it is not possible to comment further on its source or validity.

On the south side of the development lands the embankment returns to meet the R172 and south of this point there are no more flood defences. The road varies in level between 2.5 and 3.8mODM, so in places is not sufficiently high to serve as a flood defence.

In front of the embankment there is a wide expanse of saltmarsh which helps to reduce the wave climate as it approaches the embankment during extreme tidal conditions. Under normal astronomic tidal conditions wave climate does not reach the embankment due to the presence of the saltmarsh.

2 CONSULTATION, DATA COLLECTION AND REVIEW

2.1 Consultation

2.1.1 General

As part of the study a limited amount of consultation was undertaken with relevant authorities who would have responsibility for coastal protection and flood risk issues. This included Dundalk Town Council (DTC) and Department of Communications, Marine and Natural Resources (DCMNR). A summary of the main issues discussed are presented in the following sections and full minutes of each meeting can be found in Appendix B.

2.1.2 Dundalk Town Council

A meeting was held with Dundalk Town Council (DTC) on the 14th March 2007. Catherine Duff (Town Council Engineer) and Peter McVeigh (Senior Executive Engineer) were in attendance for DTC. The council had been consulted on a number of occasions in the past in respect of the Rockview Farm development and had expressed their concern about flood risk issues at the site and their desire to see a new coastal walk integrated within the overall development.

The aim of the meeting was therefore to discuss DTC policies and requirements regarding planning and development in flood risk areas, discuss the specific concerns in respect of the flood risk at the Rockview Farm site and also their aspirations and vision for a coastal walk and cycleway. Furthermore in view of DTC having commissioned a number of previous reports on the coastal walkway the meeting was a valuable opportunity to ask questions and obtain data which could be of particular relevance for the study.

Full minutes of this meeting can be found in Appendix B. The main outcomes of the meeting are summarised below:

- Historically design levels for flood protection in coastal flood risk areas have varied over the years. At the moment a property threshold level of 4.15mODM is required, which consists of the highest (anecdotal) tide level (3.65mODM) plus 20 years climate change (0.2m) and freeboard (0.3m).
- A 1:200 year standard of protection will be required for the flood defences at the development site.
- The extent of this flood risk assessment is from the R172 in the south to the Shore Road on the north is considered acceptable by DTC.
- The highest tide level in Dundalk is quoted as 3.65mODM on the 1st of February 2002. The level of this tide was recorded based on the level the water reached on Quay Street in front of the Harbour Commissioners office. The tide level was estimated to be 18 inches over the quay level.
- Access to the embankment for maintenance will be a requirement. It is not clear yet who will be responsible for maintenance of the flood defence structure and the path.
- DTC does not specifically require the coastal walk to be on top of the embankment. Their vision and objective is for people using the path to be able to see the views of the surrounding area. Consultation with the National Heritage and Wildlife Service

would be required in order to explore the possibilities and in particular their impact on the designations.

- The only concern DTC has with raised ground levels as a flood defence is the impact this might have on the environmental designations. It was pointed out however that a design which would incorporate road levels that could be flooded with houses placed at a higher level would not be acceptable.
- The provision of good amenity facilities for the area is required for granting planning permission and was one of the prerequisites for zoning of the land for development in the initial case. As such these areas should not be subjected to frequent flooding but a slightly higher risk could be accepted than that for the residential areas.

2.1.3 Department of Communications, Marine and Natural Resources

A meeting was held with the DCMNR on the 22nd March 2007. The aim of the meeting was to obtain information on policies regarding flood risk and coast protection, discuss required standards of protection (SoP), extreme tide levels, climate change recommendations and planning guidance that may have resulted from the ongoing coastal strategy study for Ireland. In addition the meeting provided an opportunity to collect information and data which might be of use to the study.

Full minutes of the meeting can be found in Appendix B, the main outcomes are as follows:

- The DCMNR Coastal Strategy Study is still ongoing and no final results are available yet.
- Work to develop planning guidance for flood risk areas has just been commissioned.
- It is suggested that in the absence of Irish policy and guidance documents the relevant documents from the UK should be looked at. This would include Development and Flood Risk Guidance for the Construction Industry (CIRIA 624) and also PPG25 and PPS 25 which are planning guidance documents.
- Coastal erosion is an important issue for DCMNR and should be included in a flood risk assessment, in particular the issue of the saltmarsh and whether it is advancing or retreating should be investigated. Ordnance Survey Ireland has aerial photographs available for 1973 and 2000, these could be a good starting point for assessing coastal erosion.
- DCMNR does not have a set SoP for developments. It is advised to take the consequences of exceeding the SoP into consideration and use the highest SoP possible.
- Extreme tide levels for the Dundalk region are not available from the DCMNR tide model yet. Early draft results might be available around mid May.
- The main considerations with regards to climate change were,
 - to make allowance for it
 - to provide flexibility in designs to accommodate for uncertainties and changes in future guidance
 - to be aware of the life cycle of a project.
- It was suggested that we should contact their consultant for the Coastal Strategy Study, RPS, who are currently undertaking extensive tidal numerical modelling of the east coast to see if they might have details of any tidal data in the Dundalk region or have preliminary results which could be of use.

2.2 Data Collection and Review

2.2.1 Topographical data

A number of plans, maps and surveys covering the study area were made available for use within the study. These include the following;

- An OS plan of the site and its surrounding area provided by Tobin.
- Drawings of the development plans provided by Tobin.
- A detailed level survey of the site and embankment dated 2004 provided by Tobin.
- A survey of the embankment consisting mainly of cross sections at 100m intervals dated December 2002 provided by DTC and RPS.

The topographic survey completed by Tobin covered the whole of the proposed development lands including the embankment along the site, but it did not cover the embankment to the north of the site as far as Shore Road. The embankment survey comprises of levels on the top and bottom of the slopes of the embankment and the drainage ditch taken at approximately 20 metre intervals. The survey did not obtain any level data beyond the seaward toe of the embankment into the saltmarsh.

The other survey provided by DTC and RPS (formerly KMM) was completed as part of the Dundalk to Blackrock Coastal Walk study. This survey commenced north of Shore Road near Soldiers Point and covers the whole of the coastal embankment south to the R172. Levels were taken on the crest of the embankment approximately every 25m and every 100m a full cross-section through the embankment was completed. At 200m intervals this cross-section was extended into the saltmarsh a distance of approximately 50 metres and also back in the hinterland. Level data along the R172 to the south of the site was also obtained as part of this survey. Both surveys have been made available in CAD format.

Both surveys of the embankment were compared against each other to check accuracy and possible changes. The main focus here was to confirm that levels of the crest and toes of the embankment were in line with each other. The comparison concluded that both survey were sufficiently similar to conclude that the accuracy of each is acceptable and that the embankment has not changed significantly over the 2 years between each survey. As such either survey could be used for the assessment.

In this respect the survey provided by DTC has primarily been used in the analysis of this study. This is because the survey was in cross-section format, extended beyond the development lands to Shore Road and also extended into the saltmarsh which would be necessary for the assessment of wave climate at the site.

2.2.2 Tide data

General

In order to undertake a reliable analysis of the tidal conditions at the site and in particular determine extreme tidal conditions it was necessary to obtain details of any measured tide data at Dundalk or as close as possible to the area. If long tidal records

were available it would be possible to undertake an extreme analysis of the data to provide predictions of extreme tide levels. In the absence of long records short period records could be used to develop a relationship between Dundalk and another location where an extreme analysis has already been completed.

Numerous authorities and organisations were contacted in an attempt to determine if tidal records existed in the Dundalk region. This included,

- Dundalk Harbour Commissioners (DHC)
- Dundalk Town Council
- Louth County Council (LCC)
- DCMNR
- Hydrographic Surveys Ltd
- Drogheda Port Company (DPC)

DHC confirmed that no tide gauge exists at the port and that they were not in possession of any tidal data. LCC suggested that tidal records have been taken over a period of several years as part of the Port Oriel development which was nearing completion and which is located a short distance south of Dundalk Bay along the east coast. However, contact with the contractor responsible for the site concluded that no tidal data was collected.

During the consultation meeting with the DCMNR it was suggested that a tide gauge existed at Drogheda Port. However, discussion with a representative of DPC confirmed that the tide gauge had been removed and all data had been lost.

The only tide measurements that were recovered for Dundalk Bay were obtained from Hydrographic Surveys Ltd and covered three tide peaks in 1992.

The highest tide in Dundalk occurred in February 2002, but only anecdotal tide level values are available for this event. Three different levels have been quoted, 3.35 and 3.6 mODM in the draft Local Area Plan, although the source of these values is not known. The third is a level of 3.65 mODM quoted by Peter McVeigh of DTC and based on a level seen on Quay Street during the event and measured at a later date.

In view of the lack of data in the Dundalk area, additional sources of data were sought at sites along the coast. Extensive tidal records for Dublin were already available and an extreme analysis of them was completed by RH as part of the Dublin Coastal Flooding Protection Project. However, additional data sets along the coast were sought to provide data against which comparisons could be made and interpolations to Dundalk undertaken. Additional sources were considered at places such as Warrenpoint, Kilkeel, Greenore and Bangor to the north and Port Erin on the Isle of Man to the east.

Whilst it was determined that a tide gauge existed at Warrenpoint, Port Erin and Bangor, only measured tide data was collected for Port Erin between 1992 and 2006 and Bangor (Northern Ireland) between 1994 to 2006. Both sets of data were obtained from the British Oceanographic Data Centre's National Tidal & Sea Level Facility website.

Full details of the analysis undertaken, using the limited amount of data available, is presented in Section 4.

Coastal Strategy Study – Tidal Study

During the consultation meeting with DCMNR it was confirmed that RPS Consulting Engineers are currently undertaking tidal hydrodynamic modelling for the whole of the east coast of Ireland with a view to determining extreme tide levels for use in flood mapping works. DCMNR suggested that RPS might have some information on measured tidal data and/or some preliminary results which could be of use to the study. In view of the lack of data uncovered RH contacted RPS to enquire about the availability of data in the Dundalk region and the Irish Sea in general.

RPS confirmed the lack of data in the region and over the east coast as a whole. They confirmed that whilst their work was not yet finalised, they might be able to provide some data based on the preliminary results of their calibration runs which could help to establish a relationship between Dublin and Dundalk for both astronomical tides and surge.

In view of the lack of data and in order to verify the tidal analysis work which had already been completed prior to consultation with RPS using the results of the tidal analysis at Dublin and the limited data for Dundalk, it was agreed with the client that this data from the RPS model would be obtained.

A discussion of this information and how it has been used to fine tune the tidal analysis for the study is presented in Section 4. The full report provided by RPS can be found in Appendix D.

2.2.3 Reports

Several reports and extracts from reports were made available for the study. This included the following,

- Draft copy of the Local Area Plan.
- Extract from the EIS in relation to hydrology and flood storage of the site.
- The DTC report commissioned for the Dundalk to Blackrock Coastal Walk and completed by KMM, dated January 2003.
- 2 page extract from a addendum report produced for the coastal walk dated August 2005 by KMM. A full version of this report was requested from DTC but the consultant noted that this was unavailable.

2.2.4 Aerial Photographs

In order to carry out an evaluation of the evolution of the saltmarsh fronting the site, a number of aerial photographs were obtained. This included a recent aerial photograph of the site dated 2004 and provided by Tobin. In addition to this, the Ordnance Survey of Ireland (OSI) was contacted and supplied aerial photographs for a further two years which included 1974 and 1995. The evolution analysis is presented in section 5.

3 SITE INSPECTIONS

3.1 General

A visual inspection of the embankment and development lands was undertaken on the 13th and 14th of March 2007 by Royal Haskoning. This included a walk over survey of the embankment commencing at the Shore Road and finishing where it meets the R172, some 2275m in total length. In addition the R172 a short distance south of the development lands and also the northern boundary were inspected. An extensive photographic record of the embankment was taken during the inspection, with details of the front and rear slopes, crest, drainage ditch and other features recorded at regular intervals and at locations of specific interest.

As the inspection commenced at Shore Road this location has been taken as zero chainage with chainage increasing in a southerly direction. Figures A1 to A4, presented in Appendix A, show the extent of the inspection including the chainage and features of particular importance. Figure A1 provides an overview key plan, while Figures A2 to A3 provides more detail over the whole embankment frontage. In addition to these plans Figures C1 to C15, presented in Appendix C, present a selection of photographs and topographic cross-sections which show the shape and condition of the embankment and other features. Both sets of figures will be referred to in the description of the embankment that is presented in the following sections.

The overall embankment, whilst generally trapezoidal in shape, varies extensively over its entire length. Inspection of the profile and condition was made difficult by the overgrown nature of the embankment with extensive long grass, shrubs and trees. In places the growth was such that access and visual inspection was not possible. For ease of description the embankment has been split up into 5 sections based on the shape and condition, although variations do occur even within these sections. These include,

- Section 1 – Shore Road to Start of Firing Range (Ch 0 – 653)
- Section 2 – Start of Firing Range to the Drainage Outfall (Ch 653 – 1275)
- Section 3 – Drainage Outfall to End of Wide Embankment Crest (Ch 1275 – 1700)
- Section 4 – End of Wide Embankment Crest to End of Main Coastal Embankment (Ch 1700 – 2050)
- Section 5 – Return Embankment to R172 (Ch 2050 – 2475)

3.2 Embankment Inspection

3.2.1 Section 1: Shore Road to Start of Firing Range (Ch 0 to 653)

This section commences at the Shore Road and extends to the start of the firing range, which is the edge of the development site, see Figure A2 in Appendix A for location.

General Profile

The start of the embankment is very heavily vegetated with grass, bushes, bramble and trees as shown on Figure 3-1 (chainage 20) making the shape and condition hard to assess. Over the remainder of the section the vegetation is similar with grass mainly on the front slope and thick brambles on the crest and back.



Figure 3-1 Heavy vegetation on the embankment

The embankment generally has a trapezoidal shape over this section. However, the front face of the embankment is generally quite steep and very irregular throughout, with dips and vertical scars at the toe in places. The crest of the embankment also varies in width over the section, from reasonably wide (≈ 2 m) to narrow (<1 m). It should be noted that these widths are all considerably less than would be recommended for a new embankment defence, where widths of 4 to 5 m are considered more appropriate.

Figure C1 and C2 show details of the embankment at two typical locations for this section. The figures show photographs of the embankment and a typical cross-section. The first is at chainage 102 and shows a typical section of embankment with a wider crest (see Photo 2). The highest crest level based on the topographic survey is around 4.01mODM. The second, at chainage 550, shows a section with a narrower crest and the level at this location is around 4.45mODM (see Photo 6). Both locations show the irregularity of the front face, see Photos 1 and 5. The back face of the embankment is also very steep and projects directly into the drainage ditch at the back of the embankment all along this section. Much of the back face over this section is too heavily covered with vegetation to closely assess its condition.

Over this section the embankment crest varies from 3.42mODM at its lowest point to 4.45mODM at its highest point.

Specific Issues

There is an access gap through the embankment at the bottom of the Shore Road. The crest of this gap is about 3.38mODM based on the topographic survey which is some 0.5 of a metre lower than the embankment on both sides and can be seen on Photo 9 in Figure C3. This access gap could be vulnerable during extreme events.

Over the first 75 m of the embankment there is a small vertical scar present on the front face at the toe as shown on Photo 10 in Figure C3. This appears to occur mainly where the embankment changes direction just south of Shore Road.

Several holes, possibly animal burrows, were noted along this stretch of embankment, with a very large one at chainage 478. This hole cuts right through the embankment

and on the rear face there is a very large scar hole. It is almost certain that a breach would occur in the embankment at this location should it be subjected to an extreme tidal event. This hole is shown on Photo 11 in Figure C3, looking from the front of the embankment. It is possible that many other holes exist and were not visible due to the heavy overgrowth on the embankment.

There are several low and potential vulnerable spots along this stretch of embankment, where the embankment crest is lower and/or narrower than general. This occurs at chainages 218, 408, 0505, 0529, 0550, 0572, 0605 and 0626. Examples at chainage 218, 408 and 572 are shown in Photographs 12, 13 and 14 respectively in Figure C3.

It is thought that chainage 218 and 550 could be the location where breaches may have occurred in the past. This is because the vegetation looks relatively new and there was some evidence of borrow pits in the salt marsh where material has been excavated to repair the breaches. The embankment at chainage 550 is shown in Figure C2 and the potential borrow pits shown in Photo 8 on that figure.

A crossing appears to exist at chainage 572 providing access between the saltmarsh and the farm land to the rear. This creates a low spot on the embankment as shown on Photo 14 in Figure C3. The drainage ditch to the rear at this location is filled with material to create the access crossing.

Conclusions

Overall the embankment over this section has reached the end of its useful life. The embankment profile including shape and crest level varies significantly over the length and the crest is very narrow in many places. Furthermore the embankment is heavily vegetated and contains many holes and is thus clearly not maintained. This would suggest that it is susceptible to failure during extreme tidal events. This is verified by the fact that up to 5 breaches occurred during the storm on the 1st February 2002. Whilst it offers some protection to the hinterland, this is currently at an unacceptable level, particularly so for residential properties.

3.2.2 Section 2: Firing range to drainage ditch outfalls (chainage 653 to 1275)

Chainage 653 marks the start of the firing range. This section of the embankment continues until just past the drainage outfall at chainage 1275. For location see Figure A2 and A3 in Appendix A.

General Profile

The shape of the embankment over this section is still very irregular. The vegetation, whilst still present, is less heavy over this stretch. The crest of the embankment is quite narrow over most of the section, at around 1-1.5m. Part of the crest is covered in bushes. The shape of the embankment is generally trapezoidal, although between chainage 1200 and 1300 the crest becomes quite pointy and the embankment becomes more triangular in shape. This can be seen in Figure 3-2.

The back face is generally steep and covered with vegetation including small trees. From chainage 653 onwards there is a distinguishable berm between the back face of

the embankment and the ditch, with a width of 2-3m. Figure C4 shows the embankment at chainage 975. Here the embankment is fairly wide and has a trapezoidal shape. Photographs 15, 16 and 17 show the front face, the crest and the back face and berm between the toe of the back face and the ditch. The heavy vegetation on the back face is shown on Photo 18.

As mentioned above from chainage 1200 onwards the embankment becomes particularly triangular and the crest becomes quite pointy. Figure C5 shows the embankment at chainage 1263. Photo 20 shows the triangular nature of the embankment here and this has also been picked up by the topographic survey as can be seen by the cross-section presented in Figure C5. The vegetation over this stretch is mainly grass, as can be seen on the Photos in Figure C5.

Over this section the embankment crest varies from about 3.7mODM at its lowest to 4.59mODM at its highest level.



Chainage 1058



Chainage 1211

Figure 3-2 Differences in embankment shape

Specific Issues

At some locations along this section there are vertical scars along the back toe of up to 1m high, an example of this can be found at chainage 835 and is shown on Photo 23 in Figure C6. This scar is located at the southern end of the firing range and just at the start of the development site.

At chainage 653 there is a pedestrian path crossing over the embankment and a small pedestrian bridge over the ditch at the back. Due to the path the embankment is low here and well rounded. At the back face there is a small vertical scar where the grass has been eroded due to people walking over it. Photo 24 in Figure C6 shows this location. At chainage 785 and 880 there are a further two paths crossing the embankment which result in slightly lower crest levels Photo 25 in Figure C6 shows the location at chainage 785 from the front face. Another weak spot is located around chainage 932 and Photos 26 and 27 in Figure C6 show the step back in the front face, the lower narrower crest and the poor condition of the back face at this location.

South of chainage 1058 the embankment looks more vulnerable. There are many dips in the front face of the embankment, the crest has many low and narrow spots and the embankment as a whole is very irregular. Two channels in the saltmarsh extend almost

to the toe of the embankment and at these locations there are dips in the front face of the embankment. This situation is shown on Photo 28 in Figure C6. This Photo also shows the existence of a grassy ridge, rather than saltmarsh vegetation, at the toe of the seaward embankment. This ridge exists over much of section 1 and this section comes to an end around chainage 1210, see Photo 31 in Figure C7. Its existence would suggest slightly higher ground levels along the toe or seaward of the toe such that salt water does not penetrate during normal astronomical tidal conditions. This would also offer some protection to the embankment in the form of helping to reduce wave climate at the toe of the embankment.

Another weak spot exists at chainage 1093, where there is a step back in the front face, the crest is low and narrow and a vertical scar exists on the back face. Photo 29 and 30 in Figure C7 shows the front and back slopes respectively at this location. There are other low or weak spots in this section at chainages 719, 975, 1127 and 1138.

The open land drains discharge through the embankment at around chainage 1275, see Photo 32 in Figure C7. At this location there is a large hole in the back of the embankment, as shown on Photo 22 in Figure C5. It is thought that this may have been left like this following the construction of the new outfall through the embankment at this location. The Photos 32 to 34 in Figure C7 show the embankment at the location of the outfalls. The outfalls themselves are discussed in section 3.3.

Conclusions

Whilst the embankment over this section is in slightly better condition than that over section 1, it is still in a relatively poor condition and would not provide adequate level of protection against extreme tidal events in the future. As with section 1 the embankment profile varies significantly over the length. There is significantly less heavy vegetation and fewer holes which suggest that this section has been better maintained. The fact that it did not breach during the February 2002 event would tend to confirm its better condition. Nevertheless whilst it offers better protection than section 1 it is still at an unacceptable level, particularly so for residential development.

3.2.3 Section 3: Drainage ditch outfalls to end of wide embankment (chainage 1275 to 1700)

This section stretches from the outfalls of the drainage ditch to chainage 1700 and covers a stretch of embankment which is in a relatively good condition with a wide crest level, see Figure A3 in Appendix A for location. Beyond this section the condition and profile of the embankment reduces again.

General profile

South of the drainage ditch outfalls the embankment profile changes significantly. The embankment becomes wider, more trapezoidal and more regular. Figure 3-3 shows the changes in the embankment around chainage 1300. The front face of the embankment is generally in a good condition over this stretch but is a bit steep. The ground along the front toe of the embankment is high, which gives the impression that the embankment is low. The crest is wide and between 3 to 5m. The back face is steep for most of the section but in a fairly good condition. There is a berm between the toe of the rear slope

and the drainage ditch of between 2 to 3 metres over all of this section. Most of this section of embankment is vegetated with grass and only a limited number of bushes.



Chainage 1300 looking north



Chainage 1300 looking south

Figure 3-3 Change in embankment shape

Figure C8 shows a typical cross-section for this stretch of embankment at chainage 1605. Photo 35 shows the well grassed front face of the embankment, with the higher ground in front of it. The crest is shown on Photo 36 and can be seen to be wide and in a good condition. Photo 37 shows a steep back face with a vertical scar at the toe. This vertical scar is shown on Photo 38 and occurs at several places on the back face of the embankment along this section. The height varies from 0.3 to 0.7m.

Photographs 39 to 43 in Figure C9 show various photographs of the embankment over this section. As can be seen from these photographs the embankment is fairly regular and in a relatively good condition.

Around chainage 1680 the embankment begins to change condition over the last 20 metres of this section. The embankment becomes more irregular and narrower beyond chainage 1700. This is shown in Photo 44.

The embankment crest level varies over this section from 3.81mODM at its lowest to around 4.26 mODM at its highest. The level range over this section is the smallest and overall the levels are generally more consistent.

Conclusions

The condition of embankment over this section is considerably better than over the rest of the sections inspected as part of this study. This is largely due to the more regular profile, wide crest and lack of heavy vegetation. The embankment provides an impression of being better maintained than the other sections. The crest levels are not any higher or lower than adjacent sections and are of a more consistent level. There is no elevated grassy ridge over this section as was seen for section 1 and 2. Instead the salt marsh comes much closer to the toe. However, the toe levels seem high and there is a wider flatter slope leading up to the main embankment slope and crest as can be seen in Figure C8. This section was not breached during the February 2002 event which also verifies its better overall condition. Nevertheless, the crest levels are not sufficiently high to provide adequate levels of protection for the future and it will be necessary to raise them in line with the options set out in section 6 of this report.

3.2.4 Section 4: End of wide embankment to end of main coastal embankment (chainage 1700 to 2050)

This section of embankment extends from the end of the wide embankment at chainage 1700 to the end of the main coastal embankment at chainage 2050, see Figure A4 in Appendix A.

General Profile

Whilst the general shape of the embankment in this section is still trapezoidal the profile is more irregular and the crest much narrower and pointy than in the previous section. Figure C10 shows the embankment around chainage 1807. The front face is fairly irregular along the section as shown on Photo 45. Over part of this section there is a wide grassy berm of up to around 9m. Photo 49 shows this berm in the distance around chainage 1807, where it is relatively wide. Where this berm occurs the wave debris is noted to be along the toe of this berm and not at the embankment itself. By the end of this section the berm has blended in with the front face and is no longer distinguishable.

Photo 46 shows the crest of the embankment which is very narrow along this section, between 1 and 2m wide. The crest is also very uneven and in places heavily vegetated. The back slope over the section is steep and generally heavily vegetated. The berm between the back slope and the ditch becomes narrower when moving to the south and disappears towards the end of the section.

Over the last 50 metres of this section the condition of the embankment becomes much better. From chainage 2050 a smaller embankment extends perpendicular from the back of the main embankment towards the west. Figure 3-4 below shows the connection between the two embankments. The smaller embankment is referred to as section 5 and is described next.



Figure 3-4 Connection of the two embankments

Typical details of the embankment between chainage 2000 to 2050 are shown in Figure C12. The cross-section through the embankment shown in Figure C12 was taken at chainage 1991 and was the closest available to this section. The cross-section is very similar and gives a good impression of the embankment over the remainder of the section.

Over this last section of the embankment the condition is good and the crest is wider. The embankment has a good trapezoidal shape and is vegetated with mainly grass. There is rock armouring placed along the front toe of the embankment to protect what is the more vulnerable end of the main embankment and this is shown on Photo 56. The back slope is not too steep and in a good condition, although it is quite well vegetated, as shown on Photo 57. The back slope extends down into the rear drainage ditch without a berm at the toe.

The crest level of the embankment over this section varies from 3.47 at its lowest to 4.06mODM at its highest. The highest level of the embankment over this section is lower than in most of the rest of the embankment.

Specific Issues

Photo 49 in Figure C11 shows the crest of the embankment at chainage 1717. The crest here is very uneven with large tufts of grass and is very narrow. This location is potentially vulnerable from wave action. At chainage 1749 the embankment becomes more heavily vegetated along the rear slope. Photo 50 shows the start of this vegetation. The vegetation includes small trees, which can form a threat for the stability of the embankment.

The grassy berm, mentioned earlier, in front of the embankment shows a rocky toe over much of its length. This can be seen at chainage 1875 on Photo 51. The Photos 52 and 53 of Figure C11 show how the berm in front of the embankment disappears towards the end of the embankment.

Conclusions

The embankment over this section is relatively poor in that it is largely irregular in shape, has a narrow and low crest and is very heavily vegetated in places. The embankment does have a wide grassy berm over much of its front toe with some rock armour along the toe of this berm. This clearly helps to protect the embankment by reducing wave action in front of the main embankment. During the storm of February 2002 no breaches occurred in the embankment. Nevertheless the overall condition could be described as poor and the crest is certainly much too low to provide adequate flood defence needs in the future.

3.2.5 Section 5: Embankment Perpendicular to Main Embankment extending back to the Blackrock Road (chainage 2050 to 2475)

This section of embankment is connected to the end of the main embankment and extends perpendicular out from it at chainage 2050 and extends to the coastal road (R172) where it ends at chainage 2475 after turning through a 60 degrees angle at

chainage 2302. This section is shown on an aerial photograph in Figure 3-5 below and also in Figure A4 in Appendix A.

General Profile

This section of embankment is smaller and lower than the main coastal embankment extending from north to south. The ground levels in front of the embankment are high making the embankment look lower. There is heavy vegetation over most of the section making it extremely hard to view the core and assess the condition of the embankment. Figure C13 shows a typical cross-section through the embankment at chainage 2201. The shape of the embankment is trapezoidal. The front face is not too steep, but heavily covered in bushes as shown on Photo 59. The crest is around 1.5 to 2m wide and also covered in heavy vegetation, as can be seen on Photo 60. The back face is shown on Photo 61. It is steep and runs straight into a wide ditch at this location. In some places there is a small berm present between the toe of the back face and the ditch. Photo 62 shows the hinterland that is not in use at the moment.

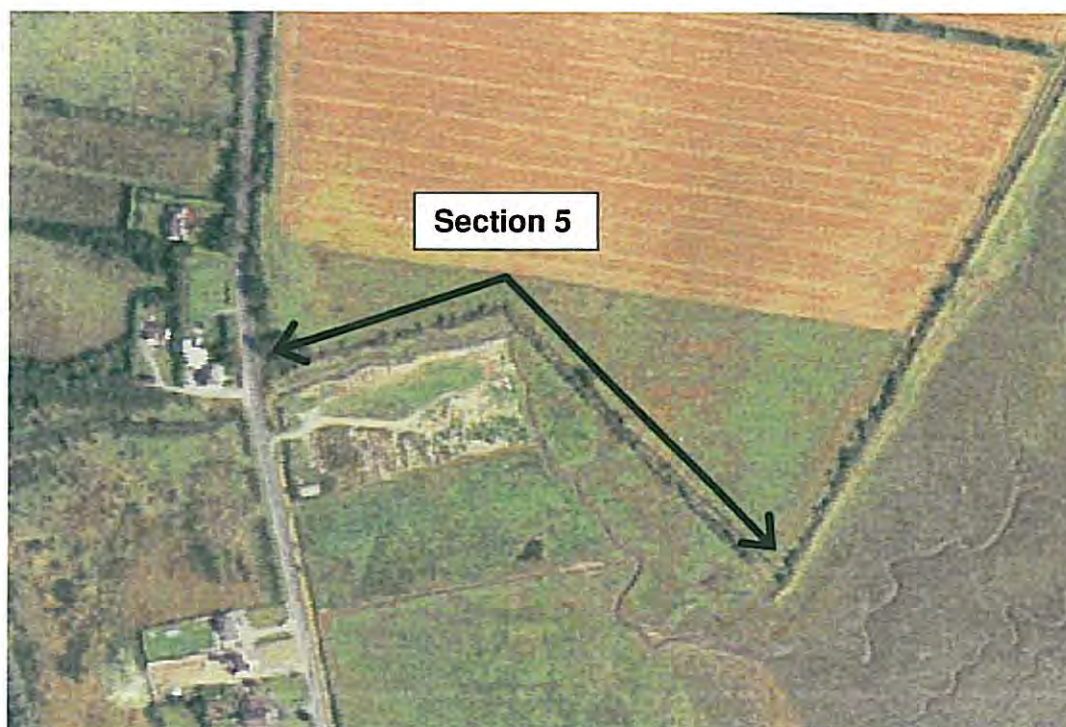


Figure 3-5 Location of section 5

The crest level of the embankment over this section is much lower than for the main coastal embankment. This is because the location of the embankment ensures that it is not subject to wave action. The levels vary from 3.0 mODM at the lowest to 3.65 at the highest. The higher levels are located toward the outer seaward end.

Specific Issues

Photo 63 in Figure C14 shows the embankment between the bend (shown in Figure 3.5 above) and the road. It shows the small tidal channel which runs in front of the embankment towards the road and which passes under the road through a concrete

culvert. The corner of the embankment itself can be seen on Photo 64, with the tidal channel that runs to the toe. There are a few weak spots in this stretch of embankment. At chainage 2408 there is a gap in the embankment and around chainage 2475 there is a low and narrow crest on the embankment. These locations are shown on Photos 65 and 66 respectively. Near the road the embankment converts to a masonry wall and this is shown on Photo 67. Photo 68 shows the end of the embankment.

Conclusions

Whilst this embankment is not subjected to wave conditions due to its sheltered location, its overall condition is poor. The embankment is much lower than the main embankment in places and is also very heavily vegetated. Whilst it has been confirmed that the proposed development lands did not flood during the February 2002 event, many parts of this embankment section fall below the water levels quoted for this event. Furthermore the road at the southern end and land to the rear was confirmed to have flooded extensively and as such it is likely that parts of the south west corner of the site may have been subjected to some local flooding. In any case the current embankment is of an inadequate condition and much too low to provide reliable flood defence in the future.

3.3 Other Site Features

3.3.1 Drainage Ditches

An open drainage ditch extends along the back of the embankment for the whole stretch of embankment that was inspected. Whilst this ditch was inspected in more detail during the site visits the remaining open drainage ditches which cross the overall site were not inspected in detail.

Main Drainage Ditch

The drainage ditch varies in size, shape and condition over much of its length. In places it is covered by very heavy vegetation and as such is difficult to see. From chainage 0 to 653 (Section 1 described earlier) the ditch is not very distinguishable from the main embankment in that it appears very wide and is not separated from the rear toe by any berm. This is visible in the section presented in Figure C2, Appendix C. Over this section the ditch is also very heavily overgrown.

Over the remainder of the ditch, chainage 653 to 2050, the ditch is more distinguishable from the embankment in that it is separated from the rear toe by a berm. The ditch itself over the length varies in width and size but is generally between 2 -3 m wide at the invert and up to 4 m in places.

At chainage 572 a crossing over the embankment exists and the ditch has been blocked to facilitate this. There does not appear to have been any pipes placed in the drain to maintain flow at this location.

At chainage 835 the ditch is again blocked by the back slope of the large earth mound on the firing range. At this location two pipes have been placed under the blockage in an attempt to maintain flow and this appears to be functioning well under normal

conditions. The situation chainage 835 at the back of the embankment is shown in Figure 3.6 below.



Blockage of drainage ditch



Pipes running under blockage

Figure 3-6 blockage of the drainage ditch at chainage 0885

At chainage 1701 there is a significant change in the shape and capacity of the ditch. As shown on Figure 3-7 the ditch is wide and well maintained to the north. However, to the south it becomes narrower and the base of the ditch appear to rise reducing its capacity to store water. It may be that the ditch to the south of the site is placed at a higher level to facilitate flow towards the outfall located at the centre of the site.



Ditch looking north



Ditch looking south

Figure 3-7 Change in the drainage ditch at chainage 1701

At chainage 1275 there are two outfall pipes through the embankment which discharge into a channel in the saltmarsh. The outfall is discussed in section 3.3.2.

3.3.2 Drainage Ditch Outfall

At chainage 1275 the drainage ditch discharges onto a channel in the saltmarsh. The outfalls and the channel are shown in Figure 3-8. The tide was out when these photographs were taken.



Overview outfalls



Channel from outfalls into saltmarsh

Figure 3-8 Drainage ditch outfalls at chainage 1275

There are two outfalls present, an old and new one; both are shown in close up in Figure 3-9. The new outfall consists of two 900mm pipes flapped with Tideflex non return valves. Water was flowing through these pipes on both occasions the site was visited.

The old outfall appears to be a rectangular culvert with a wooden flap. The flap is in a very poor condition, the wood and hinges are deteriorating. It is likely that this outfall is no longer in use and may have been sealed, although during the visit of the 14th March 2007 a small amount of flow was noted.



New outfall pipes



Old outfall

Figure 3-9 - Flapped outfalls drainage ditch



Land bridge with culvert



Outlet drainage ditch

Figure 3-10 - Back side of embankment and drainage ditch at chainage 1275

Figure 3-10 shows the back side of the embankment at chainage 1275, where the outfalls are located. The figure shows there is a land bridge present to the hinterland with a culvert running under it. The outlet from the drainage ditch to the outfalls is constructed with gabion baskets. There is a trash screen present to prevent debris from entering the outfalls.

Whilst the outfall appears to be functioning adequately it is considered that as part of the overall development of the new flood defences the outfall should be completely replaced. This would consist of a new concrete culvert and outfall chamber with a new set of non return flaps.

3.3.3 Northern boundary

The northern boundary of the site was inspected and is located along the access path which leads to the firing range. The proposed development site, including residential development is immediately adjacent to this access path. On the north side of the path there is an open drainage ditch which extends along the full length of the boundary. The land to the north appears to be at a similar level to the development site at this location. There is much vegetation along the ditch which would offer some resistance to water flowing from the north. Photos showing the northern boundary are presented in Figure 3.11 below.



Access Road
Figure 3-11 – Northern Boundary



Drainage Ditch

3.3.4 R172 south of the site

At the southern end of the site the coastal embankment meets the R172 road and terminates as described in section 3.2.5. The road at this location is around 3.1 mODM. Continuing south from the site the road remains around 3.1 to 3.2mODM until the first corner where it turns through 90 degrees to the east. At this location it rises to a level of around 3.9mODM at that corner. The road continues in an easterly direction to a second corner before turning south again. Over this stretch the level falls again to around 2.9 mODM at the corner before rising again on the approaches to Blackrock Village.

This stretch of road is low lying and at risk of flooding. Indeed it was reported to have flooded extensively during the 1st February 2002 event. The road extending north along the site boundary is also low lying and generally between 2.9 and 3.1 mODM. As such a possible flood path exists back to the site via this section of road and this will need to be addressed as part of any flood alleviation options.

4 ASSESSMENT OF COASTAL CONDITIONS

4.1 Assessment of Extreme Tide Levels

4.1.1 Methodology

As discussed in section 2.2.2 an extensive data search was undertaken for recorded tide data near the site and further along the east coast. This proved to return very little data for the site and the east coast as a whole.

The methodology proposed for the determination of extreme tide levels at Dundalk was to be centred around the transfer of extreme levels from another site to Dundalk by considering a suitable tidal relationship between both sites. Royal Haskoning recently carried out an extensive analysis of several decades of tide records at Dublin Port as part of the Dublin Coastal Flooding Protection Project (DCFPP). The analysis included a detailed probabilistic analysis of the data which resulted amongst other things in predictions of extreme tide levels in Dublin Bay. Therefore for this study it was proposed to transfer the extreme tide levels at Dublin to Dundalk by considering the relationship of the normal astronomical tides between the two sites and also by comparing a number of extreme events.

Two methods were considered for the transfer of the tide levels. The first included a method proposed in a report produced by Jerzy Graff which investigated extreme tide levels around Great Britain and which related the extreme tide level at one location to another based on Mean High Water Spring (MHWS) tide and spring range at both sites. The second considers breaking down the overall extreme tide level into its two components, astronomical tide and surge, and the use of relationships for both between the two sites. These relationships could be derived based on current relationships provided in astronomical tide tables, relationships developed through comparison of tide data at both sites or relationships developed through numerical modelling works.

Each of the methodologies and the results are described below.

4.1.2 Method 1: Jerzy Graff² Relationship

The method states that the extreme water levels can be transposed from one location to another on the same coastline by equalling the following ratio for both locations. :

$$\frac{\text{Extreme level} - \text{mean high water spring level}}{\text{Spring range}}$$

For Dublin the extreme levels can be determined from the DCFPP work, whilst the spring tide conditions for both Dublin and Dundalk can be obtained from the Admiralty Tide Tables. The astronomical tide level obtained from the admiralty tide tables for both sites are presented in Table 4.1 below.

² Graff, Jerzy. *An Investigation of the Frequency Distribution of Annual Sea Level Maxima at Ports around Great Britain*. 1981.

Table 4-1 – Astronomical Tide Levels at Dublin and Dundalk

	Dublin		Dundalk	
	mCD ²	mODM	mCD	mODM ¹
Highest Astronomical Tide	4.5	1.99	-	-
Mean High Water Spring Tide	4.1	1.59	5.1	2.09
Mean High Water Neap Tide	3.4	0.89	4.2	1.19
Mean Low Water Neap Tide	1.5	-1.01	1.6	-1.41
Mean Low Water Spring Tide	0.7	-1.81	0.6	-2.41
Lowest Astronomical Tide	-0.1	-2.61	-	-

1. These levels assumed a relationship between ODPoolbeg and ODMalin to be -2.71m as set out in the admiralty tide tables.
2. metres Chart Datum (mCD).

Applying the ratio above provided the following predictions of extreme tide levels for Dundalk shown in Table 4-2 below.

Table 4-2 Extreme water levels at Dundalk Using Jerzy Graff Relationship

	Dublin	Dundalk
Return period	Water Level mODM	Water Level mODM
February 2002 event	2.95	3.89
50	2.91	3.84
100	3.03	4.00
200	3.13	4.13
500	3.24	4.27

As can be seen in Table 4-2 this relationship gives levels that are a lot higher for Dundalk than they are for Dublin. This is due to the relatively large difference between the spring range in Dublin and Dundalk. For the February 2002 event anecdotal evidence has suggested levels of 3.35 to 3.65 mODM. This is considerably lower than the level of 3.89 mODM returned from this method. It was therefore concluded that this method does not give reliable results mainly due to the distance between the two sites over an open coastline and the larger difference between the astronomical tides at each site. The method is perhaps more applicable to sites located closer to each other or within an estuarine system.

It should be noted also at this stage that a phenomenon known as seiche has been observed to occur at Dublin from the tidal records and modelling work undertaken as part of the DCFPP. A seiche is a periodic fluctuation of water level within the overall tide curve which for Dublin can have a period of anything from a few minutes up to one hour. The mechanism causing the seiche at Dublin is not fully understood although it is believed to be the result of particular meteorological conditions and the shape of Dublin Bay. Such seiche would be built into the extreme levels presented for Dublin. However, at the time of commencing the project and this analysis it was not known whether seicheing was also observed at other locations along the east coast and in particular within Dundalk Bay. Therefore the extreme levels presented in the table above may be increasing conservative on the basis that they make no allowance for this seicheing condition. This is discussed in more detail later.

4.1.3 Method 2: Astronomical Tide and Surge Transfer Relationship

The second method considers breaking down the overall extreme tide level into its two components, astronomical tide and surge, and the transfer of these by using relationships for both between the two sites. For this method relationships need to be developed between the two sites. This was initially undertaken and researched using data from the admiralty tide tables and the limited data available following the extensive data collection exercise.

Initial Analysis of Tidal and Surge Relationships

Astronomical Tide

An astronomical tide is the periodic rise and fall of the water along the coast as a result of the gravitational attraction of the water by the moon and the sun. These tides are well understood and can be readily predicted years in advance.

The astronomical tide levels for both sites, as presented in the Admiralty Tide Tables, are shown in Table 4.1 above.

Using this data a linear relationship can be determined for the astronomical tides between Dublin and Dundalk. In Appendix D this relationship and how it has been established is presented. The relationship for the astronomical tides in mODM is as follows:

$$\text{Astrotide Dundalk} = 1.3341 \times \text{Astrotide Dublin} - 0.0116$$

This relationship was checked using measured data which was available for the site and which included measurements taken over the peak of three tides back in 1992. Measurements for the Dublin Port tide gauge were obtained for the same period and both plotted in a similar way and a linear relationship was observed within the data. This resulted in a similar but not identical relationship to the one presented above. The results of this analysis are shown in Appendix D.

Surge

Surge is defined as the difference between actual water level and predicted astronomical water level. Differences can occur from predicted astronomical tide due to methodological conditions such as air pressure and wind speed and direction. Low atmospheric pressure and winds blowing onshore tend to increase the water level resulting in a positive surge. Whilst high atmospheric pressure and winds blowing offshore tend to lower the water level and result in negative surges. Positive surges are those which will result in extreme tidal conditions.

Ideally the relationship between surges at both sites would have been investigated through the comparison of measured data for a number of storms at both sites. However, the data collection returned little tide data and none which covered any known storms such as that which occurred on the 1st February 2002.

Given the relatively close proximity (approx 65km) of the two sites on the central part of the east coast, it was felt that the level of surge experience at Dublin would be of a

similar magnitude to that at Dundalk. Furthermore given that the peak of a surge could last for up to one hour (noted for the 1st February 2002) and that the time difference of high water between the two sites was less than 0.25 hours, it was felt that if the peak of a surge coincided with high water at Dublin then it would also coincide with high water at Dundalk, giving rise to an extreme tide.

In order to investigate this perception a comparison of tide and surge data for a number of recent storm tides was undertaken for the data available at Dublin, Port Erin and Bangor. This analysis generally showed similar levels of surge between these sites i.e. <0.15m. However, on some occasions the differences were greater. As such, and given the distance of the measurements from the site, this comparison proved not to be fully conclusive.

As part of the DCFPP a tidal flood forecasting system was developed for Dublin by using forecasts obtained from the UK Met Office's storm surge forecast model. During that study details for a number of storms were obtained including animations of the surge build up within the Irish Sea. These were also evaluated to compare the predicted surge build up between both locations.

The storms in question included the event of the 1st February 2002 and the event of the 27th October 2004. Figure 4-1 below shows the surge output plots from the 1st February 2002 event. The colour bands on the plot represent the surge levels at 0.1m intervals. Whilst the plots are not of very high resolution, it can be seen that for the Dublin and Dundalk area, over the peak of the event, both areas either in the same colour band or are at most one colour band different ($\leq 0.1\text{m}$ difference)

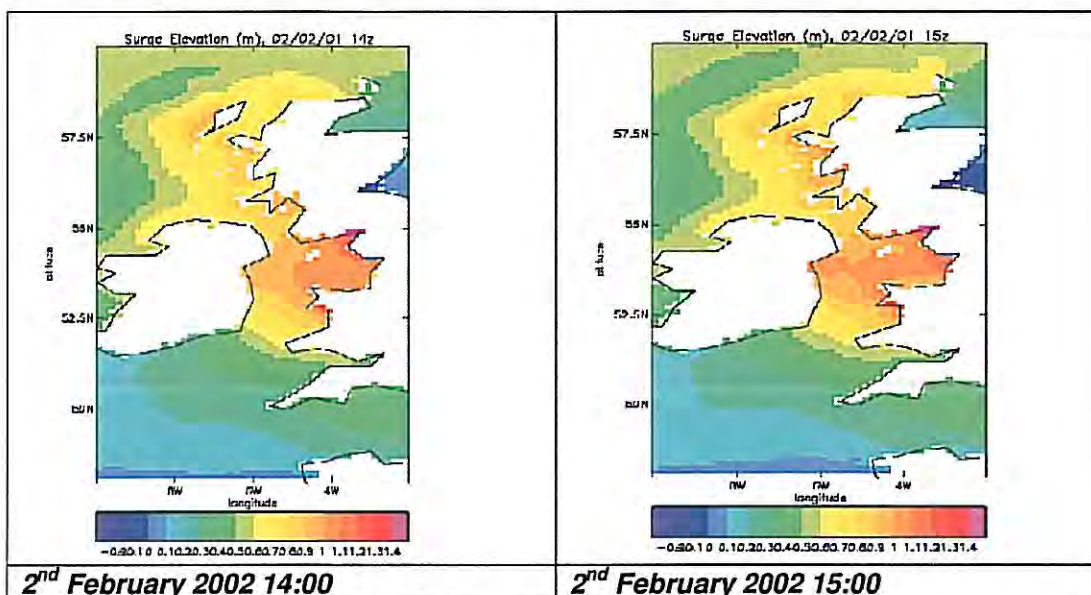


Figure 4-1 Surge Plots UK Met Office Surge Model – February 2002 Event

Figure 4-2 below shows the surge output plots from the 27th October 2004 event. The colour bands on the plot similarly represent the surge levels at 0.1m intervals. High tide on this event occurred around 23:30 on the evening of the 27th which is mid way between the two plots shown. It can be seen that for the Dublin and Dundalk area for the half hour before high tide there is a difference between the two areas of between 1

and 2 bands. However, half hour after high water the areas appear to fall more within the same band. As such it is concluded that the surge on this occasion was similar between both sites and not more than 0.1 to 0.15m in difference.

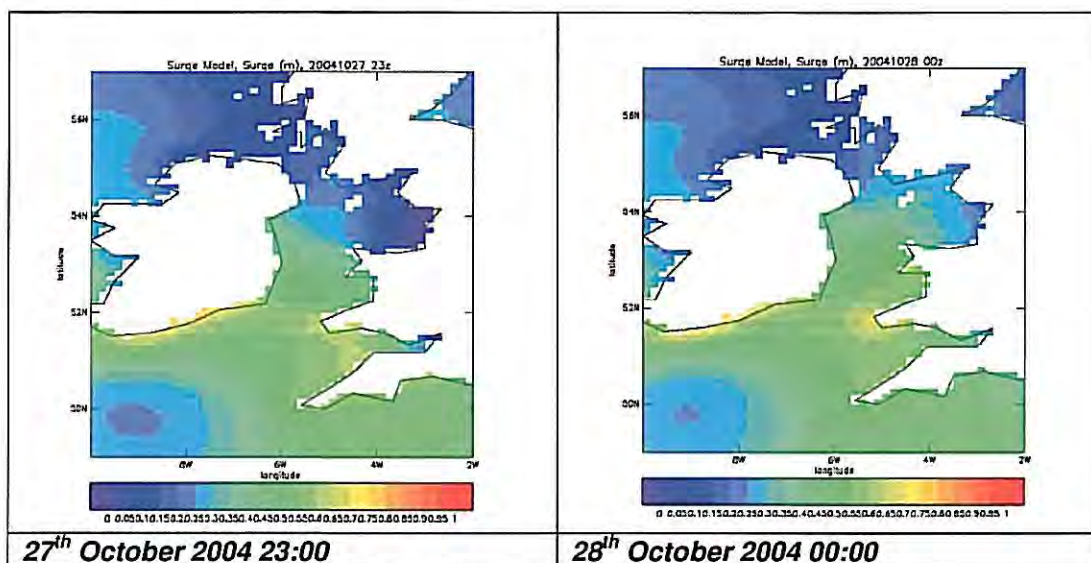


Figure 4-2 - Surge Plots UK Met Office Surge Model – October 2004 Event

On the basis of this evaluation it was concluded that an assumption to apply the same tidal surge at Dundalk as at Dublin would be justified. As such the analysis would transfer the astronomical tide component of the extreme tide to Dundalk using the relationship above and then the surge component would be transferred on a one to one basis, i.e. Surge Dublin = Surge Dundalk.

Initial Results

The above relationships were applied to a range of extreme tide conditions for Dublin to determine equivalent values for Dundalk. Obviously any extreme tide can result from a range of different combinations of astronomical tide and surge, i.e. a high astronomical tide can combine with a small surge or a lower astronomical tide can combine with a higher surge to result in the same overall water level.

As such it was deemed sensible to consider a range of combinations for any given extreme event. Thus extreme tide levels at Dublin were broken down to astronomical tides of 1.6, 1.8 and 2.0 mODM and their respective surge components based on the required extreme level. These were then transferred to the Dundalk site using the developed relationships. The February 2002 event was also transferred based on the actual astronomical tide predicted for the day. The results of this analysis can be found in Table 4-3 below.

Table 4-3 Extreme Tide Levels at Dundalk using Initial Astronomical Tide and Surge Relationships

	Dublin	Dundalk			
<i>Astronomical tide component Dublin (mODM)</i>		1.99	1.6	1.8	2
Return period	Water Level mODM	Water Level mODM	Water Level mODM	Water Level mODM	Water Level mODM
February 2002 event	2.95	3.6			
50	2.91		3.43	3.50	3.57
100	3.03		3.55	3.62	3.69
200	3.13		3.65	3.72	3.79
500	3.24		3.76	3.83	3.90

Based on the above results it can be seen that the extreme levels predicted for Dundalk are considerably lower than those resulting from method 1. Further more the analysis results in a predicted level at Dundalk for the February 2002 event of 3.6mODM which falls within the values quoted for that event.

It can also be seen that the higher astronomical tide combination give higher overall levels for Dundalk. This is because the astronomical tide relationship equation has a gradient of 1.3 in the linear relationship for the astronomical tides as apposed to 1 for the surge.

Furthermore it should be noted that the transfer of the levels direct from Dublin, as undertaken above, makes no allowance for the seiching effect noted at Dublin and thus may result in conservative levels at Dundalk.

4.1.4 Coastal Strategy Study for Ireland – Tidal Modelling

Following the completion of this initial analysis RPS Consulting Engineers were contacted on the advice of DCMNR in view of the fact that they are currently completing a tidal modelling study for DCMNR as part of their Coastal Strategy Study for Ireland. The study was focused on the east coast and aimed to determine extreme tide levels for use in assessing flood risk.

Following discussion with RPS they confirmed that whilst their work was not yet finalised, they would be able to provide some data based on the preliminary results of their calibration runs which could help to refine our relationships between Dublin and Dundalk for both the astronomical tides and surge.

RPS subsequently produced a short report setting out the results they had particularly focusing on the transfer of information between Dublin and Dundalk. The full report is included in Appendix D and the main issues are summarised below.

The results of the tidal modelling work completed by RPS produced a number of updated results and information which were used to fine tune the tidal analysis work. This included;

- New relationship between ODPoolbeg and ODMalin for Dundalk.
- Details of the astronomical tides at Dundalk including Highest Astronomical Tide (HAT).
- Linear surge relationship between Dublin and Dundalk based on calibration runs for storm event over the past 25 years.
- Insight to seicheing out side of Dublin Bay and particularly at Dundalk.

Poolbeg to Malin Relationship at Dundalk

The RPS report generally presented the results relative to mean sea level (MSL) and then converted them back to OD Malin by converting first to Chart Datum, then to OD Poolbeg and finally to OD Malin. Moving between such datums around the coastline is common practice and different relationships between chart datum and Poolbeg exist for different locations and are quoted in the Admiralty Tide Tables. The Admiralty Tide Tables also quotes a value of -2.71 between Poolbeg and Malin. The RPS report however quoted a value of -2.64m between ordnance datum Poolbeg and ordnance datum Malin for Dundalk, suggesting that this was obtained through correspondence with Ordnance Survey Ireland.

OSI control section were contacted to discuss this issue and confirmed that this value is more representative of the Dundalk region. Whilst the difference is not significant it will have a small impact on the final water level. The extreme analysis has thus been revised taking this -2.64 conversion into account.

Astronomical Tides

The report produced by RPS presented slightly different levels for the astronomical tides compared to those values given in table 4.1 above. This is partly due to the fact that the results have been obtained through their tidal modelling work and also because the conversion of -2.64 mentioned above has been used. The results are given in Table 4-5 below.

Table 4-4 Astronomical Tide Levels Dundalk from RPS Report

Tide	Astronomical tides Dundalk (mODM)
Mean High Water Springs (MHWS)	2.10
Mean High Water Neaps (MHWN)	1.20
Mean Low Water Neaps (MLWN)	-1.30
Mean Low Water Springs (MLWS)	-2.29

The difference in the levels between the high waters is less than that for the low waters.

Using these levels a new relationship between Dublin and Dundalk was derived for the astronomical tide. Appendix D shows how this relationship has been derived. The relationship is as follows:

$$\text{Astrotide Dundalk} = 1.2971 \times \text{Astrotide Dublin} + 0.038$$

This relationship is similar to the one determined earlier using the Admiralty Tide Table levels, although the gradient of the slope in the equation has reduced slightly from 1.33 to around 1.3.

Surge

Based on the results of the tidal surge modelling work for Ireland completed by RPS, an analysis of 25 storm surges was undertaken with surges greater than 0.5m. The surge residuals were extracted from the model for Dublin and Dundalk and plotted against each other, see Figure 4-3.

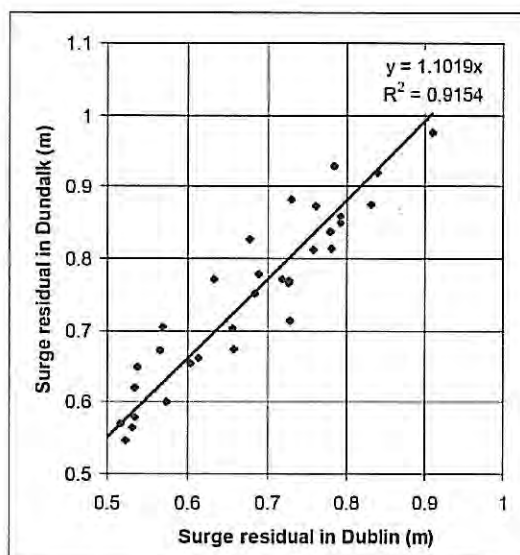


Figure 4-3 – Correlation of Surge Residual for Dublin and Dundalk³

A linear regression was applied and resulted in the following surge relationship between Dublin and Dundalk.

$$\text{Surge Dundalk} = 1.1 \times \text{Surge Dublin}$$

This relationship will produce surge estimates at Dundalk which are 10% larger than the assumption previously made for the initial analysis. Whilst this might sound significant in terms of increased surge height this will equate to a maximum increase of around 0.05 to 0.15m. Furthermore it can be seen from the above figure that there is scatter above and below the residual line which will suggest that on occasions the surges at Dundalk may be larger or smaller than represented by this relationship. It should be noted that the scatter is however greater above the line for Dundalk compared to Dublin.

Seiche

The RPS report confirmed that during their tidal analysis study they carried out tests to reproduce the effects of seiching which are noted in Dublin Bay. This was because their overall surge model did not represent any seiching effects in Dublin Bay due to the fact that meteorological effects represented in the model were defined at 3 hourly intervals. To test for seiching this interval was reduced to 0.5 hours and a finer spatial resolution used.

³ Extract from report entitled Water Level Assessment for Dundalk, prepared by RPS for RH May 2007. See Appendix D for full report.

Based on these tests they concluded that an additional 100 to 150mm would need to be added to the water levels produced by their overall model in the inner part of Dublin Bay to account for the effects of seiching in the bay.

Discussion with staff at RPS also concluded that seiching effects were not noted to occur within Dundalk Bay. As such it could be deduced that a similar allowance of 100 – 150mm reduction in the extreme tide levels estimates from the DCFPP study for Dublin Port should be made before they are transferred to regions which are not subject to seiching.

The RPS report concluded by presenting a 200 year tide for Dundalk of 3.64mODM. This was based on the transfer of a 0.02 year tide with a 1.17m surge at Dublin to Dundalk based on the above criteria.

4.1.5 Final Extreme Water Levels at Dundalk

Using the additional data provided by the RPS report, the analysis of the extreme tides using astronomical tide and surge transfer relationships (method 2), as set out in section 4.1.3, was revised.

A number of cases to incorporate some allowance for the effects of seiching in Dublin Bay were considered as follows,

- No adjustment of extreme water levels at Dublin
- A reduction in the extreme water level before transfer of 100mm
- A reduction in the extreme water level before transfer of 150mm.

Using the astronomical tide and surge relationships as described in section 4.1.4 above extreme tide levels for Dundalk are calculated and the values for the 200 year return period are shown in Table 4-5 below.

Table 4-5 - 200 Year Extreme Water Levels at Dundalk

Astronomical tide component Dublin (mODM)	Dublin	Dundalk			
		Feb 02 1.99	1.6	1.8	2
Adjustment for seiche	Water Level mODM	Water Level mODM	Water Level mODM	Water Level mODM	Water Level mODM
No reduction	2.95	3.68			
75mm reduction ¹	2.875 ²	3.59			
No reduction	3.13		3.79	3.83	3.87
100mm reduction	3.03 ²		3.68	3.72	3.76
150mm reduction	2.98 ²		3.63	3.67	3.71

1. Seiche recorded at Dublin on 1st February 2002 was noted to be small and hence slightly lower reduction has been taken.
2. Water levels have been adjusted to account for seiche before transfer.

From the results above it can be seen that when a slight allowance for seiching is made the levels predicted for the February 2002 event are in line with the anecdotal levels quoted of 3.6 and 3.65 mODM.

Based on the results above it is suggested that a present day 200 year level at Dundalk should be taken as 3.7mODM.

4.2 Sea Level Rise

For any new development it is extremely important to consider the impacts of climate change over the design life of the development or structure. For coastal developments such as Rockview Farm the main issues will be one sea level rise and increased storminess. Estimates of sea level rise are available through research and in many cases can vary greatly depending on the model used for predictions and the global emissions scenario chosen which reflect the choices people make in the future about how they live.

For the DCFPP a detailed review of climate change, and in particular sea level rise, was undertaken based on the latest guidance at that time. The review included international and national best practice and other reports from previous studies. In particular the latest guidance from the Intergovernmental Panel on Climate Change (IPCC), UK Climate Impact Programme (UKCIP) and the University College Ireland, Manooth (UCI) was reviewed.

The main conclusion of relevance to this study as follows;

- An annual average allowance for sea level rise (SLR) of 4.15mm/yr applicable to 2100 should be applied.

Through discussions with the client it has been concluded that the development should have a design life which takes it through to the end of this century and as such sea level rise up to the year 2100 should be considered.

Given that the base date for the DCFPP study was around 2005 this would result in 95 years of SLR which at 4.15mm/yr would give a value of 0.394m. For the purpose of this study this has been rounded up to 0.4m.

This is in line with other guidance given in Ireland such as the climate change policy developed under the Greater Dublin Strategic Drainage Study (GSDSDS) which recommends an allowance of 440mm for design up 2080 is considered.

In October 2006 DEFRA in the UK completed a review of sea level rise data. Although this did not cover Ireland, the increased values published for sea level rise are significantly higher than noted here. Whilst the OPW and DCMNR have not adopted these precautionary sea level rise allowances it could be prudent to design any coastal defence structure in a flexible way such that some raising of the defence in the future could be undertaken should the current recommendations be underestimated.

For the purpose of this study it is recommended that an allowance for sea level rise to 2100 of 0.4m is made. This results in the following 200 year water levels predictions for the development of the site.

- Present Day : 3.7mODM
- 2100 : 4.1 mODM

4.3 Assessment of Extreme Wave Conditions

General

The proposed development site is located along the western side of Dundalk Bay and is fronted by an extensive width of saltmarsh and then mudflat. This whole area dries out at low tide to distances of over several kilometres. During normal astronomical tides, the water level only reaches the toe of the embankment where drainage channels cut through the saltmarsh. The levels of the saltmarsh and ground along the toe of the embankment varies from around 2.2 to 2.9mODM and as such tide levels above this values would be required before the tide can extend onto the toe of the embankment itself. In some places the saltmarsh extends 40-50m out from the embankment and is slightly higher than the levels at the embankment toe.

Therefore wave conditions will only reach the site and embankment structure when tide levels exceed values of around 2.5 to 3mODM. Because of the extensive width and level of the mudflat and saltmarsh, the waves that reach the toe of the embankment during elevated extreme tides will always be depth limited. That is they will be limited by the depth of water available and regardless of how high the waves are offshore in deep water they will break down to be more or less the same height by the time they reach the site.

For such situations a reliable evaluation of the likely maximum wave heights that could exist for depth limited conditions can be made using techniques set out in the Coastal Engineering Manual, the Rock Manual and other best practice guidance documents.

Numerical wave modelling could also be used; however it was felt at this stage that given the extensive width of saltmarsh these depth limited techniques would provide reliable and slightly conservative wave heights.

Based on depth limited conditions it was concluded that the following wave heights could be expected at the site during an extreme 200 year storm.

Table 4-5 – Wave Heights at the Site

Extreme Water Level (mODM)	Wave Heights (m)
3.7	0.75 – 1
4.1	0.8 – 1.2

The wave height presented above relates to a distance of approximately 15m seaward of the toe of the embankment. The wave height range given reflects the different ground levels along length of the embankment.

5 ASSESSMENT OF FLOOD HAZARD

5.1 General

In this section the results of the work presented in sections 2, 3 and 4 of the report are used to evaluate the flood risk at the site for the given situation. The levels obtained from the topographic survey are compared with the extreme tide levels predicted and for the February 2002 event. Based on this using the results of the wave climate and overtopping analysis work an assessment of the standard of protection offered by the existing embankment is provided.

Finally in section 5.4 an assessment of the evolution of the saltmarsh is presented with a view to its importance in the role of coastal protection.

5.2 Comparison of Embankment Levels and Extreme Tide Levels

The highest crest levels taken from the coastal walk topographic survey completed in December 2002 were taken and a longitudinal section produced from Shore Road to the Blackrock Road (R172). This longitudinal section is shown in Figure 5.1 below.

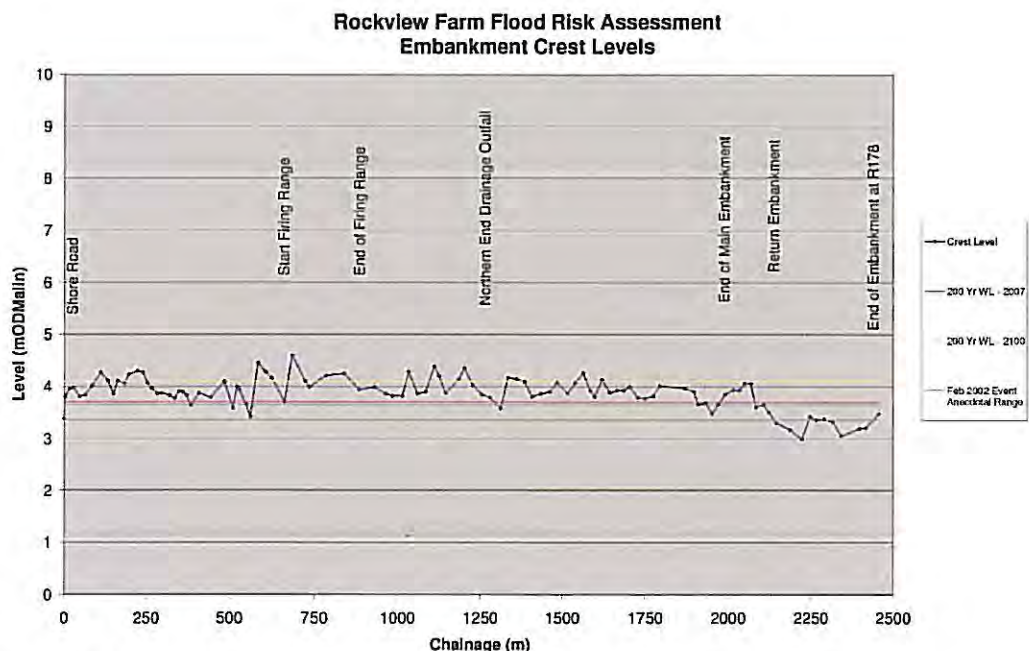


Figure 5-1 – Longitudinal Section of Coastal Embankment

As can be seen from Figure 5.1 the levels along the main coastal embankment generally vary between 3.8 and 4.1 mODM, whilst the lowest and highest levels recorded were 3.42 and 4.59 MODM respectively. The return embankment from the main coastal embankment to the Blackrock Road (R172) is much lower at generally around 3.3 mODM, although at two locations the levels fall close to 3 mODM.

A number of extreme tide levels have also been plotted on the longitudinal section and this includes;

- The anecdotal range provided for the 1st February 2002 storm (3.36 – 3.65 mODM).
- The predicted present day (2007) 200 year extreme tide level – 3.7 mODM.
- The predicted 200 year extreme tide level at the end of the century (2100) – 4.1 mODM.

As can be seen from Figure 5.1 all the embankment crest levels along the main coastal embankment are generally above the February 2002 and present day 200 year levels. There are three locations where the embankment levels begin to fall below these extreme tide levels. This occurs around chainage 550, 1310 and 1950. Despite the embankment crest over the development lands being close to the level of the February 2002 tide at two locations (ch 1310 and 1950) it did not breach. This is a function of the fact that the embankment over the site is in a better overall condition than that further to the north of the site where in fact it failed at 5 locations. It does not necessarily mean that the embankment was not overtopped by wave action.

The return embankment to the Blackrock Road is considerably lower and it is likely that this embankment was overtopped during the February 2002 event and some flooding of the south west corner of the land may have occurred combined with the overall flooding of the area adjacent to southern end of the site via the low levels along the Blackrock Road.

With the onset of climate change and in particular sea level rise, the level associated with a 200 year level at the end of the development design life, which has been taken as 2100, is expected to rise to 4.1MODM. From Figure 5.1 it can be seen that much of the embankment falls below this level. If a water level of this magnitude were to occur significant breaches would occur through the whole of the embankment and much of the hinterland would flood to a significant level.

5.3 Existing Standard of Protection

Based on the comparison presented above and the overall condition of the embankment noted during the site inspections, it can be concluded that the current embankment does not provide an adequate level of protection to the area. Much of the embankment, particularly to the north, is in a poor condition and would certainly breach during extreme events as occurred during the event in 2002. The embankment over the development lands, whilst in a better overall condition, is for the most part too low and/or too narrow and it will only be a matter of time before it is subject to breaches during an extreme event.

The return embankment, even without significant wave action on it, is much too low and narrow and will be overtopped during future extreme tides. The Blackrock Road to the south of the site is also too low and not protected by any formal defence. To the north the adjacent farm lands are also low and a potential flood path exists back through this land to the site should flooding of it occur as a result of breaches in the embankment to the north. Whilst this land was flooded during the February 2002 event, flooding did not extend back to the development site from the north.

Considering the condition of the embankment, the results of the tidal and wave overtopping analysis undertaken for the study and OPW freeboard recommendations as applicable, an assessment of the standard of protection offered by the existing

embankment has been made. Whilst there are a number of isolated low spots along the embankment which would tend to indicate a standard of protection of less than 10 years, the more consistent level of the embankment (around 3.95mODM) would suggest that a higher standard of protection of around a 1 in 40 year return period would apply. This would reduce for the embankment north of the development site given the greater likelihood of failure due to the embankments poorer condition. The embankment to the south, based on the crest levels recorded, can be considered to provide a level of protection of around 1 in 20 years.

These levels quoted whilst unacceptable for significant residential development, may be acceptable for low density rural and agriculture lands.

Whilst the values quoted are lower than that for the February 2002 event, which could be considered to have had a return period of around 100 years at Dundalk, they are considered to reflect the level and condition of the embankments over the site. The fact that the embankment over the development lands site did not fail during the February 2002 event does not mean that the embankment provided an adequate level of protection at that time and it may nevertheless have been heavily overtopped during the event.

5.4 Saltmarsh Evolution

In addition to flood defence an important issue for consideration as part of the development is the issue of coast protection. In this regard the wave climate at this site and the parameters that have an impact on it are important.

For the Rockview Farm site an extremely important element of natural protection against erosion and flooding is provided by the extensive saltmarsh which exists over the whole length of the site.

A saltmarsh generally exists between Mean High Water of a Spring Tide (MHWS) and Mean Low Water of Neap tides (MLWN) and develops through the net accumulation of sediment which then becomes colonised by a few salt-tolerant plants. As these are not subjected to tidal action on a continual daily basis the vegetation becomes more developed and also aids further deposition of sediment. The enhanced vegetation and deposition raises the level of the saltmarsh which in turn becomes covered by fewer tides. In such situations the older saltmarsh can become more of a terrestrial habitat. Indeed this can be noted at a number of locations along the Rockview Farm site where an extensive grass berm has developed along part of the toe of the embankment.

The saltmarsh, by providing raised ground levels and vegetation between the site and the sea, helps to protect against erosion and flooding by reducing the wave climate which can reach the site during higher than normal tide conditions. This is achieved by a combination of wave breaking due to reduced water depth and increased bed friction due to vegetation.

Besides its contribution to flood defence the saltmarsh also has a significant environmental value. It supports many unique species of flora and fauna and is used by significant numbers of waterfowl for roosting during high water. It is for this added

benefit that the whole area has received protection and is designated as a SPA and SAC, as discussed in section 1.2.3.

Given the importance of the saltmarsh for the site it was necessary to consider not just the impact of the development on it but also the health of the salt marsh and whether it can be relied upon to provide continued protection in the future. In particular sea level rise may have a detrimental impact on salt marsh in view of the fact that it may be covered by tides more frequently in the future.

In order to help evaluate this issue it was considered appropriate to carry out an assessment of the evolution of the saltmarsh over recent years. This was undertaken using aerial photographs obtained from Ordnance Survey of Ireland (OSI). Aerial photographs of the saltmarsh in front of the project site were available for 1974, 1995 and 2004. The extent of the saltmarsh has been determined on each of these photographs, as shown of Figures E.1, E.2 and E.3 in Appendix E. These extents were then overlain to provide an overview of the evolution of the saltmarsh over this period.

Figure 5.2 below shows the extent lines for each of these three dates on the 2004 aerial photograph. From the figure it can be seen that from 1974 to 1995 the saltmarsh accreted in most places with the exception of south of the project site. From 1995 to 2004 there has been a slight increase in the extent of saltmarsh over the whole frontage except for a small spot in the centre.



Figure 5-2 Saltmarsh evolution over the years 1974, 1995 and 2004

Overall it can be concluded that the salt marsh has been accreting over the past 30 years and is thus relatively healthy. This tends to confirm anecdotal evidence which

suggests that the salt marsh is growing rather than receding. This also reflects the view of the ornithological assessment report prepared by Biosphere Environmental Services for Tobin Consulting Engineers which states that the saltmarsh has grown considerably in the 20th Century.

What effect future climate change will have on the salt marsh can not be certain. It is possible that it could begin to recede if future estimates of sea level rise are realised. However, given the fact that the salt marsh is noted to be accreting and its overall extensive width, it is considered that the salt marsh can be relied upon for the foreseeable future. The estimates of wave height provided in section 4 of this report are considered to be on the conservative side. As such the options presented in section 6 included for armoured embankments which will allow for a degree of flexibility in the future should any change occur in the salt marsh.

Nevertheless it is recommended that the salt marsh be monitored in the future through the use of aerial photographs.

6 OUTLINE FLOOD ALLEVIATION OPTIONS

6.1 General

For the study a number of options have been developed which are deemed to provide an acceptable level of protection against coastal flooding. This has been taken to be a level of protection against all events up to and including a 1 in 200 year return period. The basis for choosing this is set out in section 6.2 below.

The options have been developed around international and national best practice and the design criteria as set out in section 6.2.

In addition to the issue of flood defence, consideration has been given to the incorporation of a coastal walkway within the final flood defence options. Whilst the walkway itself has not been developed in detail, where and how it can be incorporated within the scheme has been considered.

Three general options for achieving the required level of protection have been considered using different site land fill and embankment levels. These are set out in section 6.3. Within these options there are two embankment options which provide alternative profiles. These options are described in section 6.4. Finally for the southern end of the site an alternative embankment layout is provided. This is set out in section 6.5. The costs of the different options and alternatives are presented in section 6.6 and each of the options are evaluated in section 6.7.

A series of drawings showing each of the options in plan and cross-section have been produced. These are presented in Appendix F and will be referred to in the following sections.

6.2 Design criteria

6.2.1 Standard of Protection

For the development of options it was necessary to consider what might be considered an acceptable level of protection for the new development and area as a whole. In order to determine this, a brief review of international and national best practice was carried out. Furthermore the event of the 1st February 2002 and the flooding that resulted were also considered in the context of the extreme tidal analysis and the hazard associated with the event.

International and National Best Practice

The latest guidance from the UK in respect of indicative standards of protection is presented in the table below.

Table 6-1 - Indicative standards of protection UK

Land Use Band	Indicative range of housing Units per km of coastline	Coastal/Saline Flood Risk	
		Return Period (years)	Annual probability of Failure
A	≥ 50	100 – 300	0.003 – 0.01
B	≥ 25 to < 50	50 – 200	0.005 – 0.02
C	≥ 5 to < 25	10 – 100	0.01 – 0.1
D	≥ 1.25 to < 5	2.4 – 20	0.05 – 0.4
E	> 0 to < 1.25	< 5	> 0.2

In the Netherlands the recommendations are much more stringent with requirements to provide a level of protection equivalent to the 1 in 10,000 year (annual probability of 0.0001) event or greater against tidal flooding along most of the western coast. This is not surprising considering much of the land behind the coastal defence lies below sea level and contains areas with very high population density and high economic activity.

Recent guidance set out in the policy documents developed through the Greater Dublin Strategic Drainage Study (GDSDS) suggest that a 200 year event be used for coastal flooding design on that basis that flooding from the sea tends to be catastrophic.

Furthermore the latest guidance provided by the Office of Public Works (OPW) requires levels of protection of 100 years for fluvial flooding and 200 years for coastal flooding.

Correspondence with the DCMNR as part of this study, (see section 2.1.3), confirmed that their policy is not to recommend a set SoP for developments. They advised that the consequences of exceeding the SoP be taken into consideration and that the highest SoP possible should be used.

February 2002 Event

The February 2002 event caused extensive flooding of the land to the north of the development site and also to the Blackrock Road and properties to the south. Whilst it is not known how many properties actually were flooded during the event of the 1st February 2002, considerable properties exist in the hinterland immediately around and adjacent to the site. The areas to the north are more sparsely populated, however as a whole the area would certainly fall into the land use band B or A as set out in the UK guidelines presented in Table 6.1. The proposed development site caters potentially for the provision of some 1600 housing units and as such must require a level of protection equivalent to that for land code A or better.

The event of February 2002 is considered to have reached a level of between 3.36 and 3.65 mODM and have a return period of around 100 years based on the higher end of the range. As such it was quite an extreme event. A present day 200 year level of 3.7mODM has been predicted by the tidal analysis.

In light of this it is deemed appropriate that the new development and the overall surrounding area is provided with an indicative standard of protection of not less than 1 in 200 years (annual probability of 0.005) and that this should incorporate an allowance for sea level rise up to the end of the century (2100).

6.2.2 General shape of the embankment

Slopes

Best practice suggests that embankments for maintenance reasons should have a maximum slope of 1:2.5. Furthermore it is recommended that front slopes are set at 1:3 as this provides a more robust defence against wave action and also helps to reduce overtopping. A 1:3 slope is also accessible to all common maintenance vehicles. These parameters are acknowledged by the UK Environment Agency's policy for new and improved embankment defences.

Therefore a front slope of 1:3 and a back slope of 1:2.5 have been chosen for the development of the main embankment in each option.

Crest width

Best practice suggests that the minimum crest width for a sea embankment should be 4m. This provides enhanced stability and protection against wave overtopping and also provided adequate room for access by maintenance vehicles. These values are also in accordance with the UK Environment Agency's best practice advice.

For the development of options for the site a crest width of 4m has been chosen.

Berm

Whilst the provision of a berm on the back slope of an embankment is not necessarily required, it will greatly enhance the stability and integrity of the overall embankment. Where an embankment is extremely high, for example when land levels behind are low, it is good practice to provide a rear berm to aid maintenance and provide access along the back slope. In the case of the Rockview Farm development the berm will also provide the ideal location to facilitate the proposed coastal walk between Dundalk and Blackrock Village.

Whilst the level at which the berm is placed can vary, for the purpose of this study it has been placed 1.25m below the crest of the embankment. This is primarily to facilitate the coastal walkway. At this level it will be low enough to minimise disturbance to the birds using the SPA, while ensuring that users of the walkway will still be able to enjoy views of the surrounding area. For the purpose of incorporating the coastal walk and general maintenance access a rear berm width of 4m has been considered.

It should be pointed out that the value of 1.25m considered above is indicative at this stage and will require more detailed consideration should the proposed development progress in order to ensure minimal disturbance on the birds using the SPA. A discussion with Biosphere Environmental Services has confirmed that initially this value can be considered for much of the embankment. However, they have recommended that at particularly sensitive locations, such as bird roosting sites, the berm may need to be lowered and placed at around 1.8m below crest level. A roosting site exists towards the southern end of the site and it may be necessary to lower the rear berm over this location.

Drainage Ditch

The current situation at the site includes a drainage ditch behind the embankment. This and other ditches across the site transfer the runoff from the site and wider area to an outfall which discharges through the embankment. As such the ditch will need to be maintained behind the new embankment. In addition to the issue of land drainage, it is common practice to place a ditch behind a coastal embankment as it can collect water which might overtop the embankment. Furthermore the material excavated for the construction of the ditch can contribute to the construction of the new embankment.

In order to ensure sufficient capacity in the rear ditch for drainage from the hinterland and minimise the risk of blockages a minimum base width of 2m has been chosen with side slopes of 1:1.5. These slopes should also improve the stability of the ditch.

6.2.3 Crest level of the embankment

The crest level of the embankment has been determined by the amount of overtopping that is acceptable for any given situation. Table 6-2 below gives the maximum mean overtopping rates for different situations in litres per second per metre run, as given by the CUR Rock Manual [2] and HR Wallingford report W178 [3].

Table 6-2 Maximum Mean Overtopping Rates in l/s/m

	Results if overtopping (l/s/m) > indicated value	
	CUR Rock Manual [2]	HR Wallingford W178 [3]
Danger for pedestrians	>0.03	
Damage if crest not protected	1.7 to 20	2 to 20
Damage if back slope not protected	20 to 80	20 to 50
Damage even if fully protected	>80	>50

The combination of tide levels and wave heights, as determined in chapter 4, together with the crest level and shape of the embankment will determine the mean overtopping that could occur.

The options developed in section 6.3 have been tested for overtopping using a number of methods and developed to provide an acceptable level of overtopping for a given defence situation. The defence situations and the results of the overtopping analysis are presented for each option.

Three methods for assessing the overtopping within this study have been considered and include,

- A. Wave overtopping model AMAZON.
- B. An overtopping spreadsheet based on the EA report W178 "Wave overtopping of seawalls"[3] produced following research by HR Wallingford.
- C. An overtopping spreadsheet to assess the mean overtopping for an embankment developed by University of Liverpool.

6.2.4 Armouring of the embankment

The design conditions for the armouring of the embankment are determined by the wave conditions that act on the embankment. Best practice guidance would suggest that for grassed embankments a maximum wave height of 0.5m on the front face is considered to be the safe limit. Values above this and higher overtopping rates, as set out in Table 6.2, and it is considered necessary to provide protection to the embankment on the front and also the crest and rear slopes depending on the situation.

Considering the wave heights determined in section 4 that could potentially exist in front of the embankment, it is considered that some form of armour should be provided on the front slope. Furthermore depending on the option presented, the overtopping rates would suggest that in some cases the crest and the rear slope should also be protected.

A range of different armour protection methods could be used and will depend on the size of the waves against which protection is required. It is considered that given the location and wave climate at the site rock armour will not be required. Instead some form of proprietary armour protection system would be suitable. For the purpose of this study the Armourflex system has been chosen. This is mainly due to its suitability against wave climate and for its relatively easy installation process. The Armourflex system comes in either solid or cellular block mats. With the cellular block mats up to 35% of the surface area is open. This permits the establishment of vegetation across the armour. For the Rockview Farm development use of the cellular mats is recommended and the opening filled with topsoil and seeded. Details of this system are presented in Appendix G.

A revised system providing similar characteristics could be used if deemed suitable at the detailed design stage.

6.3 General options

6.3.1 Option 1 – Improve Main Coastal Defence

The first option considers improvements to the main coastal defence, provision of a new cross embankment at the northern boundary, improvements to the southern return embankment and the provision of a new embankment along the Blackrock Road at the southern end of the site. This option does not allow for any raising of the development lands but assumes the land would be developed at a level of +3mODM. This option is presented on Figure F1 in Appendix F.

Given the low lying level of the proposed development land to the rear of the embankment it is extremely important that the design of the embankment considers an extremely low probability of failure in the future. This can be achieved by providing an embankment design that allows for virtually no overtopping. By limiting overtopping to a minimum the failure of the embankment will also be limited. Given the design wave heights predicted during extreme tides the option would require the front face of the embankment to be armoured with armourflex.

This option requires a relatively high and large embankment but requires only a limited amount of landfill to provide a uniform level over the site. Based on the results of the

overtopping assessment a crest level of 6.25mODM has been chosen. Typical sections through this embankment are shown in Figures F2, F3 and F4 in Appendix F.

The overtopping quantities associated with this option for each of the overtopping models presented in section 6.2.3 is shown below in Table 6-3.

Table 6-3 Overtopping of Option 1 Main Embankment

Tide Conditions	Overtopping Model – Mean Overtopping		
	A (l/s/m)	B (l/s/m)	C (l/s/m)
WL – 3.7mODM	0.0013	7.98	0.006
WL – 4.1mODM	0.8	24	2.4

It can be seen from the results presented above that model A and C provide similar results while model B provides high overtopping estimates. Experience suggests that model B tends to predict very high overtopping. The results of model A and B are considered to be acceptable based on the allowable limits presented in Table 6.2.

This option only requires armouring on the front slope of the embankment. In order to stabilise the armour in the short term it is extended onto the crest by four blocks and anchored into the embankment. It is recommended that the improved main embankment should be continued for approximately 100 metres beyond the northern end of the site and the start of the northern boundary embankment.

At the southern end of the site the return embankment also requires improvement. Due to the sheltered location of this embankment, being in the lee of the main coastal embankment, it will not require as high a crest or armour protection. For this location the embankment crest has been set at 4.6mODM. A typical section through this embankment is shown in Figure F13 in Appendix F.

Where the return embankment meets the Blackrock Road, the embankment will be required to extend along the road until it ties into high ground just before Blackrock Village. To facilitate the construction of the new embankment the culvert under the Blackrock Road which discharges a small stream in this area will need to be extended. To secure the hinterland against tidal flooding a tidal flap would need to be provided on the seaward side of the culvert.

The embankment along the road should be constructed at a slightly higher level due to the possibility of some low wave action penetrating into this area during an extreme event. Given the gap through which the waves must propagate and the high levels and vegetation in this area, the waves are likely to be considerably lower than for the main coastal frontage. For this study the embankment has been developed considering a wave of $\leq 0.5\text{m}$. As such the embankment should be constructed with a crest level of 5.25mODM and can consist of a grass embankment only. A typical section through this embankment is shown in Figure F14 in Appendix F.

It is recommended that the wave climate in this area be clarified during the detailed design stage to confirm the design of the embankment at this location.

Where the embankment moves out of this area and around the corner towards Blackroad Village the crest level should be raised and the front face armoured due to the more exposed location.

At the northern boundary a new return embankment would be constructed back from the main embankment around the firing range and along the access road to meet the Blackrock Road. This embankment would be constructed to a level of 4.6mODM due to the sheltered nature of the location. A typical section through this embankment is shown in Figure F13 in Appendix F.

Whilst this embankment would effectively seal off the site from the north, the Blackrock Road and land to the west are low lying. As such a flood path in theory could still exist into the development lands via the road should extensive flooding of the land to the north occur. However, it is considered that this is extremely unlikely given the extensive width of land to the north over which the flood would be required to pass. Furthermore given that the development lands did not flood during the event of the 1st February 2002 when no such embankment was in place, it is unlikely that sufficient volumes of water would exist to out flank the northern embankment.

To further safe guard the site the embankment could be returned along the western boundary and Blackrock Road and the land around the access to the site landscaped to provide access over the defence. Due to the open drains which pass through the site, several culverts with penstocks would be required to fully seal the site during an event. This would introduce the added difficulty in that someone would need to be warned of an event in advance to close the penstocks in order to fully seal the site. This would also minimise the storage capacity of the open drains which would back up outside of the development with an increased risk of flooding from this mechanism beyond the site boundaries.

For these reasons this option is not considered to be the most sustainable from a long term perspective and is thus not preferred.

6.3.2 Option 2 – Improve Main Coastal Defence and Raise Development Lands

The second option considers improvement to the main coastal defence but to a lesser extent than proposed in option 1. It also considered the provision of a new northern boundary embankment over the limit of the recreational lands and the improvement of the return embankment at the southern boundary back to the Blackrock Road. In addition to this the option also incorporates the infill of the development lands to a level above the design extreme tide level. In this way the flood defence is built into the development itself by placing the properties and access roads at a level above the extreme tide. Option 2 is shown in Figure F5 in Appendix F.

Given the 200 year design tide level in 2100 of 4.1mODM, it is considered that a further limited freeboard of 0.15m is applicable and that the residential lands should be infilled to a level of 4.25mODM. It is proposed that the amenity lands should be infilled to a lower level of 3.5mODM. This is considered acceptable since it is also proposed that the main embankment will be improved thus providing protection to this area. Furthermore any club houses etc that are proposed to be developed within the amenity

lands to facilitate sports pitches should be constructed with a higher threshold level of 4.25mODM.

Given the level of infill to the residential area of the development lands, it might seem unnecessary to improve the condition of the main embankment. However, this is necessary for a number of reasons which include

- To provide long term coast protection to the overall area
- To provide protection to the amenity lands
- To safely facilitate the future provision of the Dundalk to Blackrock coastal walk.

Whilst ensuring that each of the above objectives are met the embankment can still allow increased overtopping quantities compared to that which was acceptable for option 1. In this way the crest can be lower. The main task of the embankment is to keep the overtopping to an acceptable level so that the amenity area is not flooded frequently and persons can use the coastal walkway during normal tidal events.

Based on the acceptable overtopping limit criteria an embankment with a crest level of 5.6mODM is proposed. Since some higher overtopping levels would be permitted during the design 200 year event the crest of the embankment would also need to be armoured. Typical sections through option 2 for the main embankment are shown in Figures F6, F7 and F8 in Appendix F.

The overtopping quantities associated with this option for each of the overtopping models presented in section 6.2.3 is shown below in Table 6.4.

Table 6-4 Overtopping of Option 2 - Main Embankment

Tide Condition	Overtopping Model – Mean Overtopping		
	A (l/s/m)	B (l/s/m)	C (l/s/m)
WL – 3.7mODM	0.18	27.8	1.8
WL – 4.1mODM	6.4	75.6	21.1

It can be seen from the results presented above that model A and C again provide more similar results while model B provides high overtopping estimates which are considered to be over conservative. The values provided by model A and C are considered to be acceptable based on the limits provided in Table 6.2 for an armoured embankment front face and crest.

Whilst higher overtopping rates are expected they are deemed acceptable on the grounds that the volumes will be sufficiently low so as not to significantly flood the amenity lands. Indeed if the overtopping rate in 2100 for model C is considered to act for 1 hour then it would result in a volume of water which would flood the amenity lands to a depth of not more than about 0.3m deep. Furthermore it is assumed that the walkway would not be used during storm events of return period greater than about 1 in 20 years when overtopping quantities would be higher and uncomfortable.

At the northern boundary of the site a return embankment would only be required over the length of the amenity lands in order to protect them from flooding to the north. The residential lands would not require further protection given the elevated level to which the area would be filled.

To the south the options for the return embankment back to the Blackrock Road would be similar to that of option 1. The embankment would protect the amenity land and should be continued back along the residential boundary also at 4.6mODM. This is considered necessary in view of the slightly more exposed nature of this southern boundary in that direct inundation by the sea of the adjacent land to the south is possible. This is not the case at the northern boundary of the residential area which is set back some distance from the sea and adjacent to an area of land which is not normally inundated by the sea due to it being protected by the existing embankment which continues north of the proposed development site.

This option is considered to be sustainable in the longer term as it makes use of elevated property levels which provides protection to the new development with out enhance flood risk at adjacent sites. The option also actively provides the foundation for the completion of the coastal walk by ensuring it is adequately catered for within the proposed development and improved coastal defences.

6.3.3 Option 3 – Improve Main Coastal Embankment and Provide New Secondary Embankment.

The third option considers improvements to the main coastal defence, but to a lesser extent than proposed in option 2, and a new secondary embankment placed at the interface between the residential and amenity areas. It also considered the provision of a new northern boundary embankment back to the Blackrock Road, the improvement of the return embankment at the southern boundary and the provision of a new embankment along the Blackrock Road towards Blackrock Village. In addition to this the option also incorporates the infill of the proposed development lands to a level of 3.5mODM whilst leaving the amenity lands at a level of 3mODM. Option 3 is shown in Figures F9 in Appendix F.

The aim of this option was to secure the development site by using a secondary embankment and to minimise the amount of infill required. By providing a secondary embankment increased volumes of overtopping can be permitted on the front embankment and hence a smaller embankment can be permitted. This would increase the flood risk to the amenity lands and effectively promote its use more in the role of a flood storage area. The residential lands would be raised to a level above the Blackrock Road which would reduce the risk in the unlikely event that flooding to the north should extend as far back as the road.

Based on the overtopping assessment, it is considered that a minimum safe crest level of 5.0mODM would be used for the front embankment. This is generally around 0.5 to 1m higher than the level of the current embankment. This level would also ensure that overtopping is kept to a minimum during normal operational conditions and hence promote the safe use of the coastal walk during such time. During a design 200 year event the quantities of overtopping would be sufficiently high to warrant a fully protected embankment and as such armour would be required on the front slope, crest and rear slope. During such extreme events it is considered that the walkway would not be in use. Typical sections through option 3 for the main embankment are shown in Figures F10, F11 and F12 in Appendix F.

The overtopping quantities associated with this option for the main embankment and each of the overtopping models presented in section 6.2.3 is shown below in Table 6.5.

Table 6-5 Overtopping of Option 3 - Main Embankment

Tide Condition	Overtopping Model – Mean Overtopping		
	A (l/s/m)	B (l/s/m)	C (l/s/m)
WL – 3.7mODM	2.7	63.3	11.1
WL – 4.1mODM	28.6	140	59.7

It can be seen from the results presented above that rates are considerably higher than for option 2. Again the results for model B are considerably larger than the other two. However, in general it can be seen that the results generally fall within the limits provided in Table 6.2 for fully armoured embankments i.e. < 50 to 80 l/s/m.

The crest level of the secondary embankment around the residential area has also been set at 5.0mODM. A typical section through the embankment is shown in Figure F15 in Appendix F.

Whilst a lower level could be considered due to the lack of wave action at this set-back location, it is considered prudent to place it at a similar level to the main embankment in order to reduce the risk of flooding across it. This will ensure that water ponding within the amenity lands during a design storm will not overflow the lower secondary embankment, but would either be stored in the amenity area and discharged out via the drainage system on the falling tide or could overtop the slightly lower embankments to the north and south side which have been set at 4.6mODM should the levels of flooding reach these levels. The latter is considered to be very unlikely and if the overtopping rate in 2100 for model C is considered to act for 1 hour then it would result in a volume of water which would flood the amenity lands to a depth of around 1m. With the lands set at a level of 3mODM, it is more likely that the water will be stored and discharged on the falling tide.

For this option similar northern and southern boundary options would be required as set out for option 1.

As such a potential flood risk exists via a very extreme event from the northern lands. However, the slightly raised residential lands further reduces this risk as depths of water in excess of 0.5 metres deep would be required at the northern end of the Blackrock Road which is considered to be very unlikely.

Nevertheless whilst this option is considered to be better than option 1, it is less sustainable in the longer term than option 2 and is thus not preferred.

6.4 Embankment options

One of the objectives of DTC is for the provision of a coastal walk from Dundalk to Blackrock Village. As such any improved flood defence options considered for the proposed new development site must ensure that this vision can be facilitated now or in the future. Within this study we have considered how such a walkway can be incorporated within the overall scheme. However, specific details of the walkway itself

have not been included. To accommodate for the walkway two different lay-outs for the embankment have been developed.

In addition to this it is considered that a degree of flexibility should be designed into the improved embankments to allow for the possibility of future raising should actual values for sea level rise exceed those which have been recommended for use in this report. To this end a flat slope, wide embankment crest and rear berm will all provide for increased flexibility in the future.

Embankment 1

Embankment 1 consists of a typical trapezoidal embankment with a berm at the toe of the back slope between the drainage ditch and the toe. The level of the berm would be placed at the same as the level of the amenity area after the site has been levelled or landfilled, depending on the option. Typical sections showing this embankment layout for option 1 are shown in Figures F2, F3 and F4, for option 2 in Figures F6, F7 and F8 and for Option 3 in Figures F10, F11 and F12. All of these Figures can be found in Appendix F.

With this embankment the walkway could be placed on the crest of the embankment or on the berm at the rear toe.

Walkway on Crest

With the walkway placed on the crest it would need to be of hard construction such as concrete and would effectively replace the armour on the crest if it was required. The front and/or rear face armour could be anchored into the concrete walkway on the crest.

This option would have the advantage that it maximised the views of the surrounding area and would thus be preferred in this respect. However, it would be the option that has the most impact of the SPA as the users would be clearly visible and hence most disturbing to the birds. To minimise this some form of screening could be placed along the front of the crest which would act to hide the greater footprint of the users and also prevent access, particularly for pets, to the saltmarsh. The other disadvantage of placing the walkway on the crest is that it would be more subjected to direct overtopping particularly for options 2 and 3 where the crest level is lower. The environmental impacts of such an option have been address in more detail in the ornithological assessment report produced by Biosphere Environmental Services⁴.

Walkway on the Toe Berm

With the walkway placed at the toe of the rear slope, it would be placed a minimum of 2 metres below the crest of the embankment for any of the options proposed. As such the users would not have unrestricted views of the surrounding area, which is one of the objectives of DTC for the walkway. It would be the best option from the point of view of minimising disturbance on the designated sites, since the users would not be visible. Furthermore it would move the walkway away from direct overtopping and hence increase the safety aspect during more extreme conditions. In order to minimise access

⁴ Proposed Dundalk to Blackrock Coastal Walkway, Ornithological Assessment, May 2007, Biosphere Environmental Services.

to the saltmarsh and ensure the safety of the walkway users it is considered that two fences would be required along the walkway. One placed on the toe of the rear slope to prevent access over the embankment to the saltmarsh and one placed along the edge of the drainage ditch.

Embankment 2

Embankment 2 is similar to the first with the exception that a berm is placed on the rear slope. This has added value both from a flood defence point of view and also for the provision of the coastal walk. In the first case a berm placed on the rear slope increases the overall width of the structure and hence improves the structures robustness and integrity. This is particularly important for the options where increased overtopping is permitted. For the walkway it provides an elevated platform on which the walkway can be placed, without the direct impacts that placing it on the crest would have.

For this option a berm placed at a level of 1.25m below the embankment crest level has been chosen. This will ensure that most users, with the exception of small children, have a view over the surrounding area. The user's silhouette will largely be hidden behind the crest of the embankment thus minimising disturbance to the fauna present on the salt marsh. Typical sections showing this embankment layout for option 1 are shown in Figures F2, F3 and F4, for option 2 in Figures F6, F7 and F8 and for Option 3 in Figures F10, F11 and F12. All of these Figures can be found in Appendix F.

Where embankment 2 has been considered in options 1 and 2, an extra 2m wide berm has been placed along the toe of the rear slope and the ditch. This serves a number of purposes in that, it stabilises the long slope between the berm and the bottom of the ditch, aids maintenance access along the ditch and improves safety of the walkway by further segregating them from the drainage ditch. Furthermore should additional vegetation be required to further reduce silhouetting of the users, this berm could be used to facilitate this function thus ensuring that bushes and trees are not planted into the flood defence structure.

As mentioned in section 6.2.2, discussions with Biosphere Environmental Services on the options proposed has indicated that for this embankment option the level of the berm on the rear slope may need to be lowered over particularly sensitive sections of the walkway. This would be the case for sections adjacent to roosting sites. A know roosting site exists over the central section of the site and at this location it would be proposed to lower the berm level such that it was 1.8m below the crest.

Conclusions

It is concluded that the coastal walkway can best be facilitated within the improved defences by constructing a berm on the rear slope just below the crest of the improved defence. This will improve the overall integrity of the defence and ensure the future vision for the walkway can be achieved with minimal disturbance to the designated sites. The level of the berm may need to be placed at a low level relative to the crest at bird roost sites.

6.5 Southern Boundary

6.5.1 Proposed option south

The proposed option to seal the southern boundary of the site has been described in section 6.3 for the main options. This includes improvements to the return embankment and a new embankment along the Blackrock Road towards Blackrock Village.

6.5.2 Alternative south

Another alternative solution at the southern boundary would be to continue to the main embankment south to meet the Blackrock Road just north of Blackrock Village. This option is shown in Figure F16. This would remove the need for a lengthy embankment around the inlet to the south of the site.

As this embankment would be subjected to wave action it will have the same dimensions as the primary embankment for each of the proposed options. This alternative south option is applicable mainly to option 1 which provides for no secondary protection to the proposed residential land. However, it could also be considered for the other options with a view to providing protection to the area adjacent to the southern end of the site and as it would provide the most direct route for the proposed coastal walk.

The main disadvantage is the fact that the embankment would cross a number of environmental designations as shown in Figure 1-4. Whilst the ornithological assessment concludes that it is unlikely to have any negative impact on the waterbirds in the area, it may have impacts on the habitats being located within the Dundalk Bay candidate Special Area of Conservation and would thus required further assessment.

6.6 Costs of the options

Outline costs estimates for each of the options have been prepared and are presented below for each option and each embankment alternative. The costs have focused on the provision of improved embankment works and do not included for the following

- Any infill for the site required under option 1, 2 or 3.
- Maintenance costs.
- The new coastal walkway.
- Consultancy fees.
- Other third party costs.

Table 6-6 – Option Costs for Improved Embankments

	Option 1		Option 2		Option 3	
	Embankment 1	Embankment 2	Embankment 1	Embankment 2	Embankment 1	Embankment 2
Main option	€6,655,000	€6,970,000	€5,230,000	€5,310,000	€6,235,000	€6,130,000
Alternative south	€6,770,000	€7,160,000				

The options costs are inclusive of VAT.

The options include preliminary items at 15% and contingencies at 10%.

The above costs include for the provision of the following lengths of new and improved embankments as detailed in section 6.3 to 6.5.

Table 6-7 – Length of New and Improved Embankment for Each Option

Embankment Details	Main Options			Alternative South
	Option 1	Option 2	Option 3	Option 1
Main Coastal Embankment	1400m	1400m	1400m	1400m
Southern Return Embankment	385m	385m	385m	
Northern Return Embankment	1260m	745	1260	1260
New Southern Embankment along Blackrock Road	640m		640m	
Set-back Embankment			1150	
Alternative South Embankment				320
Total	3685	2530	4835	2980

It should be noted that whilst options 2 and 3 are cheaper, additional costs associated with infill of the site have not been included here. If these were to be included option 2 would be the most expensive option.

6.7 Evaluation of options

Each of the options presented above will provide adequate flood protection to the new development site against flooding up to and including a 200 year event. Option 1 and 3 are considered to be less sustainable in the longer term due to the lower level of the development site and the fact that protection is provided mainly by embankment defences. Option 2 is considered to be the most sustainable option in the longer term. It provides adequate protection to the development through raised ground levels, but also provides long term coast protection and protection to the amenity lands through the incorporation of improvements to the existing embankment. Option 2 will be the most expensive option given the extensive land fill required. However, it is considered to be the preferred option and is recommended to be adopted for the development site.

In respect of embankment options, embankment 2 is preferred. It provides the best foundation for the incorporation of the coastal walk with minimal impact on the designated area. It also provides a more robust flood defence structure.

For the southern boundary of the site it is considered that an embankment placed around the inlet will raise the least objections from an environmental perspective and thus should be considered.

APPENDICES

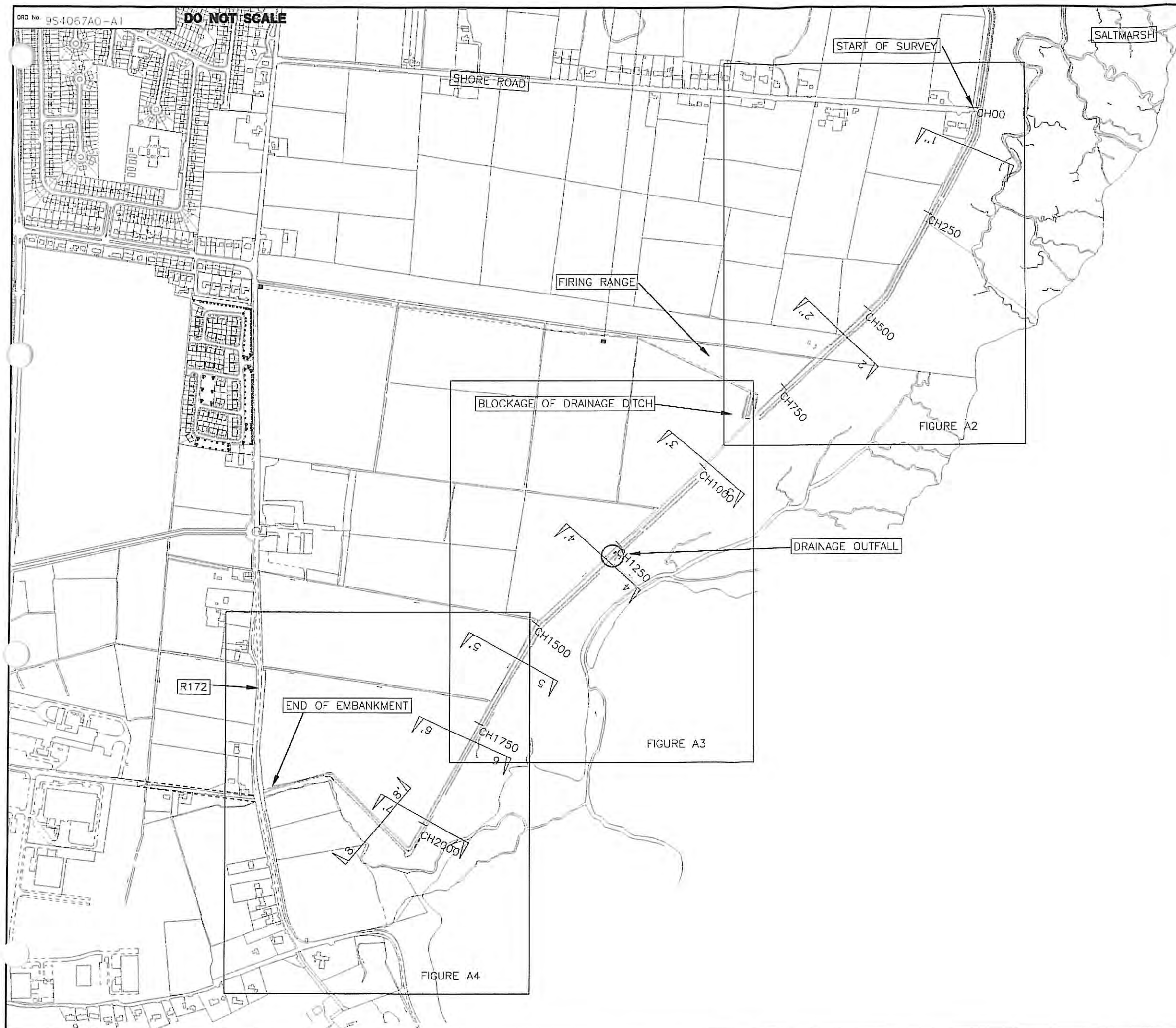
Appendix A Site Plans

DO NOT SCALE



NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDINANCE DATUM MALIN HEAD



REV	DATE	DESCRIPTION	BY	CHK	A
-----	------	-------------	----	-----	---

REVISIONS

ROCKVIEW FARM FLOOD RISK ASSESSMENT

TITLE

OVERVIEW SITE PLAN

HASHING (IRELAND) LTD

19 LOWER SAGGOT STREET
DUBLIN 2
CO D03 (01) 6346300
CO D03 (01) 6346300
e.moroney@royalhashing.com
www.royalhashing.com

Telephone
7
E
Irish

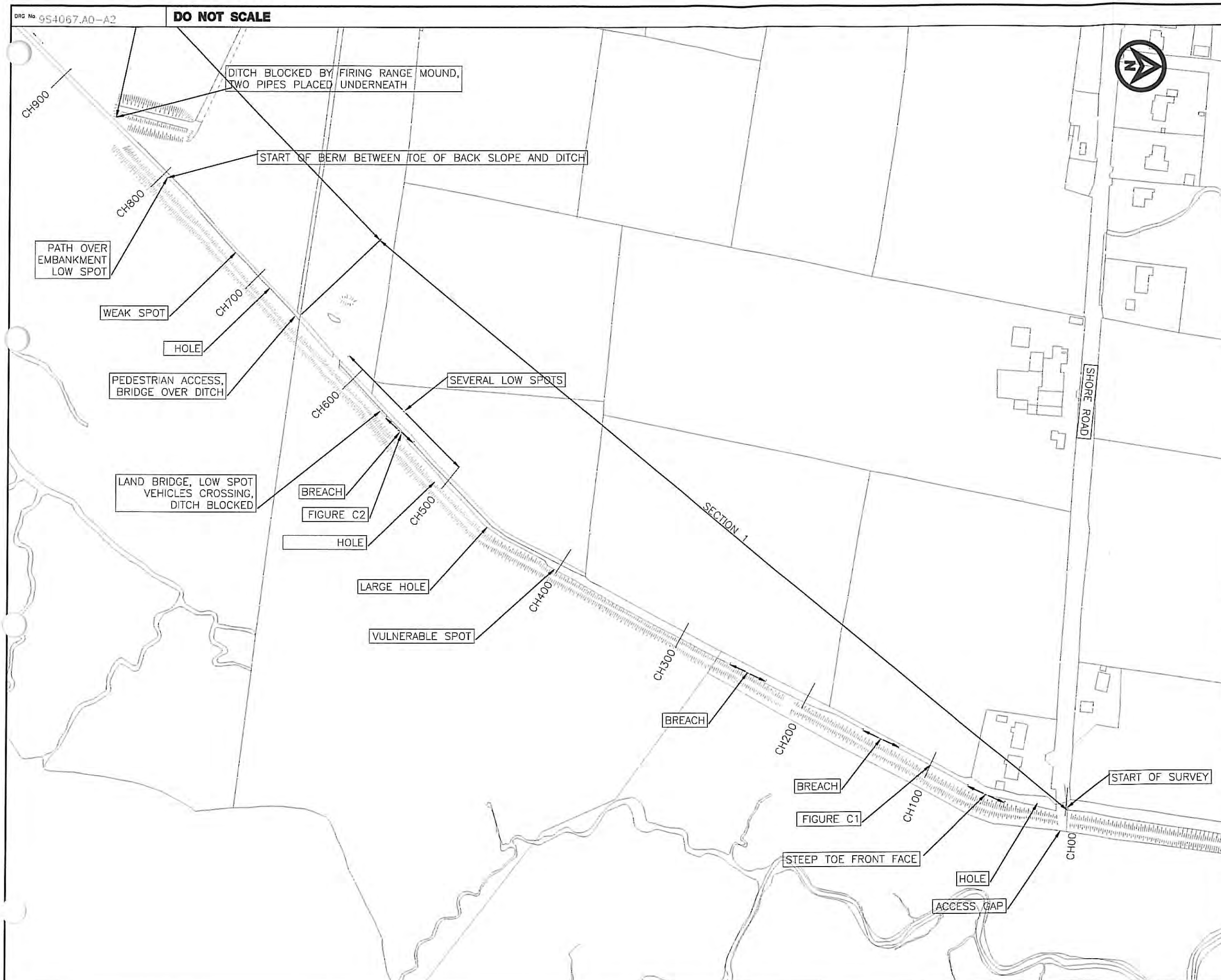
DRAWN GG	CHECKED NM	PASSED
----------	------------	--------

DATE 02-05-07	CLIENT'S REF.
---------------	---------------

SCALE: AT A1	AUTOCAD REF. 9S4067AO-A
--------------	-------------------------

DATE	REVISION
DATE	REVISION

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MAIN HEAD.
3. FIGURE C1 SHOWS THE LOCATION OF THE PHOTOGRAPHS AND CROSS-SECTIONS PRESENTED IN FIGURE C1 IN APPENDIX C.



REV	DATE	DESCRIPTION	BY	CHK	APP

REVISIONS

ROCKVIEW FARM FLOOD RISK ASSESSMENT

TITLE
DETAILED SITE PLAN
1 OF 3

ROYAL HASKONING

19 LOWER BAGOOT STREET
DUBLIN 3
01 563 0154/0154235
01 563 0154/0154235
r.haskoning@royalhaskoning.com
www.royalhaskoning.com

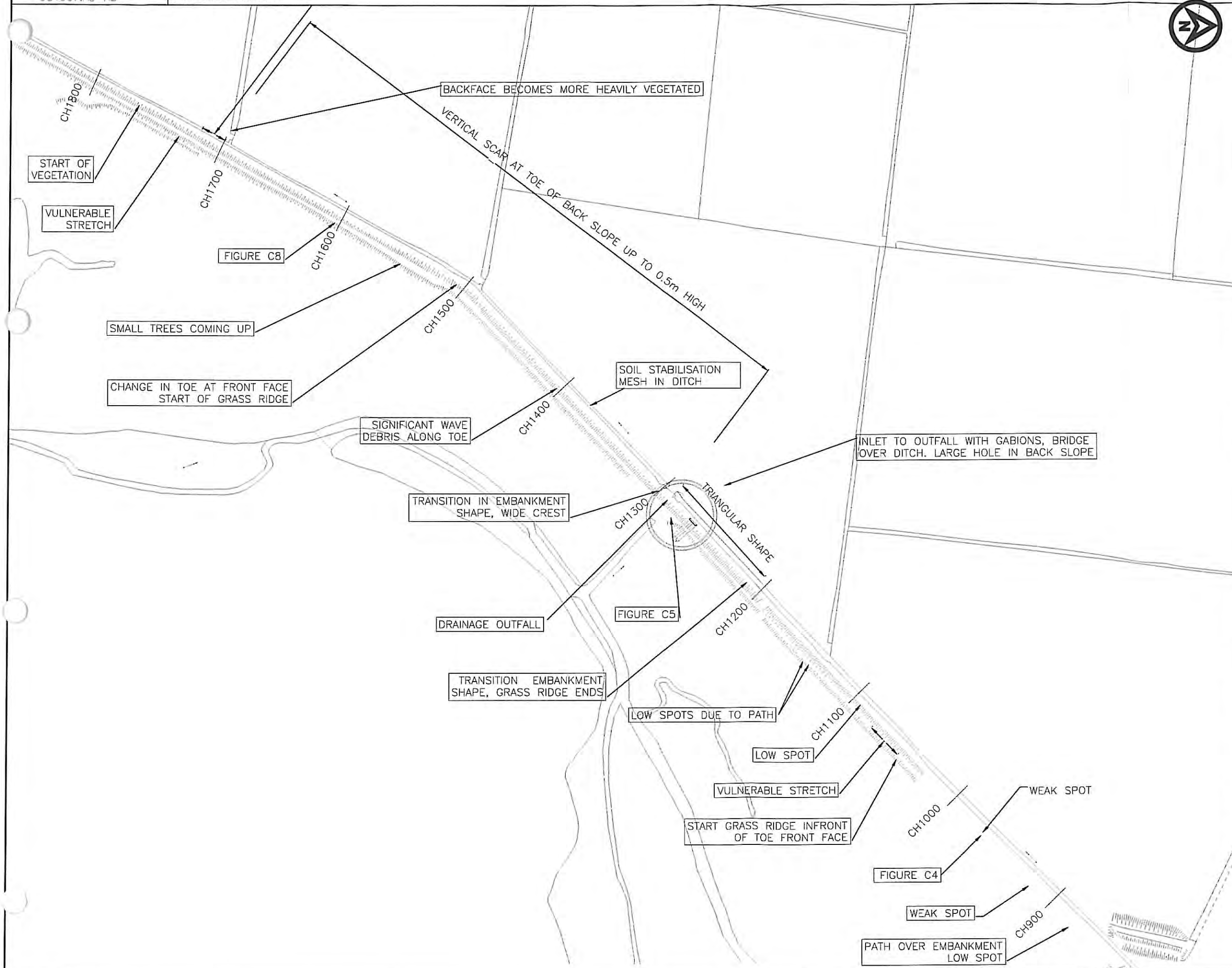
Telephone
Fax
E-mail
Internet

DRAWN	GG	CHECKED	NM	PASSED
DATE	02-05-07	CLIENT'S REF.		
SCALE	AT A1	AUTOCAD REF.	9S4067.A0-A2	
DRAWING No.	FIGURE A2	REVISION		



NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MAUN HEAD.



REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

REVISIONS

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

DETAILED SITE PLAN
2 OF 3

ROYAL HASKONING

18 LOWER BAGOOT STREET DUBLIN 3
01 353 0154333 Telephone
01 353 0154333 Fax
e.haskoning@royalhaskoning.com Email
www.royalhaskoning.com Website

DRAWN GC	CHECKED NM	PASSED
----------	------------	--------

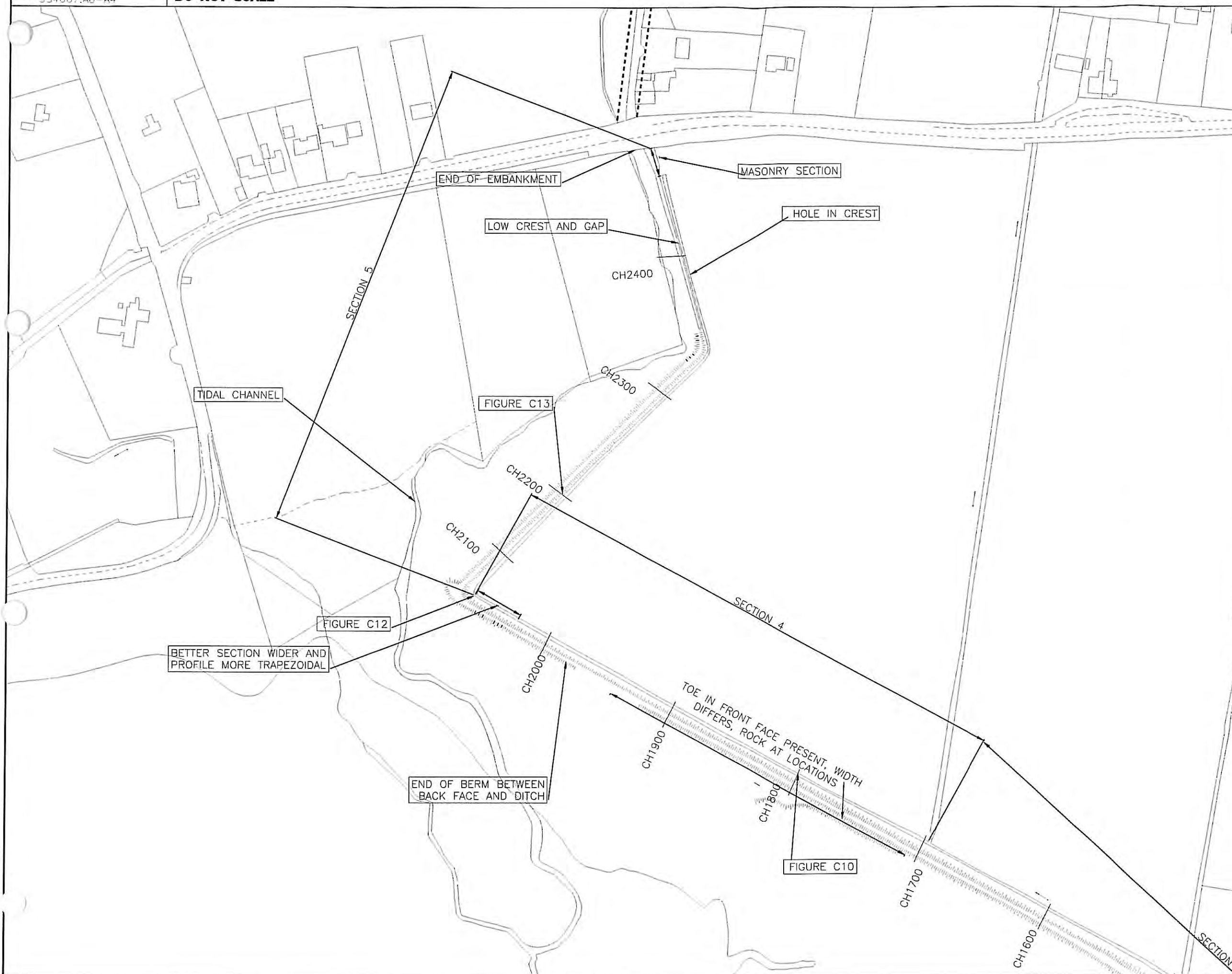
DATE 02-05-07 CLIENT'S REF.

SCALE AT A1 AUTOCAD REF. 9S4067.A0-A3

DRAWING No. REVISION

FIGURE A3

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MAJN HEAD.



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

DETAILED SITE PLAN
3 OF 3

ROYAL HASKONING 19 LOWER BAGOOT STREET DUBLIN 8 00 353 (0)1 454 5555 a.haskoning@royalhaskoning.com www.royalhaskoning.com			
DRAWN GG	CHECKED NM	PASSED	
DATE 02-05-07	CLIENT'S REF.		
SCALE AT A1	AUTOCAD REF. 9S4067.A0-A4		REVISION
DRAWING No. FIGURE A4			

Appendix B
Minutes of Consultation Meetings

Appendix B1
Minutes of Meeting DTC

Minutes

HASKONING UK LTD.
COASTAL & RIVERS

Present : DTC: Catherine Duff (CD), Peter McVeigh (PMcV)
RH: Owen McManus(OMcM), Niels Minnen (NM)
TCE: Tony Curran (TC)

Absent :

Date : 14/03/2007: 11.30 am

Copy : Hamish Hall (RH), John Sherwood (TFP), Sean
Lambe (SL)

Our reference : 9s4067A0/C/330004/lrel2

Subject : Meeting Dundalk Town Council (DTC) Rockview
Farm Flood Risk Assessment

Action

These minutes have been produced based on the topics set out in the agenda for the meeting. A copy of the agenda has been enclosed for information.

1. Introduction

- 1.1 OMcM introduced RH and its activities in Ireland to DTC. DTC comments that they knew of RH.

2. Rockview Farm Development General Overview

- 2.1 All parties are familiar with the proposed developments. DTC comments that RH should have a copy of their development plan. This is available on CD and DTC will provide RH with a copy

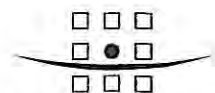
DTC

3. DTC Planning and development policies and requirements

- 3.1 OMcM enquired about design floor levels for properties in areas subjected to coastal flood risk. DTC indicated that historically planning permission was granted on floor levels but that these have varied over time based on the latest information available.
- 3.2 PMcV explained the back ground to the current guidance and threshold level recommendations.

The highest anecdotal measured tide to date is 3.65 mODM on the February 2002 event. DTC consulted the OPW who advised a further allowance of 0.5 for climate change (relevant for 50 years). In addition to this DTC considered a freeboard of 0.3m, which gave a total level of 4.45mODM.

- 3.3 However, PMcV explained that the councils feeling is that Dundalk would be provided some level of protection in the future due to its



Action

status should climate change really take off. As such they reduced the requirement for current floor level to 4.15 mODM, which consists of the highest tide level plus a reduced allowance for climate change to 20 years (0.2m) and 0.3 freeboard. This way DTC can review the issue of climate change in the future and see how it is developing.

3.4 CD explained the history of the Rockview Farm land development was that from a technical side DTC recommended that the site should not be developed due to flood risk.

3.5 In terms of the proposed walk/cycleway and study undertaken in 2003, DTC explained that the proposed walk/cycleway at that time was considered in the context of flood risk potential against pedestrians using the walkway and not flood risk of the land to the rear. As such a 100 year event was considered at the time but now in the context of development flood risk a 1:200 year standard of protection against flooding would be required.

3.6 Outstanding issues regarding the Rockview Farm property development are:

- flood risk not resolved yet;
- recommendations need to be made regarding the walkway, taking flood risk into account;
- the SPA and SAC present in the area and the environment in general have to be taken into account and the impact of any flood risk options considered in some detail.

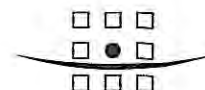
CD was concerned as to the extent of our remit in this context.

OMcM explained that our brief did not cover environmental assessment and that we assumed this would be undertaken by Tobin or Brian Madden working on Tobin's behalf. RH would of course liaise with the environmental scientists in respect of options being considered such that they could assess the impacts. The client has made it clear that they want all issues clearly resolved and covered. RH will work together with Brian Madden on this issue. CD requested that Tobin revert back with some detail as to how this is to be dealt with.

TCE

3.7 OMcM explains that the current brief for the RH flood risk assessment runs from the Shore Road to the southern border of the lands to be developed, including looking at the possibilities for flood water to enter the site from the area to the south via the road to Blackrock (R172). OMcM asked if this was satisfactory and explained that we would not be looking at the embankment north of Shore Road. DTC agreed that this extent was acceptable for the flood risk assessment.

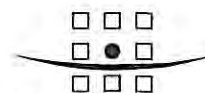
3.8 OMcM asks how the highest Dundalk tide level (3.65mODM quoted) was determined. DTC explains that the highest tide recorded in Dundalk was at the 1st of February 2002. The level of this tide was recorded based on the level the water reached on Quay Street in front of the Harbour Commissioners office. The tide level was estimated to be 18 inches over the quay level. The water crossed the road and came close to flooding some basements of the building behind but did



- not.
- 3.9 DTC explained that the preliminary design of the coastal walkway by KMM was required to be for a 1:100 year event. Now in terms of flood risk a 200 year level of protection is more appropriate. KMM were subsequently asked to consider this issue in the context of deriving the 1:200 year event and determining how much higher the embankment might need to be to achieve this level of protection. They have not redesigned it. This is probably the basis of the additional 2 pages which RH has received from Tobin with revised extreme tide levels date August 2005.
- 3.10 OMCM asks about the requirement for the walkway, in that it will be required to serve pedestrians, cyclists and wheel chair users. DTC agreed. In addition to this OMCM asked about maintenance of the walkway and flood defence structure, both incorporation of facilities in the design and the responsibilities for it. DTC agreed that this would certainly be something that needs to be considered and that maintenance access to the walkway/flood defence should be included in the design. Whether this is regular access points or continuous access needs to be decided. OMCM suggested that in line with experience of similar defence structures elsewhere it is usually appropriate to have continuous access along the defence for maintenance. The responsibility for the maintenance of the walkway and flood defence structure is not clear yet, DTC will be responsible for the maintenance of the walkway and although the owner of the land the flood defence is on should be responsible for maintenance of the defence as well, DTC envisage that they will end up also responsible for it. DTC would want information re likely maintenance costs etc
- 3.11 OMCM asks whether DTC's requirement for the walkway will be specifically to have it on top of the embankment or will they consider alternatives. CD explained that this is not fixed, DTC has the objective for people to be able to see the views of the area when using the walkway, and they are open to how this is achieved. Consultation with the National Heritage and Wildlife Service would be required in order to explore the possibilities and in particular their impact on the designations.
- 3.12 OMCM enquires about the DTC point of view on raising the development land in the context of flood defence and whether they had any objects or concerns as this is likely to be one of the options put forward. DTC indicates that raising the land might impact on the SPA and SAC, this is their main concern and this needs to be considered adequately. DTC did point out however that they did not want to see a design that would incorporate levels that would allow for the roads to get flooded every so often but that the houses would stay dry, this was mentioned before and is not a desirable option. This issue of the recreational area was then discussed in the context of its level and could it be lower than the adjacent land. DTC were concerned that if this area including the sport pitches was left lower it could be at risk from catastrophic failure of the defences. OMCM

Action

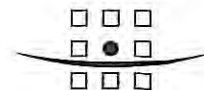
- explains that this is unlikely to be a major issue particularly if the defences are improved. The failure mechanism in this type of embankment does not result in catastrophic failure such that for example the whole of the defence would failure or disappear. What tends to happen is that breaches as noted before of a defined size will occur. These breaches of the embankment would occur due to wave overtopping, or as a result of a very weak location, where the waves would start eroding the embankment and the situation would get progressively worse increasing the breach depth and size. This would occur over a period of time and the extent and depth of flooding would progressively increase but time would be available to react. There would be little change of people being washed away. However this requires a situation where the embankment is maintained and certainly the condition of the current one would require improvements.
- 3.13 DTC explains that the provision of a good amenity for the area was one of the main aspects on which the land was permitted for development and as such they want to see that this is achieved and they don't want to see this land therefore being subjected to flooding frequently. As such a defence may be required to protect it.
- 4 Other Data & Information**
- 4.1 DTC has photographs of the February 2002 event and will provide this to RH via e-mail. OMCM indicated that he has noted some contradiction in the highest level quoted within the reports reviewed to date. One level quoted 3.6, while others quoted 3.36 and today the value of 3.65 was given. Further clarity on this would need to be sought. DTC
- 4.2 DTC explains that all five breaches in the embankment as mentioned in the KMM report occurred during the February 2002 event. Two of them occurred north of the Shore Road and three south of it.
- 4.3 DTC has no tide levels in or around Dundalk. They suggest to contact Marc O'Callaghan in Louth County Council. RH
- 4.4 DTC indicate that the OS plans will have road levels on them, they will however also check their records for other data that might be of use and will be happy to provide any of these to RH. DTC
- 4.5 DTC has level information on the existing embankment which was obtained for the KMM report of January 2003 and will provide this to RH. DTC
- 4.6 RH has a copy of two pages of a 2005 version of the KMM report and asks DTC whether they have a full copy of this newer version. DTC will chase this up with KMM. DTC
- 5. RPS (KMM) Walkway Report – January 2003**
- The list of questions as presented in the agenda was discussed. It was agreed that these would be forwarded to KMM for comment, nevertheless some details are presented below for each.
- 5.1 In relation to breaches this was covered under point 4.2 above.
- 5.2 DTC will make this topographic survey available. DTC



ROYAL HASKONING

		Action
5.3	The tide levels stems from the relationship that KMM derived in the report, but it is unclear how it was actually done and whether climate change was included in the analyses. DTC will clarify this with KMM.	DTC
5.4	DTC will make drawing showing the chainages available if it exists.	DTC
5.5	Breaches were dealt with under point 4.2 above.	
5.6	The 200 year tide level is given in the copied pages from the 2005 report. DTC will clarify the methods used to come to the tide levels with KMM. DTC not sure if tidal model used.	DTC
5.7	DTC will clarify the wave heights and how they were calculated with KMM.	DTC
5.8	This is further clarified in the copied pages of the 2005 report. DTC will see if a full copy of that report can be made available.	DTC
5.9	As stated under point 3.10 above, this needs to be taken into account.	RH/TCE
5.10	DTC will clarify / confirm this with KMM.	DTC
5.11	DTC will clarify / confirm this with KMM.	DTC
5.12	DTC will get this information from KMM.	DTC
5.13	DTC will put this question to KMM.	DTC
6.	Ground Investigation KMM – 2004	
6.1	DTC provided RH with a colour copy of the SI photographic report after the meeting.	
7.	RPS (KMM) Report Dated January 2005	
7.1	As stated under point 4.6 above DTC will get a copy of this from KMM and provide it to RH.	DTC
8.	General issues discussed at the end of the meeting	
8.1	PMcV mentions the East Dundalk Drainage Scheme which runs through the Rockview Farm project area. He expresses his concerns about the effects that raising the ground levels would have on the drainage ditches running through the Rockview Farm project area. In particular with regards to maintenance of the ditches and safety issues.	
8.2	DTC suggest that this was completed by Tobin and that they should provide RH with a copy of the East Dundalk Drainage Scheme report. The area that drains through the Rockview Farm lands extends as far back as the Xerox plant. TCE will provide a drawing showing the whole catchment that drains through the Rockview Farm project area.	TCE
8.3	PMcV is the main contact for DTC for this project.	
8.4	PMcV announces that DTC will have an external review on the report(s) produced for the Rockview Farm development. They have not yet decided who will undertake that review.	
8.5	TCE will come back to DTC on the link between engineering and environment in this project.	TCE

Appendix B2
Minutes Meeting DCMNR


ROYAL HASKONING

Minutes

HASKONING UK LTD.
COASTAL & RIVERS

Present : DCMNR: Jim Casey (JC) and Richael Duffy (RD)
Royal Haskoning: Owen McManus (OMM) and Niels Minnen (NM)

Absent :

Date : 22/03/2007

Copy : Hamish Hall (RH), John Sherwood (TFP), Sean Lambe (TFP) and Tony Curran (TCE)

Our reference : 9s6740/C/330004/Irel2

Subject : Meeting DCMNR Rockview Farm

Action

1. **Introduction**
OMM explains the purpose of the meeting.
2. **Rockview Farm Development General Overview**
OMM describes the project and the area involved and explains what the RH brief for the project encompasses.
3. **DCMNR Coastal Strategy Study (CSS)**
 - 3.1 New policies on flood risk & coast protection

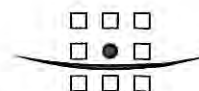
JC explains that the DCMNR CSS study looks at locations subjected to coastal erosion and tidal flooding. The study is still ongoing and that there are no usable results available yet.

JC explained that whilst they don't have any specific policies yet, that a new work package has been let recently in conjunction with the OPW to look at planning guidance. This has only just been commissioned and will consider all flood risk and not just coastal and fluvial flooding. The guidance will then be followed up by policy on the subject.

Part of the CSS study results will be flood extent maps based on LIDAR data around the whole coastline of Ireland. These maps for the area are under investigation and might come available later in the summer. DCMNR might have draft versions of the LIDAR data by the end of April.

JC advises that the impact of the developments on coastal flooding should be considered. For instance whether other areas around the site could get flooded as a result of the developments.

JC suggests that in the absence of policy and guidance documents here in Ireland at present we should look at relevant documents from



Action

the UK. This would include Development and Flood Risk Guidance for the Constructing Industry (ciria 624) as a starting point. Also PPG25 and PPS 25 which are planning guidance documents.

All flooding mechanisms need to be addressed, not only coastal and fluvial. Others are for instance failure of the infrastructure, ground water flooding, estuarine flooding and overland flooding.

JC advises to stay close to UK development and planning guidance and those of other EU countries. Especially the balance between development and flood risk should be appropriate. And flood risk protection measures will only be taken when economically and environmentally viable, if not so then other measures will have to be installed such as flood early warning systems.

Another thing to keep in mind is the quality of flood records and to check especially what the flooding mechanisms were.

JC states that the loss of storage area for water during an event is a huge problem. If a water storage area is no longer accessible for water due to development that water will go somewhere else and might cause flooding there. This issue has to be addressed within the flood risk assessment for a development within a flood risk area.

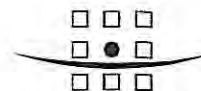
JC points out that coastal erosion is an important issue for DCMNR and should therefore be included in a flood risk assessment. Ordnance Survey Ireland has aerial photographs available for 1973 and 2000, these could be a good starting point for assessing coastal erosion at the site.

OMM replies that due to the extensive salt marsh present in front of the site erosion is not likely going to be an issue. Moreover anecdotal evidence indicates that the salt marsh is even growing in area.

JC is of the opinion that although this goes for the present day, especially with climate change ongoing that might not be the case for the future, the situation could differ over the next decades.

3.2 Required standards of protection

The flood risks mapped by DCMNR will be for a 1:200 and 1:1000 year return period, the last is called the extreme flood outline. The DCMNR policy is to on these maps show all low lying areas as flood risk areas regardless of whether defences are in place or not. It is most likely that no distinction between defended and undefended flood risk will be made.



OMM asks what the DCMNR required standard of protection (SoP) is for a new development.

JC explains that DCMNR does not have a standard SoP and is not in favour of a fixed SoP for developments. The guidance JC gives is to look at the consequences of exceeding the SoP and try to minimise these. Should try to persuade the developer into providing the highest SoP possible.

3.3 Extreme tide levels

OMM enquires about the DCMNR tide model that is being made as part of the CSS and whether any extreme tide results are available, especially for the Dundalk region. JC explains that this is part of work package 2 and 3 of the study and they will be available together with the flood maps or slightly earlier. The output of the model will consist of output points every 5 or 10 km along the whole coastline and will provide extreme tide levels.

OMM asks whether there would be any early draft results available to use as a comparison to the RH work. JC tells that these might be available in 6-8 weeks time.

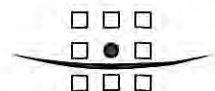
3.4 Climate change recommendations, i.e. sea level rise

Climate change is to be included in the guidelines under development. No definitive choices have been made yet as to how to include climate change, what scenarios to use and so on. JC points out two reports, one on the implications of EU climate change regulations for Ireland by Sweeney and the IPCC summarising report for policy makers. The DCMNR focus with regards to climate change will be on four issues: sea level rise, increase in high sea level frequency, increase in storm surge magnitude and habitat impacts. The message with regards to climate change is to be aware of the uncertainty in the analyses and to apply flexibility and adaptability in designs. And to be aware of the life cycle of a project.

The DCMNR don't recommend any specific value to be adopted but suggest review of these documents.

3.5 Planning guidance for option development, i.e. raise ground levels V traditional flood defences

This issues was covered in general in the discussion relating to points 3.1 to 3.4. Main issue is impacts need to be addressed, i.e. impacts of raising defence levels on other areas, impacts of raised ground levels on hydrology / storage etc



4 Other Data & Information

4.1 Historic flood risk records

OMM asks whether DCMNR would have any historic flood risk levels in the area. JC says that DCMNR does not have these, but RPS might have some information. RH can contact them if required.

RH

4.2 Tide level data & tide gauges

OMM asks whether DCMNR would have any tide level data available from their modelling work. JC says that these are with RPS and that Bjoern Elsaesser in Belfast would be the person to contact. He also points out the UK tide gauge network with gauges at Warrenpoint, Drogheda, Bangor and Port Erin.

RH

OMM asks about a tide gauge in Port Oriel. JC says that Donal Clarke in Louth County Council would be the person to talk to about this.

JC advises to contact hydrographic surveyors and names Barry Jenkins and Jim Walsh as contact persons.

JC pointed out that a report on the national tide gauge network exists, describing all the gauges, and give RH an extract for Drogheda. Information on the Irish Tides web site was also provided.

4.3 Wave climate data

OMcM asks whether DCMNR has a wave model constructed as well. JC explains that from a 200 year return period and higher wave action can be a significant part of the joint probability. Therefore DCMNR will have a wave model constructed, which is done by RPS, but not available yet.

JC points out reports from HMRC, a wave climate atlas and a wind atlas, written for the location optimisation of wave energy devices and wind farms respectively. Tony Lewis and Jimmy Murphy would be the contact here. Another contact is Jeffrey O'Sullivan from the Marine Institute.

4.4 Wind data

See 4.3.

4.5 Bathymetric data

JC tells that DCMNR has some spot levels around the whole coast for the purpose of checking the LIDAR data. There will probably be some levels on either side of the embankment, but there will be very few.

4.6 Topographic data

JC tells that the LIDAR data should be available by Easter time, but that it needs validation. Before validation it will have an accuracy of +/- 0.5m for sure, but it is expected to have an accuracy of +/-0.2m.

Also there are oblique aerial images of the area from September 2003.

4.7 Other data

JC advises to look at the UK planning guidance PPG25 and PPS25 as their own guidance will be at least partially based in these.

Appendix C
Embankment Inspection Photographs and Sections



Photo 1
Front face of the embankment



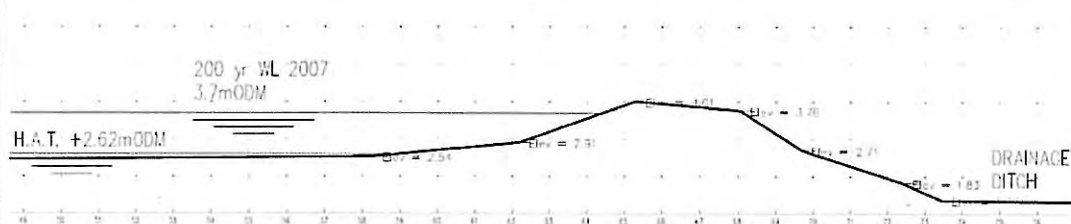
Photo 2
Crest of the embankment looking South



Photo 3
Back face of the embankment



Photo 4
Crest of the embankment looking North



Cross-section 1 through the embankment at chainage 85

Chainage 102. Typical wider part of the embankment in Section 1

Figure C1. Description of the embankment at chainage 102



Photo 5
Front face of the embankment



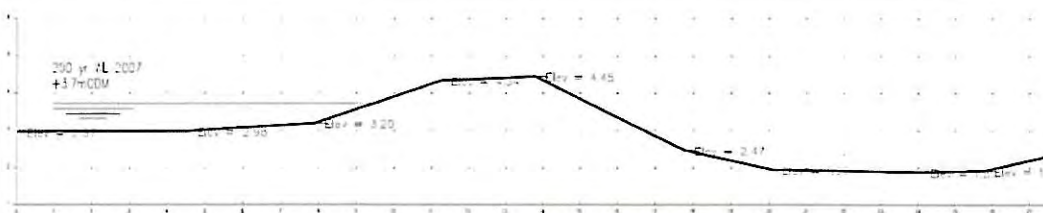
Photo 6
Crest of the embankment



Photo 7
Back face of the embankment



Photo 8
Salt marsh in front of the embankment



Cross-section 2 through the embankment at chainage 585

Chainage 550. Typical narrower part of the embankment in Section 1

Figure C2. Description of the embankment at chainage 550



Photo 9 – chainage 0
Access gap at the bottom of Shore Road



Photo 10 – chainage 47
Steep toe front face of the embankment



Photo 11 – chainage 478
Animal hole through the embankment



Photo 12 – chainage 218
Location of possible breach



Photo 13 – chainage 408
Vulnerable spot



Photo 14 – chainage 572
Low spot on the embankment

Figure C3. Various photos section 1



Photo 15
Front face of the embankment



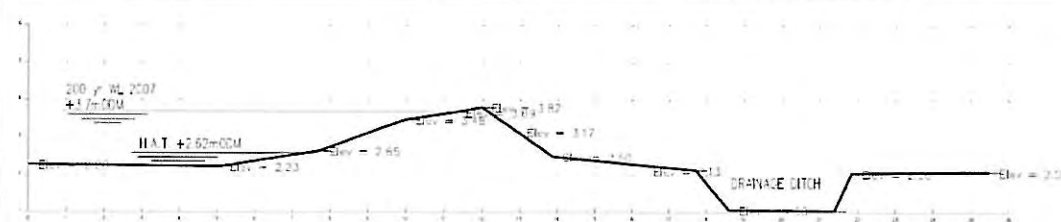
Photo 16
Crest of the embankment



Photo 17
Back face of the embankment



Photo 18
Salt marsh in front of the embankment



Cross-section 3 through the embankment at chainage 985

Chainage 0975. Typical part of the embankment in Section 2

Figure C4. Description of the embankment at chainage 975



Photo 19
Front face of the embankment



Photo 20
Crest of the embankment

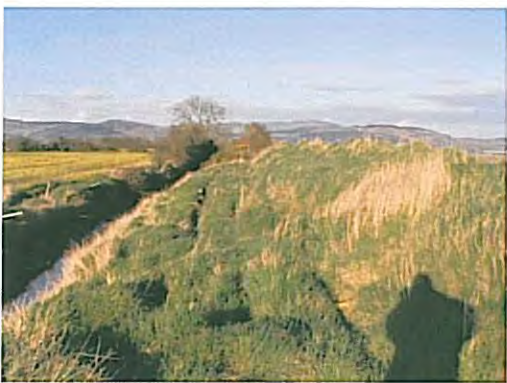
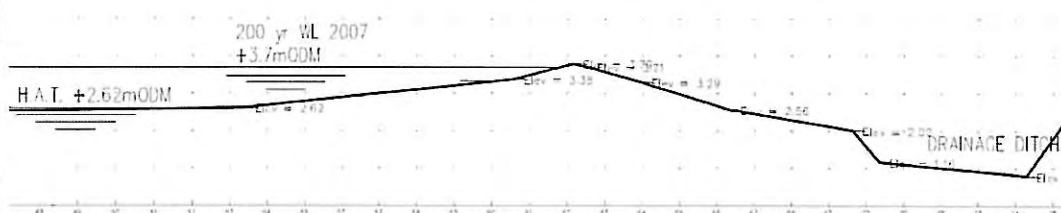


Photo 21
Back face of the embankment



Photo 22
Hole in the back face of the embankment



Cross-section 4 through the embankment at chainage 1279

Chainage 1263. Typical part of the embankment in Section 2

Figure C5. Description of the embankment at chainage 1263



Photo 23 – chainage 835
Steep toe back face



Photo 24 – chainage 653
Pedestrian path crossing the embankment



Photo 25 – chainage 785
Path crossing over the embankment



Photo 26 – chainage 932
Weak spot in the embankment front



Photo 27 – chainage 932
Weak spot in the embankment back



Photo 28 – chainage 1058
Vulnerable section of embankment

Figure C6. Various photos section 2



Photo 29 – chainage 1093
Steep toe back face



Photo 30 – chainage 1093
Pedestrian path crossing the embankment



Photo 31 – chainage 1211
End of the grassy ridge



Photo 32 – chainage 1275
Front of embankment at outfalls



Photo 33 – chainage 0932
Crest of embankment at outfalls



Photo 34 – chainage 1275
Back of embankment at outfalls

Figure C7. Various photos section 2



Photo 35
Front face of the embankment



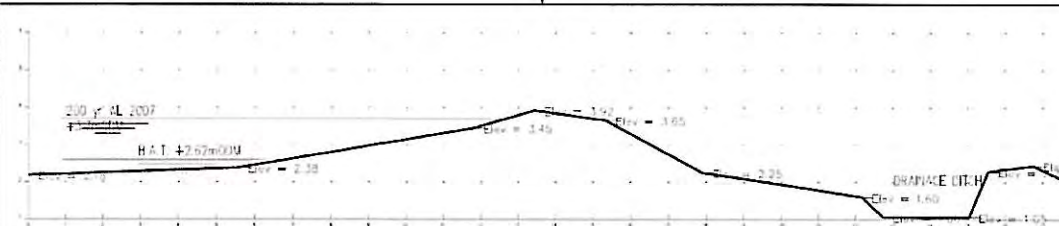
Photo 36
Crest of the embankment



Photo 37
Back face of the embankment



Photo 38
Vertical toe back face of the embankment



Cross-section 5 through the embankment at chainage 1585

Chainage 1605. Typical part of the embankment in Section 3

Figure C8. Description of the embankment at chainage 1605



Photo 39 – chainage 1327
Slipped look back face



Photo 40 – chainage 1402
Front face dip and much debris



Photo 41 – chainage 1450
Widest part of the embankment



Photo 42 – chainage 1500
Front face of embankment changes



Photo 43 – chainage 1680
Start of vegetation back face



Photo 44 – chainage 1680
Change in shape of the embankment

Figure C9. Various photos section 3



Photo 45
Front face of the embankment



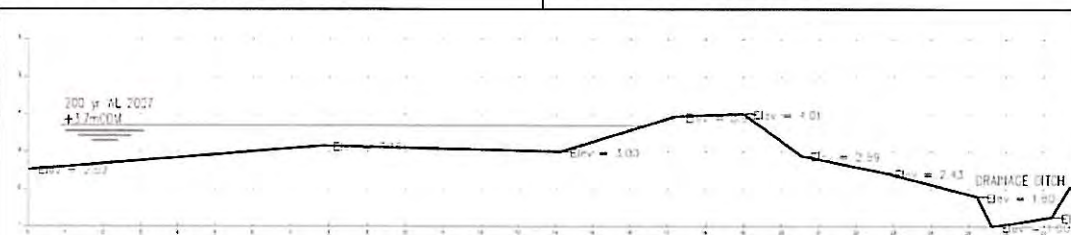
Photo 46
Crest of the embankment



Photo 47
Back face of the embankment



Photo 48
Ridge at toe front face



Cross-section 6 through the embankment at chainage 1795

Chainage 1807. Typical part of the embankment in Section 4

Figure C10. Description of the embankment at chainage 1807



Photo 49 – chainage 1717
Uneven crest bad condition and grass
berm in distance



Photo 50 – chainage 1746
Start of heavy vegetation



Photo 51 – chainage 1875
Larger berm in front with rock toe



Photo 52 – chainage 1937
Berm and front face growing together



Photo 53 – chainage 1975
Rock toe and berm disappearing



Photo 54 – chainage 1975
No more berm from back face toe to ditch

Figure C11. Various photos section 4



Photo 55
Front face of the embankment



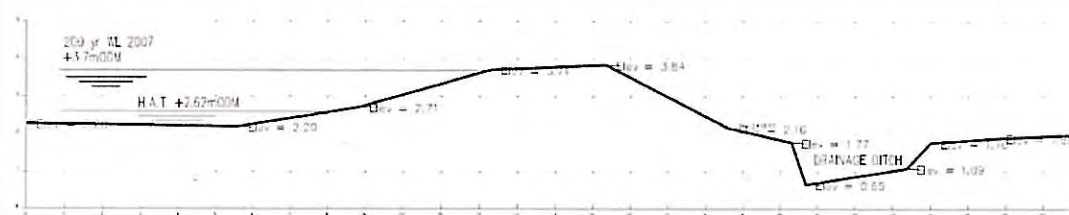
Photo 56
End of the embankment and toe



Photo 57
Back face of the embankment



Photo 58
Salt marsh in front of the embankment



Cross-section 7 through the embankment at chainage 1991

Chainage 2065. Typical part of the embankment in Section 4

Figure C12. Description of the embankment at chainage 2065



Photo 59
Front face of the embankment



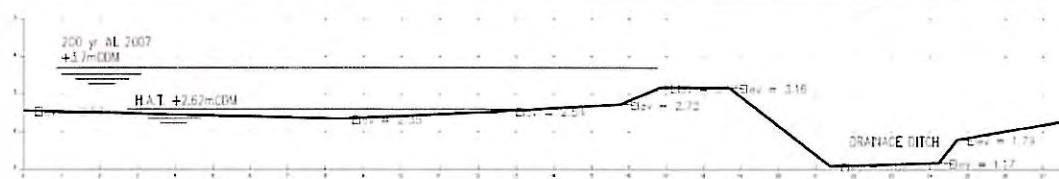
Photo 60
Crest of the embankment



Photo 61
Back face of the embankment



Photo 62
Hinterland behind the embankment



Cross-section 8 through the embankment at chainage 2188

Chainage 2201. Typical part of the embankment in Section 5

Figure C13. Description of the embankment at chainage 2201



Photo 63 – chainage 2302
Embankment around corner



Photo 64 – chainage 2346
Trench ending against embankment



Photo 65 – chainage 2408
Gap in embankment



Photo 66 – chainage 2475
Weak spot on embankment



Photo 67 – chainage 2475
Masonry end of embankment



Photo 68 – chainage 2475
Last stretch of embankment

Figure C14. Various photos section 5



Photo 69 – chainage 0077
Ditch around start inspection



Photo 70 – chainage 0572
Blockage of the ditch



Photo 71 – chainage 0653
Ditch with heavy vegetation



Photo 72 – chainage 1211
Typical section of ditch



Photo 73 – chainage 1275
Ditch at outfalls



Photo 74 – chainage 1275
inlet structure outfalls

Figure C15. Various photos drainage ditch



Photo 75 – chainage 1300
Typical section drainage ditch



Photo 76 – chainage 1380
Form of cloth in ditch



Photo 77 – chainage 1380
Form of cloth in ditch



Photo 78 – chainage 1680
Perpendicular ditch joining and bridge



Photo 79 – chainage 1975
Typical section drainage ditch



Photo 80 – chainage 2105
Typical section drainage ditch

Figure C15. Various photos drainage ditch

Appendix D

Tidal Analyses

Appendix D1

Tide Relationships

Tide relationship between Dublin and Dundalk based on the Admiralty Tide Tables

Table D-1 Spring tide levels Admiralty tide tables

	Dublin		Dundalk	
	Chart datum	mODM	Chart datum	mODM
Mean High Water Springs (MHWS)	4.1	1.59	5.1	2.10
Mean High Water Neaps (MHWN)	3.4	0.89	4.2	1.20
Mean Low Water Neaps (MLWN)	1.5	-1.01	1.6	-1.40
Mean Low Water Springs (MLWS)	0.7	-1.81	0.6	-2.40
HAT	4.5	1.99		2.64

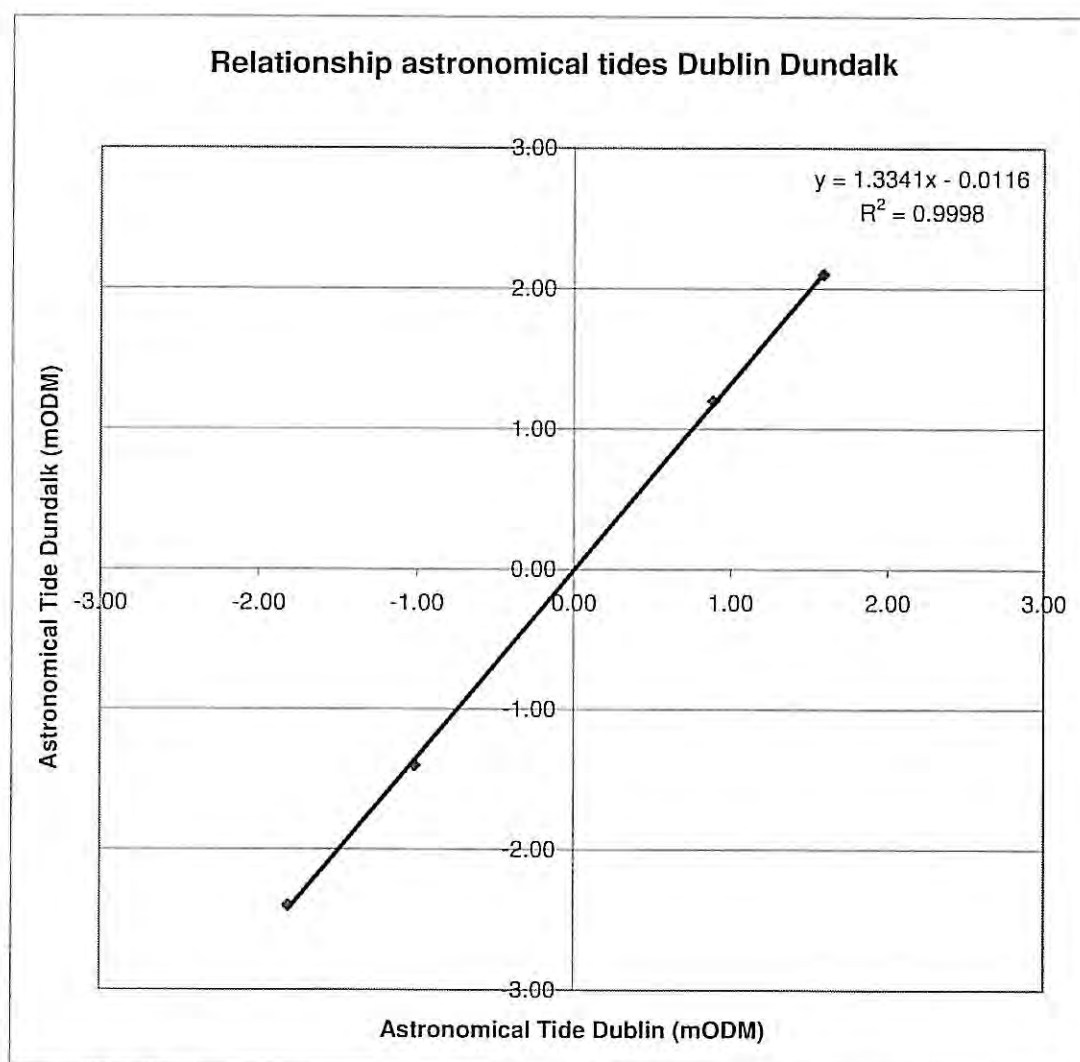


Figure D-1 Relationship astronomical tide levels Admiralty tide tables

Tide relationship between Dublin and Dundalk based on measured data 17-19 May 1992

Table D-2 Measured tide levels May 1992

Date	Time	Dublin		Dundalk	
		Chart datum	mODM	Chart datum	mODM
17-May	07:00	0.6	-1.91	1.05	-1.95
17-May	09:00	1.8	-0.71	2.78	-0.22
17-May	11:00	3.1	0.59	4.4	1.4
17-May	12:45	3.8	1.29	4.9	1.9
17-May	14:30	3.1	0.59	4.13	1.13
17-May	16:30	1.5	-1.01	1.92	-1.08
17-May	18:30	0.6	-1.91	0.83	-2.17
18-May	07:30	0.7	-1.81	0.93	-2.07
18-May	09:30	1.9	-0.61	2.6	-0.4
18-May	11:30	3	0.49	4.27	1.27
18-May	13:15	3.8	1.29	4.9	1.9
18-May	15:00	3.4	0.89	4.23	1.23
18-May	17:00	1.6	-0.91	2.12	-0.88
18-May	19:00	0.8	-1.71	0.82	-2.18
19-May	08:00	0.9	-1.61	0.8	-2.2
19-May	10:00	1.6	-0.91	2.4	-0.6
19-May	12:00	3	0.49	4.1	1.1
19-May	14:00	3.6	1.09	4.8	1.8
19-May	16:00	3.2	0.69	3.95	0.95
19-May	18:00	1.6	-0.91	1.88	-1.12
19-May	20:00	0.8	-1.71	1	-2

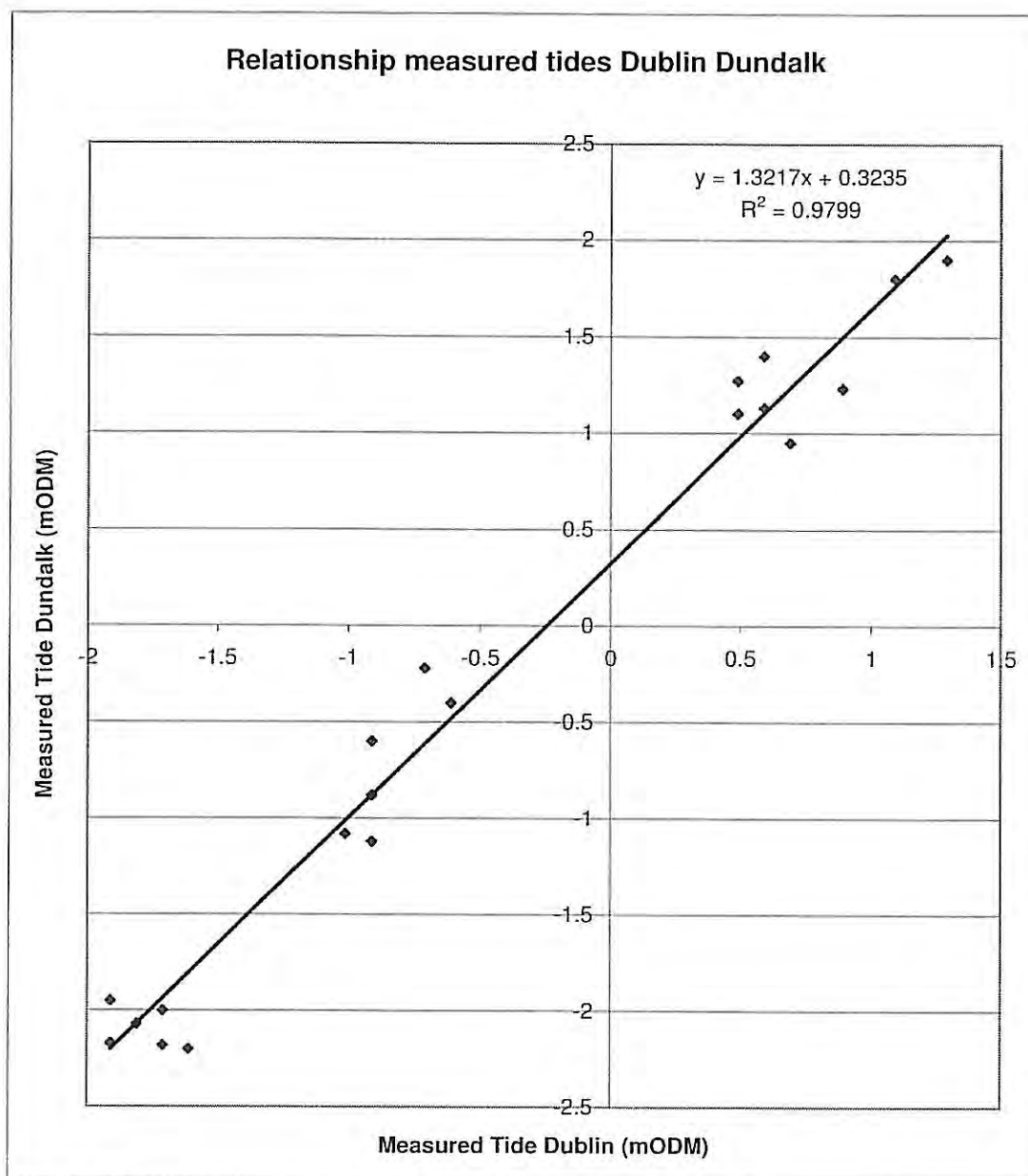


Figure D-2 Relationship measured tide levels May 1992

Tide relationship between Dublin and Dundalk based on RPS tide data analyses

Table D-3 Spring tide levels RPS

	Dublin		Dundalk	
	MSL	mODM	MSL	mODM
Mean High Water Springs (MHWS)	1.69	1.59	2.19	2.1
Mean High Water Neaps (MHWN)	0.99	0.89	1.29	1.2
Mean Low Water Neaps (MLWN)	-0.91	-1.01	-1.21	-1.3
Mean Low Water Springs MLWS)	-1.71	-1.81	-2.2	-2.29
HAT	2.09	1.99	2.71	2.62

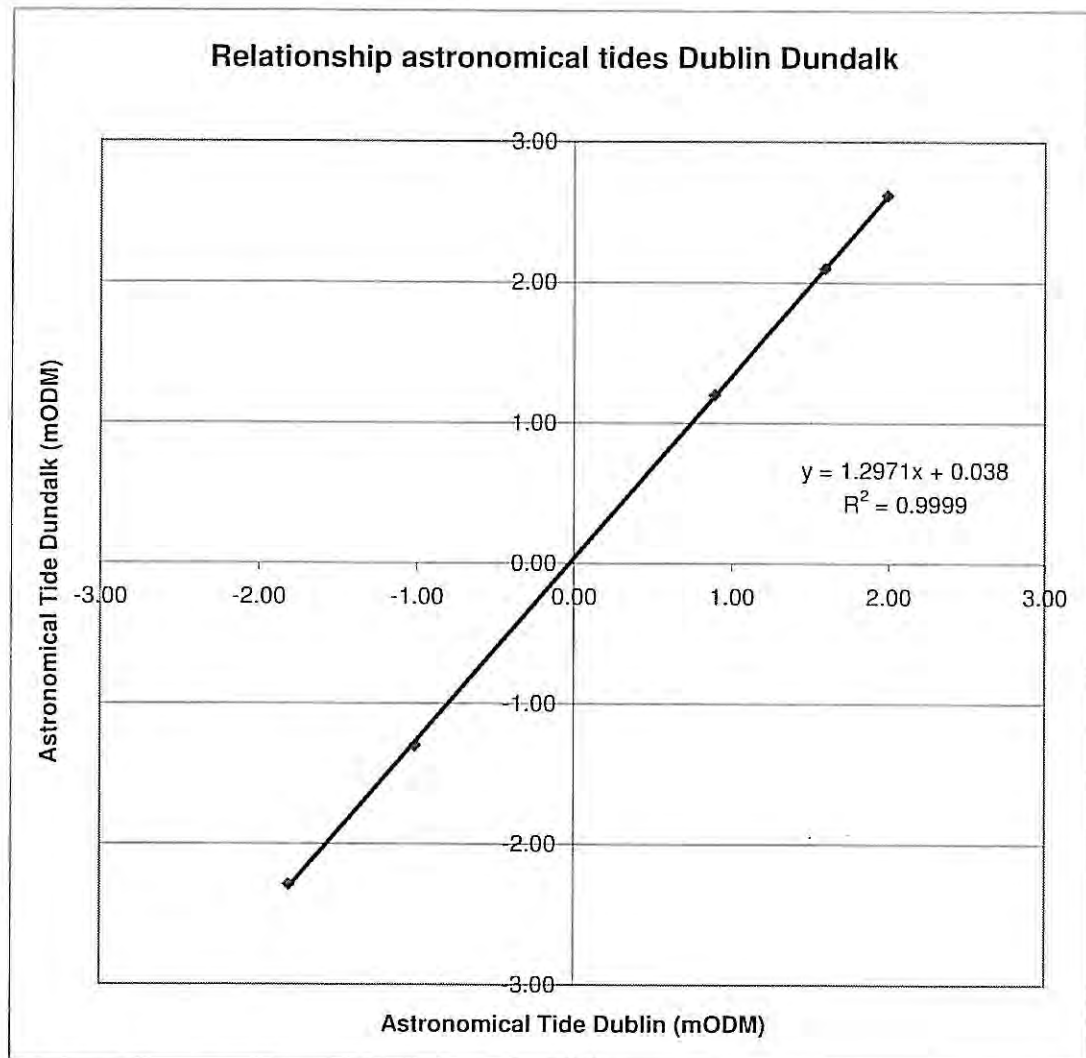


Figure D-3 Relationship astronomical tides RPS levels

Appendix D2
RPS Water Level Assessment Dundalk



Water Level Assessment for Dundalk

DOCUMENT CONTROL SHEET

Client	Haskoning Ireland Ltd					
Project Title	Water Level Assessment for Dundalk					
Document Title	Water Level Assessment for Dundalk					
Document No.	IBE0102/R01/NS_DRAFT					
This Document Comprises	DCS	TOC	Text	List of Tables	List of Figures	No. of Appendices
	1	1	10	1	1	0

Rev.	Status	Author(s)	Reviewed By	Approved By	Office of Origin	Issue Date
0	Draft	NS	BE		Belfast	May 2007

TABLE OF CONTENTS

1	INTRODUCTION	1
2	TIDAL SURGE MODEL FOR IRELAND	2
3	ASSESSMENT OF WATER LEVELS FOR DUNDALK.....	6
4	SUMMARY	10

LIST OF FIGURES

Figure 2.1: Extent of Tidal Surge Model for Ireland	2
Figure 2.2: Validation of Tidal Surge Model – Surge Residual Dublin Feb 2002.....	4
Figure 2.3: Validation of Tidal Surge Model – Surface Elevation Dublin Feb 2002	5
Figure 3.1: Simulated Storm Surge Residual Dundalk Feb 2002	6
Figure 3.2: Correlation of Surge Residuals for Dublin & Dundalk.....	7
Figure 3.3: Surge Residual Illustrating Seiching in Dublin Bay	9

LIST OF TABLES

Table 3.1: Derived water levels at Dundalk.....	6
Table 3.2: Conversion from MSL to OD Malin <i>information only</i>	8

1 INTRODUCTION

RPS Consulting Engineers were commissioned by Haskoning Ireland Ltd to assess the water levels at Dundalk for a range of tide and storm surge conditions, since at present no adequate recorded data is available for undertaking an extreme value analysis. RPS Consulting Engineers' tide and storm surge model for Ireland was used to derive scale factors between Dublin and Dundalk water levels. These water levels originate from calibrated storm surge simulations of the past 25 years. They were used to assess the water levels in Dundalk for the required conditions. This report provides details of the tidal model in addition to the water level assessment.

2 TIDAL SURGE MODEL FOR IRELAND

The RPS Consulting Engineers' tidal surge model for Ireland stretches from the Northwestern end of France including the English Channel to Dover to 16° West into the Atlantic, including the Porcupine Bank and Rockall. In the south it reaches from the Northern part of the Bay of Biscay to just south of the Faroes Banks in the North. Overall the model covers the Northern Atlantic Ocean up to a distance of 600km from the Irish Coast as illustrated in Figure 2.1.

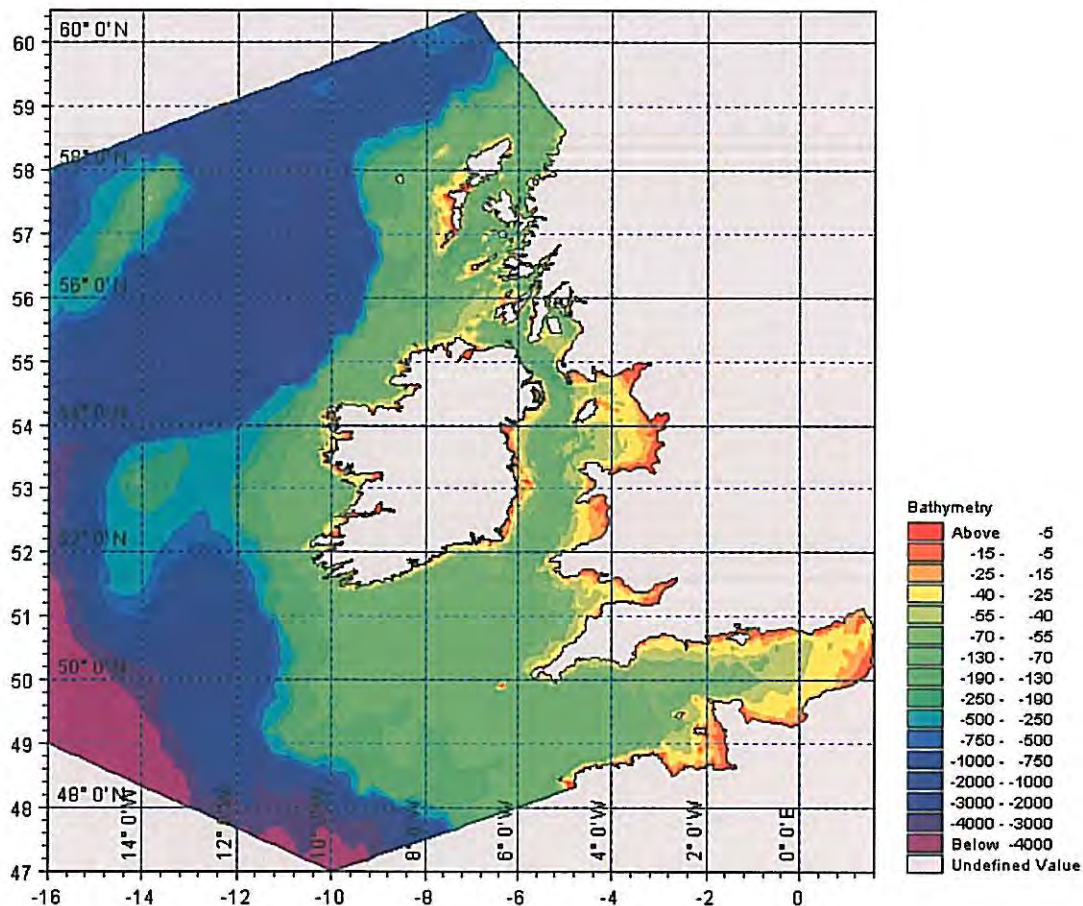


Figure 2.1: Extent of Tidal Surge Model for Ireland

The model is based on the solution of the depth-integrated incompressible Reynold's averaged Navier-Stokes equations using a cell-centred finite volume method. The model was constructed using unstructured flexible mesh technology allowing the size of the computational cells to vary depending on user requirements. Along the Atlantic boundary the model features a mesh size of 13.125' (24km). The Irish Atlantic coast has been discretised using cells of on average 3km size. In the Irish Sea, which is the area of greatest interest to this study, the maximum cell size is limited to 3.5 km decreasing to 200m along most of the Irish coastline.

The bathymetry was generated using a number of different sources. Large parts of the bathymetry information were obtained from Admiralty Charts, as produced digitally by C-MAP of Norway. Surveys of several banks and coastal areas have also been included covering in parts or all of

- | | |
|---|-------------------------------------|
| ☐ Wexford & approaches | ☐ Dublin Bay |
| ☐ Blackwater bank | ☐ Malahide |
| ☐ Arklow bank | ☐ Rogerstown |
| ☐ Codling bank | ☐ Greystones |
| ☐ Carlingford Lough | |

Both survey data commissioned by RPS Consulting Engineers and the digitised charts were quality checked by RPS engineers and compared with Admiralty data and known benchmarks. Recent surveys of Dublin Bay and adjacent areas carried out by Geological Survey Ireland (GSI) were also incorporated. Data from surveys carried out by GSI to the West of Ireland as part of the Irish National Seabed Survey (INSS) are also included in the model. Finally the datum of the bathymetry data was reduced to mean sea level using over 350 reference levels. A custom made routine interpolated mean sea level corrections for the relevant area and adjusted the bathymetry accordingly.

The simulation of the astronomic tides in the model area is mainly driven by the oscillation of water levels along the open boundaries. The tidal surge model has 6 open boundaries, 5 in the Atlantic and one in the English Channel. The time series of tidal elevation along these boundaries were generated using a global tidal model designed by a team at the Danish National Survey and Cadastre Department (KMS). The KMS global tidal model is based on the prediction of tidal elevations using 8 semidiurnal and diurnal tidal constants (as opposed to UKHO which uses 4-6 constants). These constants were derived through the simulation of the effect of astronomic forces due to the sun and moon on the water surfaces. The model output was further refined with the use of satellite derived altimetry data. These harmonics were complimented with additional data from GLOSS and PSMSL from the British Oceanographic Data Centre. Thus, for the simulation of the actual storm surges, seasonal and monthly components have been included; giving rise to a model which is driven by 15 harmonic constants on the open boundaries.

For the simulation of surge events, two different data sources were used to provide meteorological boundary conditions, both obtained from the European Centre of Medium Range Weather Forecasts (ECMWF). The parameters applied to generate surge within the model are mean sea level atmospheric pressure and the 10 minute averaged 10m wind speeds (u and v component).

ECMWF have been operating an atmospheric model with analysis running at 6 hourly intervals and 0.5° resolution since 1991. Furthermore in 2003 a re-analysis project was completed, which included the simulation of the meteorological conditions since 1957 at 6 hourly intervals and 1.125° resolution (ERA 40). In order to improve the model prediction additional wind and pressure data were acquired from the ECMWF. These data sets are taken from various forecast simulations and were combined into a single data set which covered each 24 hour period with 8 time steps, providing sufficient information on water level variations to simulate the development and progression of the storm surges.

The tidal surge model was validated against measured data mostly from UK NTSLF tide gauges. An example of the surge model simulation compared with measured surge residuals is shown in Figure 2.2; where the surge residual extracted by RPS from the measured water level is shown by the blue trace and the red trace indicates the simulated value at the same location in Dublin Bay. Figure 2.3 shows the comparison of surface elevation between the simulation (red trace) and measured values (blue trace) within Dublin Bay for the same period.

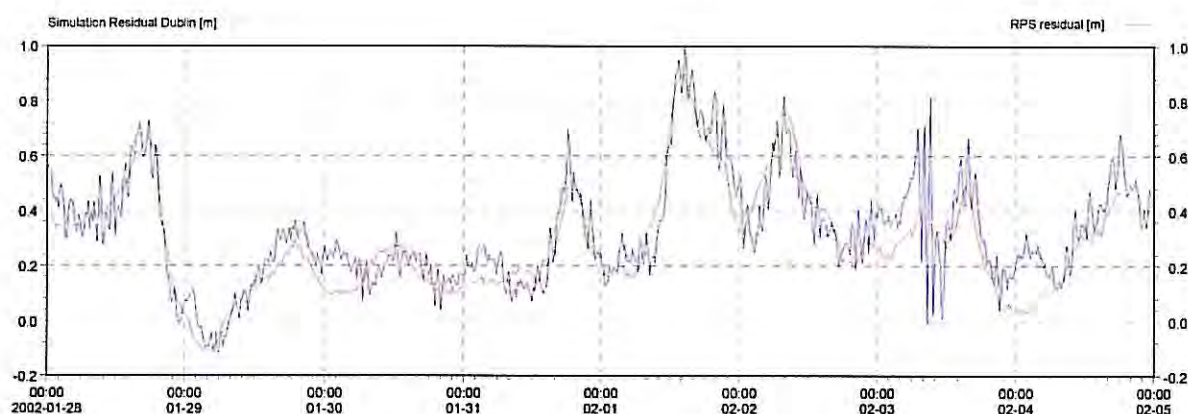


Figure 2.2: Validation of Tidal Surge Model – Surge Residual Dublin Feb 2002

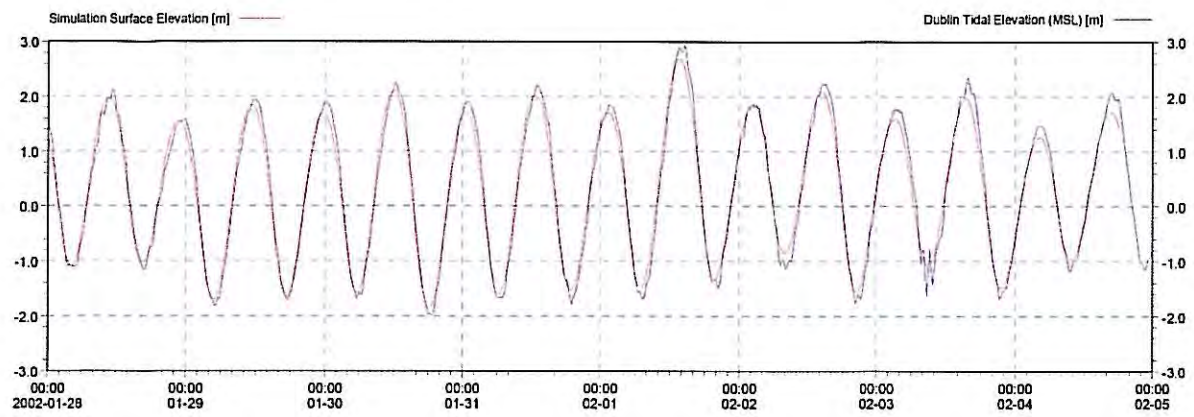


Figure 2.3: Validation of Tidal Surge Model – Surface Elevation Dublin Feb 2002

3 ASSESSMENT OF WATER LEVELS FOR DUNDALK

The tidal surge model, outlined in the section above, was used to derive a relationship between water levels at Dublin and Dundalk. Based on our tidal model, the values for the astronomic tides were determined and these are given in Table 3.1.

	Dublin	Dundalk Bay
HAT (m to MSL)	2.09	2.71
MHWS (m to MSL)	1.69	2.19
MHWN (m to MSL)	0.99	1.29
MLWN (m to MSL)	-0.91	-1.21
MLWS (m to MSL)	-1.71	-2.20
0.02 year RTP (m to MSL)	1.88	2.44
MSL* to CD	2.41	2.85

* Note that the mean sea level conversion is an estimated value; but is required for conversion from OD Poolbeg.

Table 3.1: Derived water levels at Dundalk

The tidal surge model for Ireland was used to undertake a number of storm surge simulations; using the tidal and surge model and applying meteorological data from the ECMWF operational surface analysis and forecast in addition to the tidal conditions for the relevant period. The following analysis is based on 25 storm simulations with surge values above 0.5 metre covering the period 1990 to 2005, whereby the surge residual was extracted from the surge simulation for both the entrance to Dublin Harbour and Dundalk Bay. An example is given in Figure 3.1 which shows the simulated surge residual for Dundalk during February 2002 (Figure 2.2 and Figure 2.3 showed the validation plots for Dublin at this time).

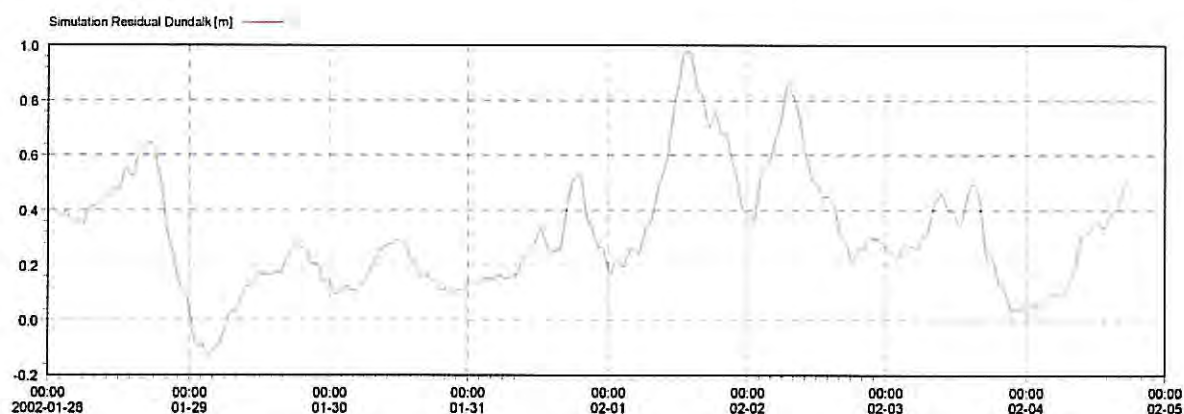


Figure 3.1: Simulated Storm Surge Residual Dundalk Feb 2002

The correlation between the different storms is illustrated in Figure 3.2. Using linear regression, a factor of 1.1 was estimated for surge water levels in Dundalk Bay relative to Dublin Harbour. It has to be noted, that the scatter in the surge residual diagram is greater above the diagram (for Dundalk) compared to Dublin.

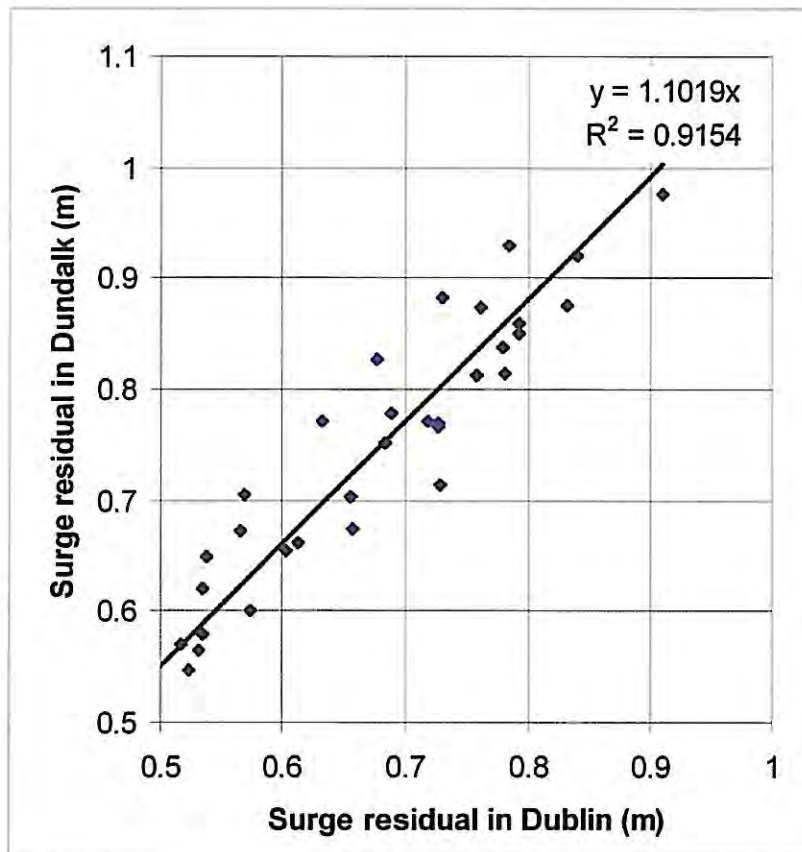


Figure 3.2: Correlation of Surge Residuals for Dublin & Dundalk

Based on an assumed surge residual water level in Dublin Bay of 1.17 metre (joint return period 1:200, surge RP 1:140), the 1 in 200 year water level at Dundalk is assumed to comprise of a surge water level of 1.29 metre and a tidal level of 2.44 metre. These combine to give 3.73 metre above MSL. The conversion from MSL to OD Malin is outlined in Table 3.2 for your information. RPS Consulting Engineers are currently in discussion with OSI as theoretically OD Malin (being mean sea level at Malin) should also be MSL at Dundalk. However, this is not the case, since the isogravity potential going through OD Malin at Malin is not identical with the OD Malin geoid. Therefore the following conversion is given for guidance purposes only.

	Level in metres
1 in 200 water level Dundalk Bay to MSL	3.73
MSL to CD	2.85
CD to OD Poolbeg ⁺	-0.30
ODP to OD Malin [#]	- 2.64
1 in 200 water level Dundalk Bay to ODM	3.64

⁺ Source: Admiralty Tide Tables 2007, Table IV

[#] Source: OSI Control Section, RPS communication

Table 3.2: Conversion from MSL to OD Malin *information only*

During the analysis of Dublin North Wall tidal data it became apparent that the still water levels at the site are affected by seicheing in the Bay. The surge modelling did not indicate any seicheing effects due to the nature of the meteorological conditions; which were defined at 3 hourly intervals. In order to test if fluctuations in wind speed and direction would cause seicheing in Dublin Bay, a set of meteorological conditions was altered in the following manner: The average wind speed was interpolated to 30 minute intervals and a spatial resolution of $1/4^{\circ}$. This average wind speed was overlaid with a gamma distributed variation in wind speed, depending on the magnitude of the average wind. As a result, a wind field for the model was generated, which had the same average characteristics as the 3 hourly dataset.

Figure 3.3 shows the comparison of surge residual for Dublin Bay with average wind (blue trace) and with the influence of gusts (red trace). The surge residual using the 3 hourly wind data resulted in a maximum elevation of 0.94, whereas with the 30 minute wind data a maximum elevation of 1.04 was simulated.

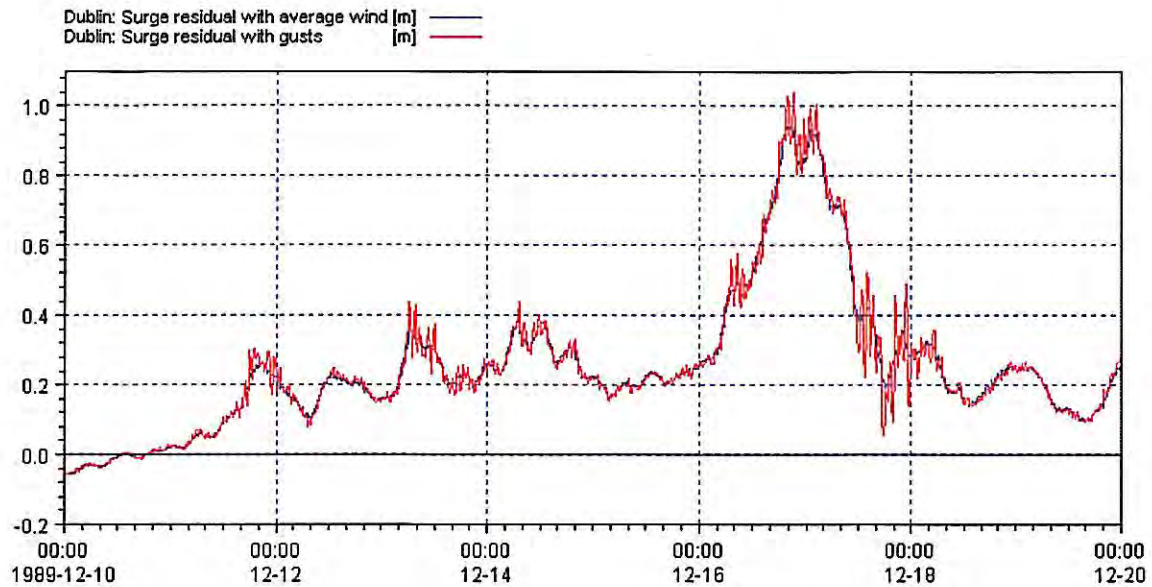


Figure 3.3: Surge Residual Illustrating Seiching in Dublin Bay

Using the results of these simulations it was possible to compare the differences in surge water level caused by varying the wind speeds on a sub-hourly basis in the entire model domain. From this analysis, it was concluded that in addition to the extreme water levels simulated by the model, an additional 100-150mm needs to be added to the water levels in the inner part of Dublin Bay to account for the effects of seiching. This is also evident from the calibration data shown in Figure 2.2, where the model is able to predict the surge reasonably well, however the overlaying seiching is not reproduced.

4 SUMMARY

The RPS Consulting Engineers tidal surge model for Ireland was used to derive a relationship between water levels at Dublin Bay and Dundalk for both astronomical tides and storm surge events. Based on these relationships the astronomic tide levels for Dundalk were assessed and the 1 in 200 year water level for Dundalk Bay was determined to be 3.64m OD Malin.

Appendix E

Saltmarsh Evolution



Figure E-1 Extent of the saltmarsh in 1974



Figure E-2 Extent of the saltmarsh in 1995



Figure E-3 Extent of the saltmarsh in 2004

Appendix F
Option Drawings

- NOTES
1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
 2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MSLM HERE.



CONCRETE CULVERT WITH
NO-RETURN FLAP VALVE

RESIDENTIAL AREA +3.0mODM

AMENITY AREA +3.0mODM

REV	DATE	DESCRIPTION	BY	CHK	APP

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

EMBAKMENT OVERVIEW
OPTION 1 - EMBAKMENT 2

ROYAL HASKONING

18 LOWER BACCHETT STREET
DUBLIN 3
01 855 8154
01 855 8154
haskoning@royalhaskoning.com
www.royalhaskoning.com

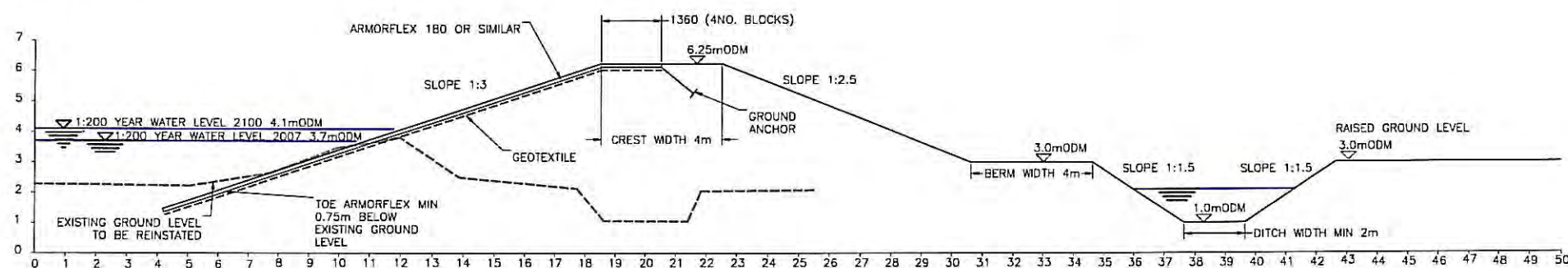
DATE: 02-06-07

SCALE: AS SHOWN

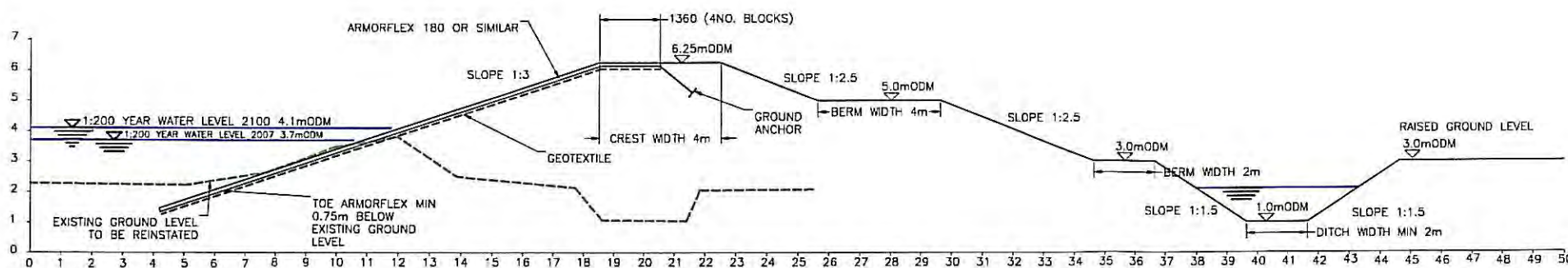
FIGURE NO. F1

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION A
OPTION 1
EMBANKMENT 1
CHAINAGE 985



SECTION A
OPTION 1
EMBANKMENT 2
CHAINAGE 985



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

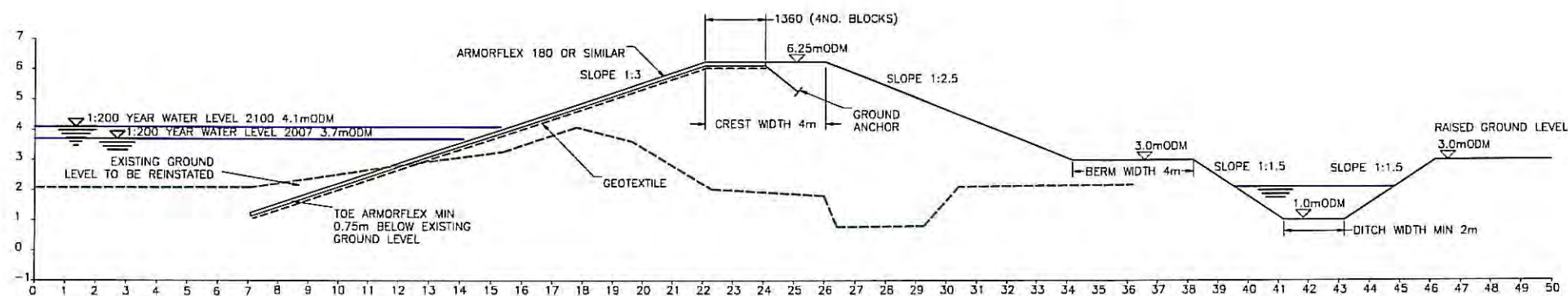
ROCKVIEW FARM
FLOOD RISK ASSESSMENT

MAIN COASTAL EMBANKMENT
CROSS-SECTION A, OPTION 1
EMBANKMENT OPTIONS 1 & 2

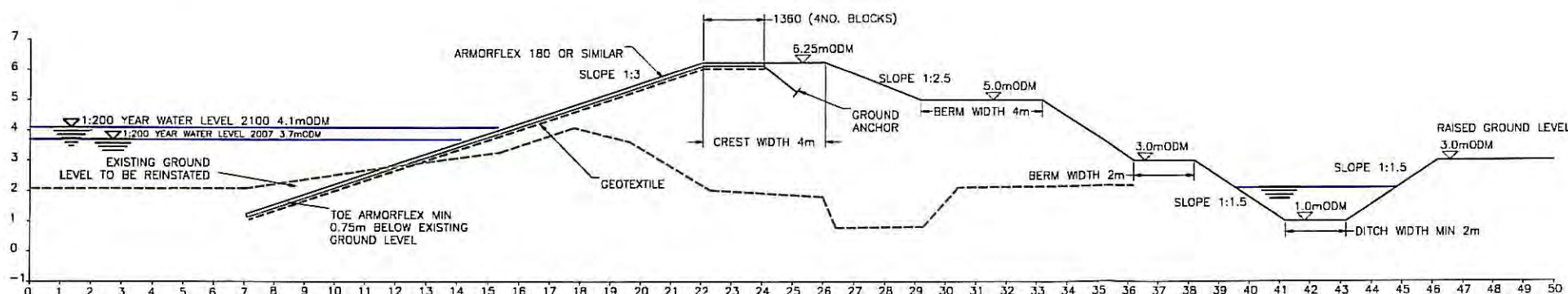
ROYAL HASKONING 18 LOWER BAGINOT STREET DUBLIN 2 01 353 6704333 01 353 6704335 a.haskoning@royalhaskoning.com www.royalhaskoning.com		
DRAWN GG	CHECKED NM	PASSED
DATE 17-05-07	CLIENT'S REF.	
SCALE AT A1	AUTOCAD REF. 9S4067AO-F2	
DRAWING No.	REVISION	
FIGURE F2		

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION B
OPTION 1
EMBANKMENT 1
CHAINAGE 1385



SECTION B
OPTION 1
EMBANKMENT 2
CHAINAGE 1385



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

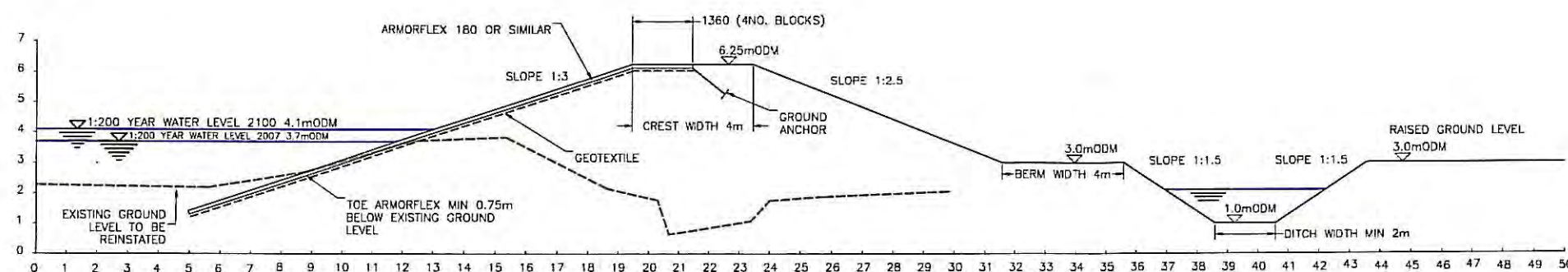
ROCKVIEW FARM
FLOOD RISK ASSESSMENT

MAIN COASTAL EMBANKMENT
CROSS-SECTION B, OPTION 1
EMBANKMENT OPTIONS 1 & 2

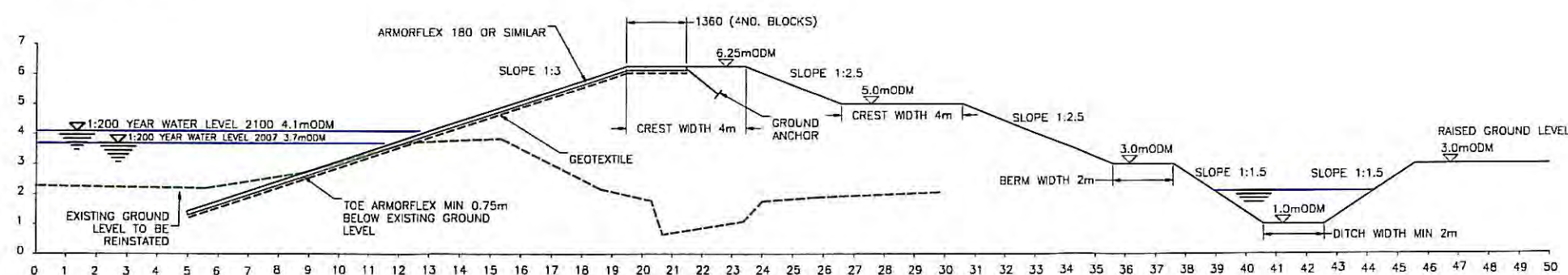
ROYAL HASKONING 18 LOWER BAGGOT STREET, DUBLIN 3 01 553 6710/4533 Fax 01 553 6710/4533 Email info@royalhaskoning.com		
DRAWN GG DATE 17-05-07 SCALE AT A1	CHECKED NM CLIENT'S REF. AUTOCAD REF 9S4067A0-F3	PASSED REVISION
FIGURE F3		

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION C
OPTION 1
EMBANKMENT 1
CHAINAGE 1991



SECTION C
OPTION 1
EMBANKMENT 2
CHAINAGE 1991



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE

MAIN COASTAL EMBANKMENT
CROSS-SECTION C, OPTION 1
EMBANKMENT OPTIONS 1 & 2

ROYAL HASKONING 18 LOWER BACCHY STREET DUBLIN 2 01 232 0123 (L) 01 232 0123 (F) 01 232 0123 (M) www.royalhaskoning.com		
DRAWN GG	CHECKED NM	PASSED
DATE 17-05-07	CLIENT'S REF.	
SCALE AT A1	AUTOCAD REF. 954067A0-F4	
DRAWING No.	REVISION	
FIGURE F4		

END OF EMBANKMENT TO BE
LANDSCAPED IN WITH SURROUNDING
GROUND LEVELS

RESIDENTIAL AREA +4.25mODM

AMENITY AREA +3.5mODM

NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORENNICE DATUM MAIN HEAD.

REV	DATE	DESCRIPTION	BY	CHK	APP

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

EMBAKMENT OVERVIEW
OPTION 2 - EMBANKMENT 2

ROYAL HASKONING
19 LOWER BACCHY STREET DUBLIN 2
01 261 8140000 Telephone
01 261 8140000 Fax
haskoning@royalhaskoning.com E-mail
www.royalhaskoning.com Website

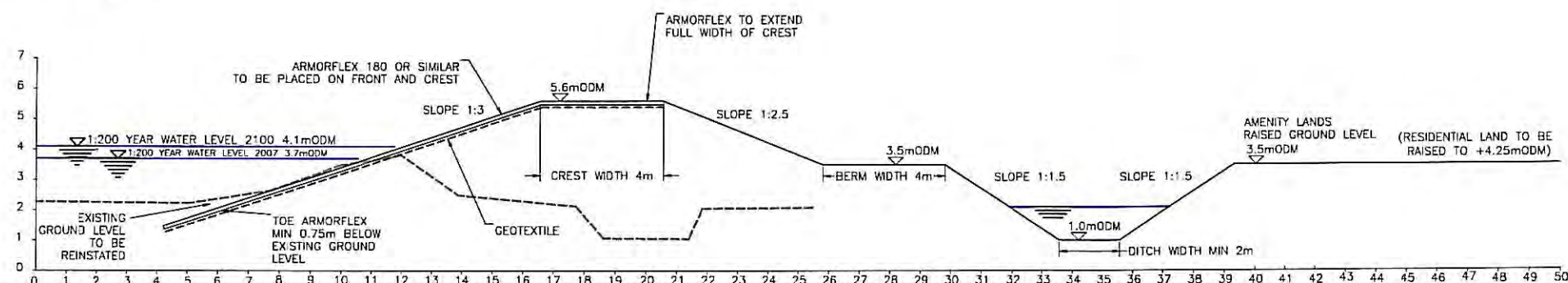
DRAWN MVD CHECKED NM
DATE 02-05-07 CLIENT'S REF.
SCALE AS SHOWN AUTOMATIC REF. 034067A0-F5

FIGURE F5

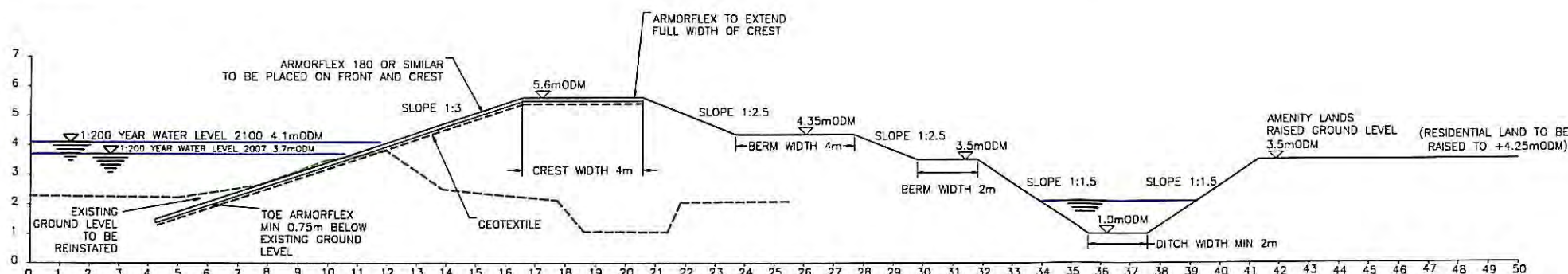
A

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MAIN HEAD.

SECTION A
OPTION 2
EMBANKMENT 1
CHAINAGE 985



SECTION A
OPTION 2
EMBANKMENT 2
CHAINAGE 985



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE
MAIN COASTAL EMBANKMENT
CROSS-SECTION A, OPTION 2
EMBANKMENT OPTIONS 1 & 2

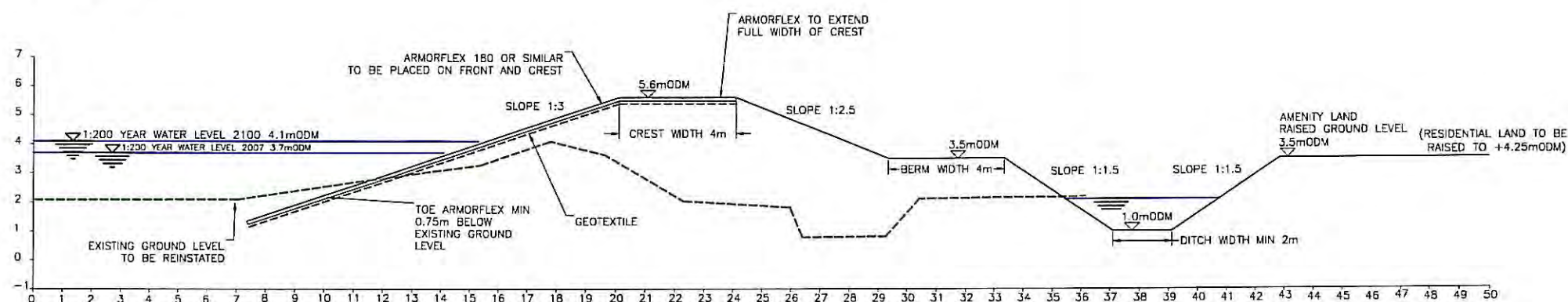
ROYAL HASKONING
18 LOWER BAGGOT STREET
DUBLIN 2
01 261 0140
01 261 0140
www.royalhaskoning.com

DRAWN GG	CHECKED NM	PASSED
DATE 17-05-07	CLIENT'S REF.	
SCALE AT A1	AUTOCAD REF. 9S4067A0-F6	
DRAWING No.	REVISION	

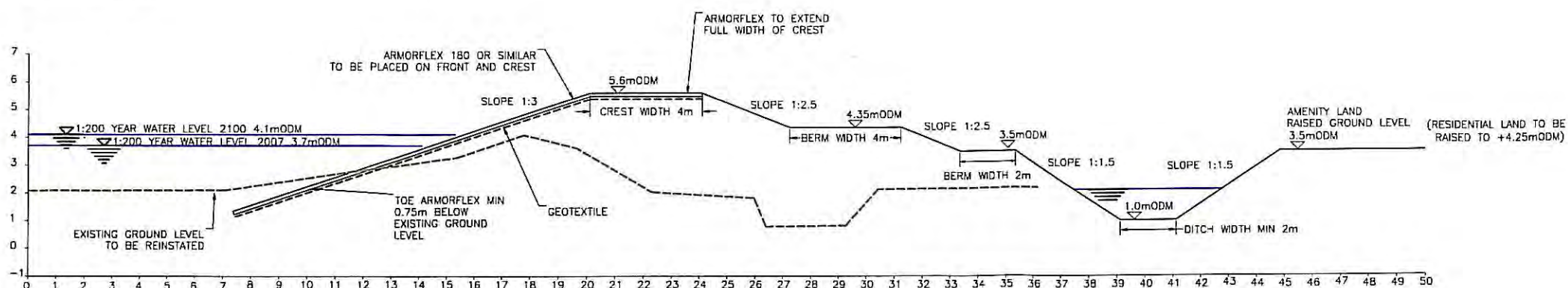
FIGURE F6

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION B
OPTION 2
EMBANKMENT 1
CHAINAGE 1385



SECTION B
OPTION 2
EMBANKMENT 2
CHAINAGE 1385



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE
MAIN COASTAL EMBANKMENT
CROSS-SECTION B, OPTION 2
EMBANKMENT OPTIONS 1 & 2

ROYAL HASKONING
19 LOWER BAGGOT STREET, DUBLIN 2
01 353 0740000 01 353 0740000
www.royalhaskoning.com

DRAWN: GG	CHECKED: NM	PASSED
DATE: 17-05-07	CLIENT'S REF.	
SCALE AT A1	AUTOCAD REF. 9S4067A0-F7	
DRAWING No.	REVISION	

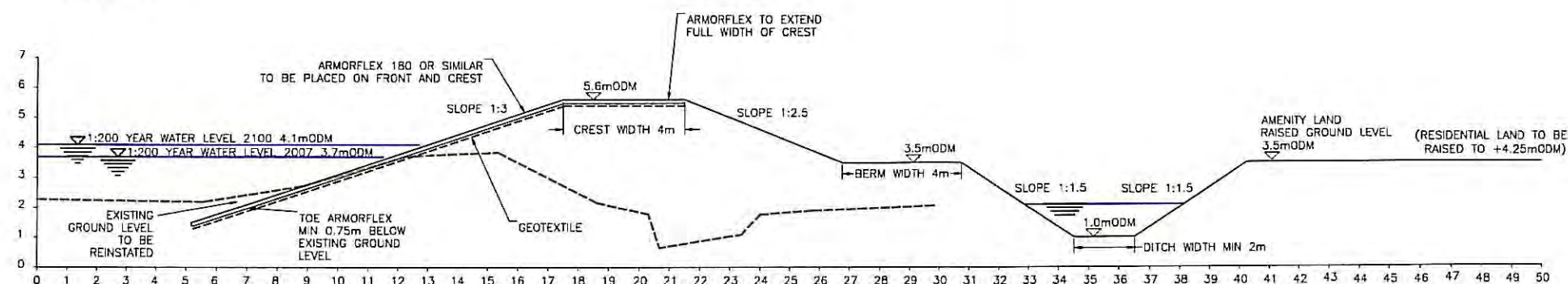
FIGURE F7

DO NOT SCALE

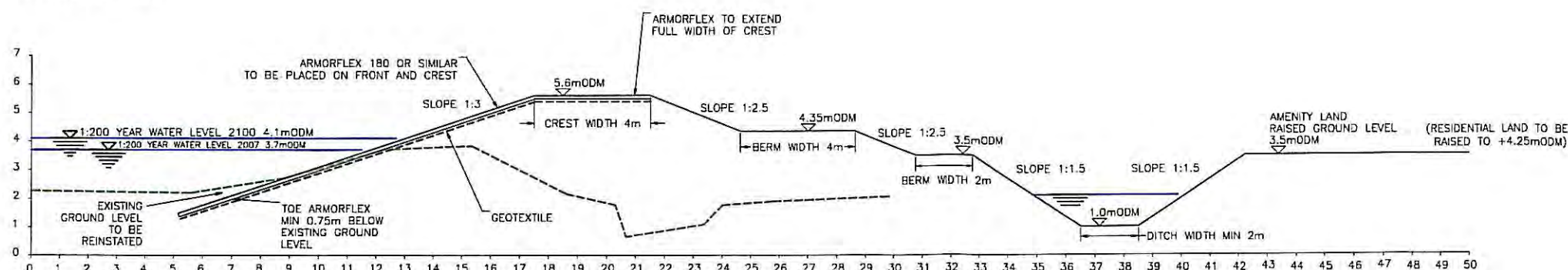
NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION C
OPTION 2
EMBANKMENT 1
CHAINAGE 1991



SECTION C
OPTION 2
EMBANKMENT 2
CHAINAGE 1991



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

MAIN COASTAL EMBANKMENT
CROSS-SECTION C, OPTION 2
EMBANKMENT OPTIONS 1 & 2

ROYAL HASKONING

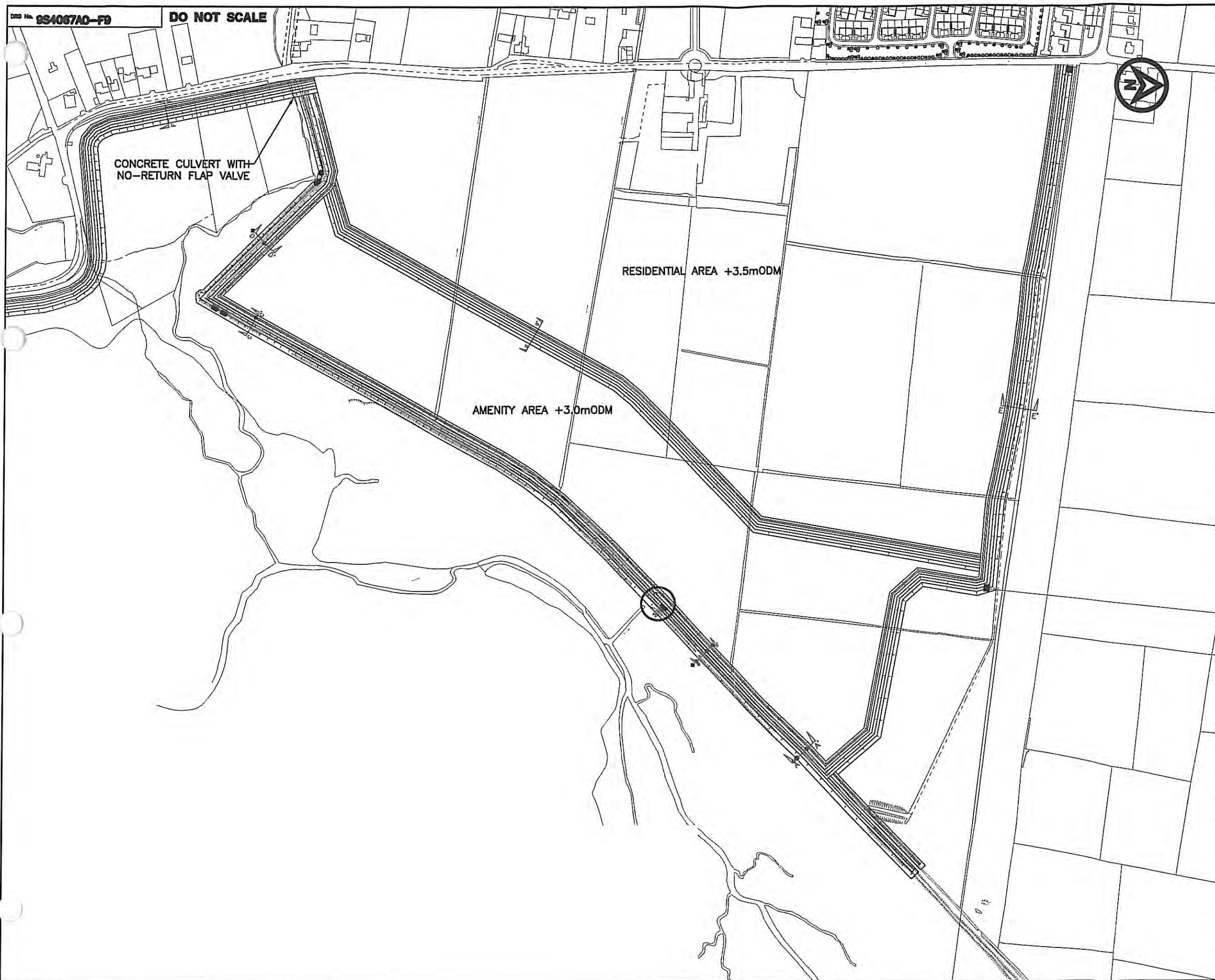
18 LOWER BAGOBY STREET, DUBLIN 2
01 533 0143/0333 01 533 0143/0333
www.royalhaskoning.com

DRAWN: GG	CHECKED: NM	PASSED:
DATE: 17-05-07	CLIENT'S REF:	
SCALE: AT A1	AUTOCAD REF: 9S4067A0-F8	
DRAWING No.	REVISION	

FIGURE F8

NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO OVERSICCE DATUM 100M IN HEAD.



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

EMBANKMENT OVERVIEW
OPTION 3 - EMBANKMENT 2

ROYAL HASKONING

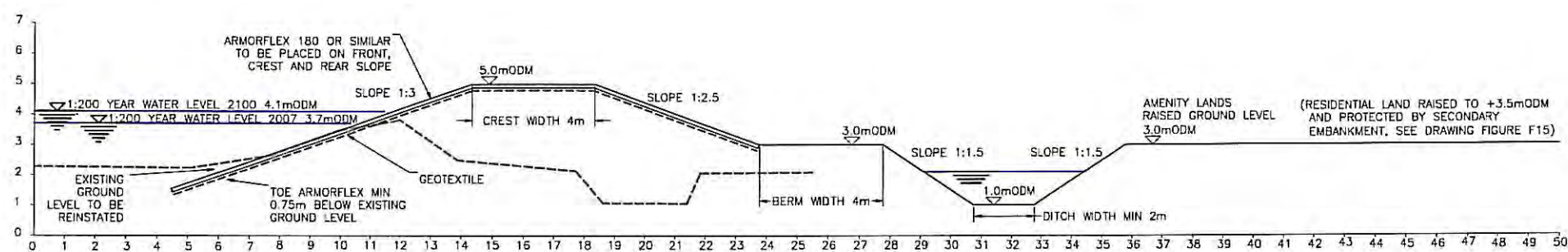
18 LOWER BACCHY STREET, DUBLIN 2
01 855 0154/0155
01 855 0154/0155
www.royalhaskoning.com

Telephone
Fax
Email

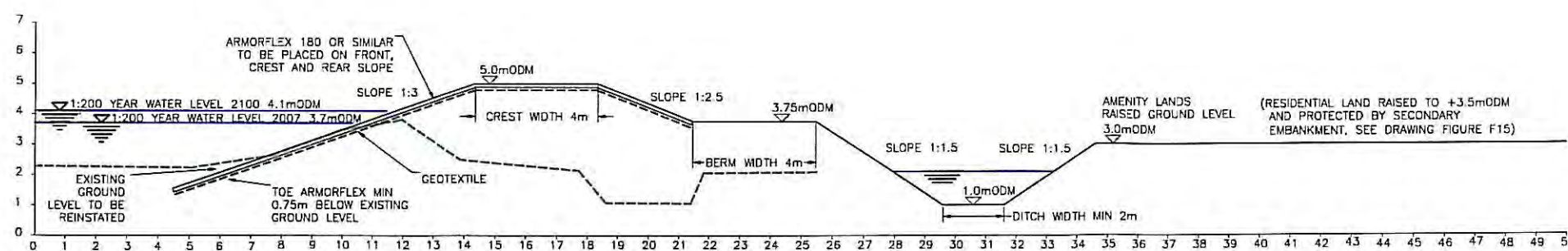
DESIGN: CG	CHECKED: HM	ISSUED:
DATE: 02-05-07	CLIENT'S REF:	
SCALE: AT A1	AUTOMATIC REF: 034067A0-F0	
DRAWING NO.	FIGURE F0	

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION A
OPTION 3
EMBANKMENT 1
CHAINAGE 985



SECTION A
OPTION 3
EMBANKMENT 2
CHAINAGE 985



REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

REVISIONS

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE
MAIN COASTAL EMBANKMENT
CROSS-SECTION A, OPTION 3
EMBANKMENT OPTIONS 1 & 2

ROYAL HASKONING
18 LOWER BACCHY STREET, DUBLIN 8
01 563 8704/5/6/7/8/9
01 563 8704/5/6/7/8/9
www.royalhaskoning.com

DRAWN	GC	CHECKED	NM	PASSED
-------	----	---------	----	--------

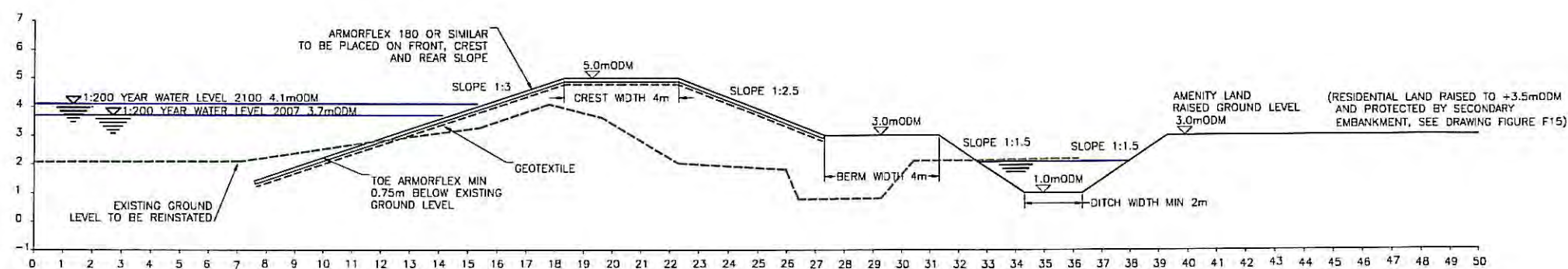
DATE	17-05-07	CLIENT'S REF.
------	----------	---------------

SCALE	AT A1	AUTOCAD REF.	9S4067A0-F10
-------	-------	--------------	--------------

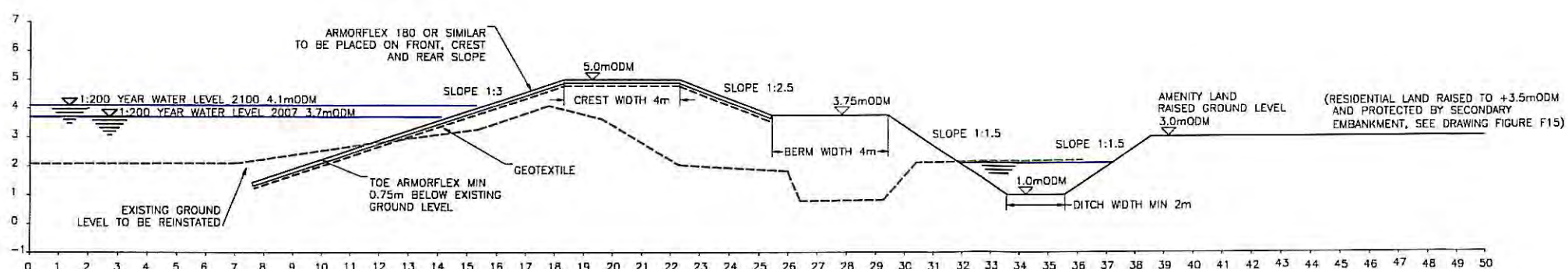
DRAWING No.	FIGURE F10	REVISION
-------------	------------	----------

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION B
OPTION 3
EMBANKMENT 1
CHAINAGE 1385



SECTION B
OPTION 3
EMBANKMENT 2
CHAINAGE 1385



REV	DATE	DESCRIPTION	BY	CHK	APP
REVISIONS					

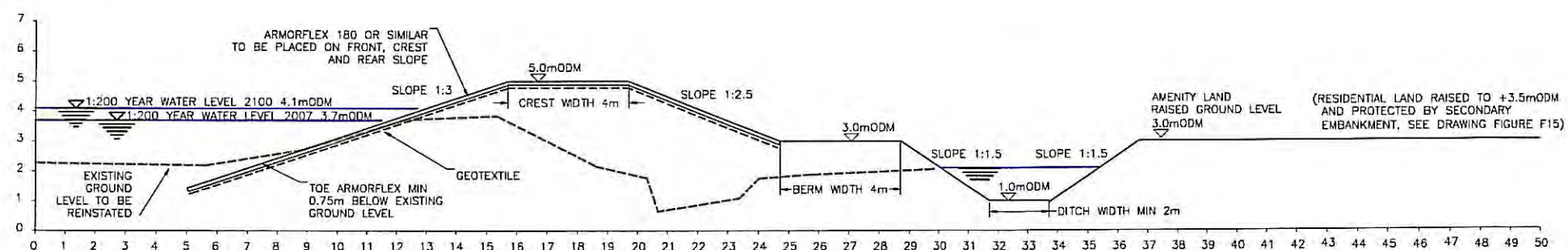
ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE
MAIN COASTAL EMBANKMENT
CROSS-SECTION B, OPTION 3
EMBANKMENT OPTIONS 1 & 2

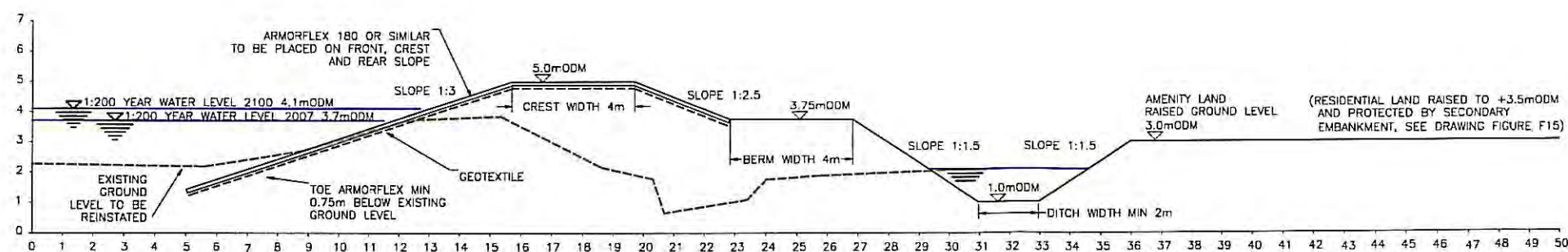
DRAWN GG DATE 17-05-07 SCALE AT A1 DRAWING No.	CHECKED NM CLIENT'S REF. AUTOCAD REF. 9S4067A0-F11	PASSED REVISION
FIGURE F11		

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

SECTION C
OPTION 3
EMBANKMENT 1
CHAINAGE 1991



SECTION C
OPTION 3
EMBANKMENT 2
CHAINAGE 1991



REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

REVISIONS

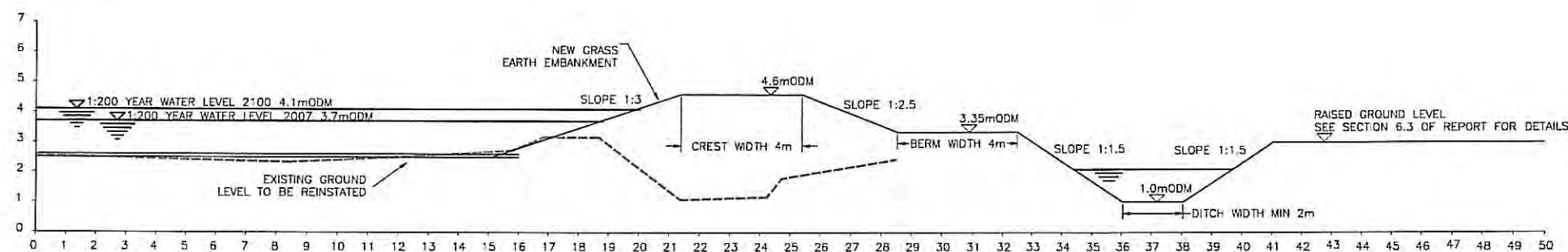
ROCKVIEW FARM
FLOOD RISK ASSESSMENT

MAIN COASTAL EMBANKMENT
CROSS-SECTION C, OPTION 3
EMBANKMENT OPTIONS 1 & 2

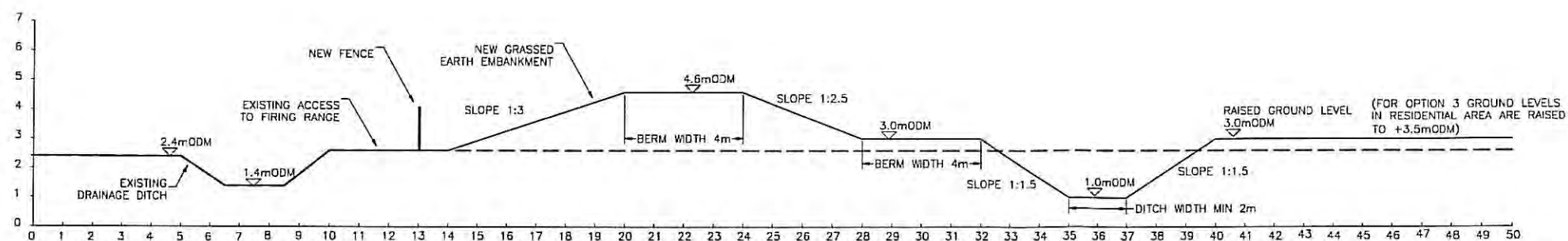
ROYAL HASKONING		
DRAWN GG	CHECKED NM	PASSED
DATE 17-05-07	CLIENT'S REF.	
SCALE AT A1	AUTOCAD REF. 954067AO-F12	
DRAWING No.	REVISION	
FIGURE F12		

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.
3. SECTION D: GIVEN THE LOWER LEVEL OF THE RETURN EMBANKMENT FOR ALL OPTIONS, ONLY ONE EMBANKMENT OPTION HAS BEEN CONSIDERED WITH A BERM LEVEL OF +3.35mODM.
4. NOTE FOR OPTION 2 SECTION E. THIS NORTHERN EMBANKMENT IS ONLY REQUIRED OVER THE AMENITY LANDS SITE AS THE RESIDENTIAL LANDS ARE ALREADY RAISED TO +4.25mODM.

SECTION D
OPTION 1, 2 & 3
EMBANKMENT 1
CHAINAGE 2188



SECTION E
OPTION 1, 2 & 3
EMBANKMENT NORTH BOUNDARY SITE



REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

REVISIONS

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

TITLE
SOUTHERN & NORTHERN
RETURN EMBANKMENTS
CROSS-SECTIONS D & E
OPTIONS 1, 2 & 3

ROYAL HASKONING

19 LOWER BAGGOT STREET
DUBLIN 2
01 223 0143/014333
01 223 0143/014333
r.haskoning@royalhaskoning.com
www.royalhaskoning.com

Telephone
Fax
E-mail
Internet

DRAWN GG CHECKED NM PASSED

DATE 17-05-07 CLIENT'S REF.

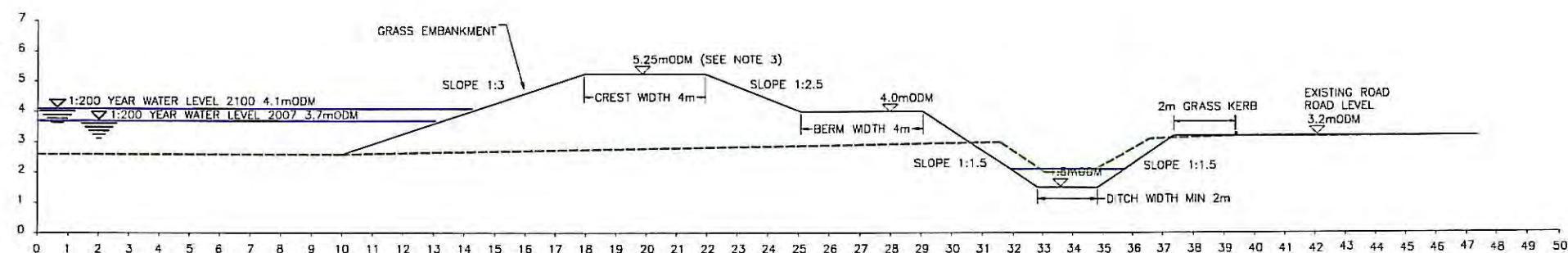
SCALE AT A1 AUTOCAD REF. 954067AO-F13

DRAWING No. REVISION

FIGURE F13

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.
3. CREST LEVEL ASSUMES WAVES OF 40.50m. AS EMBANKMENT MOVES OUT ONTO THE MAIN COASTLINE TOWARDS BLACKROCK VILLAGE THE CREST LEVEL SHOULD INCREASE IN LINE WITH MAIN COASTAL EMBANKMENT TO THE NORTH AND SHOULD BE ARMOURFLEX WITH ARMORFLEX AS REQUIRED. SEE PLAN FOR NOTES.

SECTION F
OPTION 1, 2 & 3
ALONG BLACKROCK ROAD, SOUTH OF SITE



REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

REVISIONS					
-----------	--	--	--	--	--

ROCKVIEW FARM
FLOOD RISK ASSESSMENT

EMBANKMENT ALONG
BLACKROCK ROAD
CROSS-SECTION F

ROYAL HASKONING
19 LOWER BAGGOT STREET
DUBLIN 2
01 352 0140/0141
01 352 0142/0143
www.royalhaskoning.com

DRAWN	GC	CHECKED	NM	PASSED
-------	----	---------	----	--------

DATE	17-05-07	CLIENT'S REF.
------	----------	---------------

SCALE	AT A1	AUTOCAD REF.	9S4067AO-F14
-------	-------	--------------	--------------

DRAWING No.	FIGURE F14	REVISION
-------------	------------	----------

Diagram illustrating the proposed drainage system cross-section. The diagram shows the profile of the ground, including the existing ground level (dashed line) and the proposed raised ground level (solid line).

Key features and dimensions:

- AMENITY AREA RAISED GROUND LEVEL 3.0m ODM**
- GRASS EMBANKMENT**
- SLOPE 1:3**
- 5.0m ODM** (Crest)
- CREST WIDTH 4m**
- SLOPE 1:2.5**
- 3.5m ODM** (Berm)
- BERM WIDTH 4m**
- SLOPE 1:1.5**
- 1.5m ODM** (Ditch Bottom)
- DITCH WIDTH MIN 2.0m**
- SLOPE 1:1.5**
- RESIDENTIAL LAND RAISED GROUND LEVEL 3.5m ODM**
- EXISTING GROUND LEVEL TO BE REINSTATED**

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MALIN HEAD.

REVISIONS

TITLE
SECONDARY EMBANKMENT
FOR OPTION 3
CROSS-SECTION G

ROYAL HASKONING

DRAWN	GG	CHECKED	NM	PASSED
-------	----	---------	----	--------

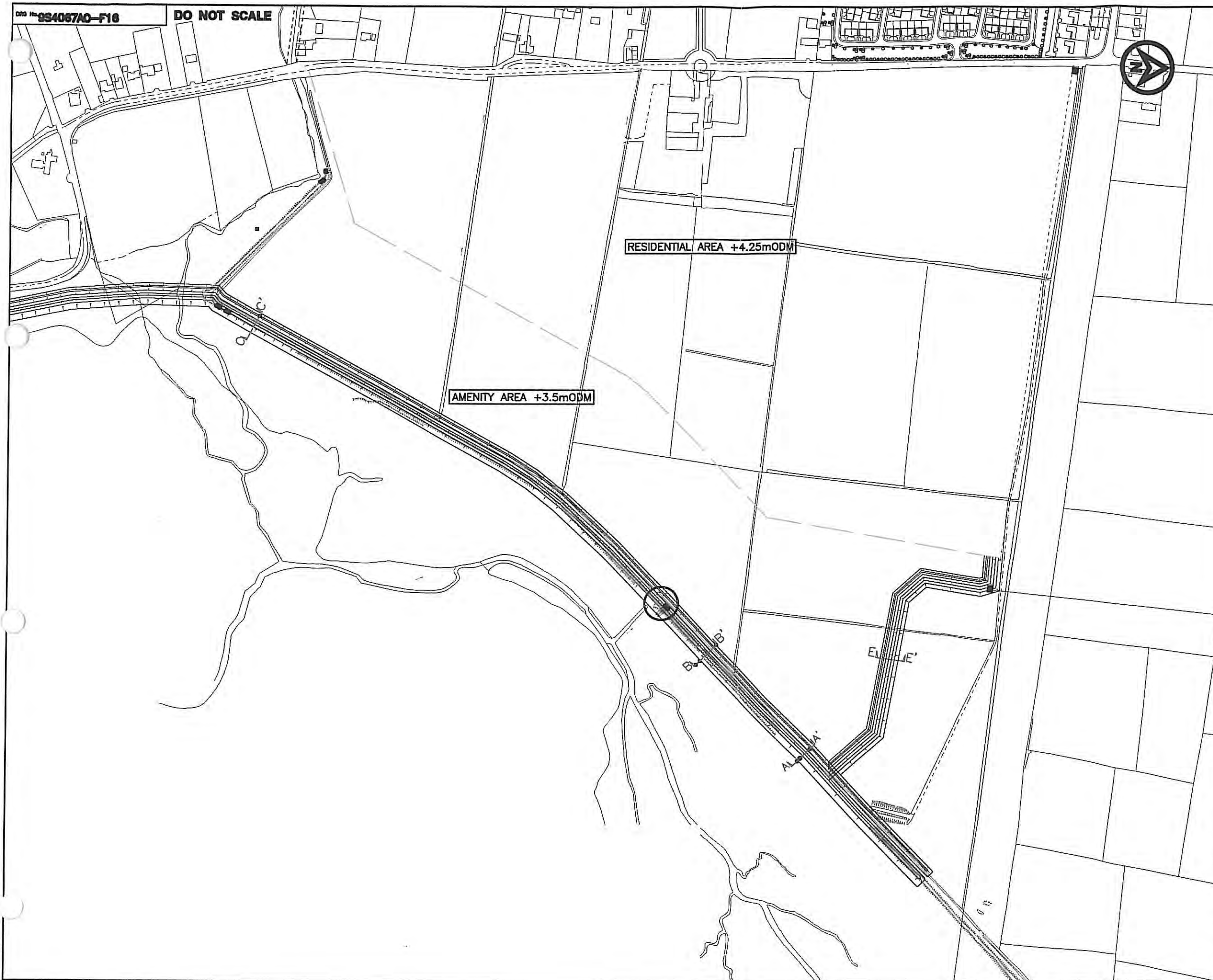
DATE 17-05-07	CLIENT'S REF.
---------------	---------------

DATE	TIME	COUNTY
SAME AS A1	AUTOCAD REF.	9S4067AQ-F1

SCALE AT AT	ASTORIO REF. DE TENDAS
DESCRIÇÃO DE...	DESCRIÇÃO DE...

FIGURE E15

FIGURE 413



NOTES

1. ALL DIMENSIONS IN MM UNLESS OTHERWISE STATED.
2. ALL LEVELS RELATIVE TO ORDNANCE DATUM MAIN HEAD.

REV	DATE	DESCRIPTION	BY	CHK	APP
-----	------	-------------	----	-----	-----

**ROCKVIEW FARM
FLOOD RISK ASSESSMENT**

**EMBANKMENT OVERVIEW
OPTION 2 - ALTERNATIVE SOUTH**

ROYAL HASKONING

18 LOWER DARTMOUTH STREET
DUBLIN 8
D02 YK30
01 253 6714
01 253 6715
01 253 6716
01 253 6717
01 253 6718
01 253 6719
01 253 6720
01 253 6721
01 253 6722
01 253 6723
01 253 6724
01 253 6725
01 253 6726
01 253 6727
01 253 6728
01 253 6729
01 253 6730

DRAWN	CG	CHECKED	NM	DESIGNED	
DATE	02-05-07	CLIENT'S REF.			
SCALE	AS SHOWN	PROJECT REF.	034067AD-F10		

FIGURE F10

Appendix G
Product Information Armourflex

ARMORFLIX®

ARTICULATING CONCRETE BLOCK REVETMENT SYSTEM

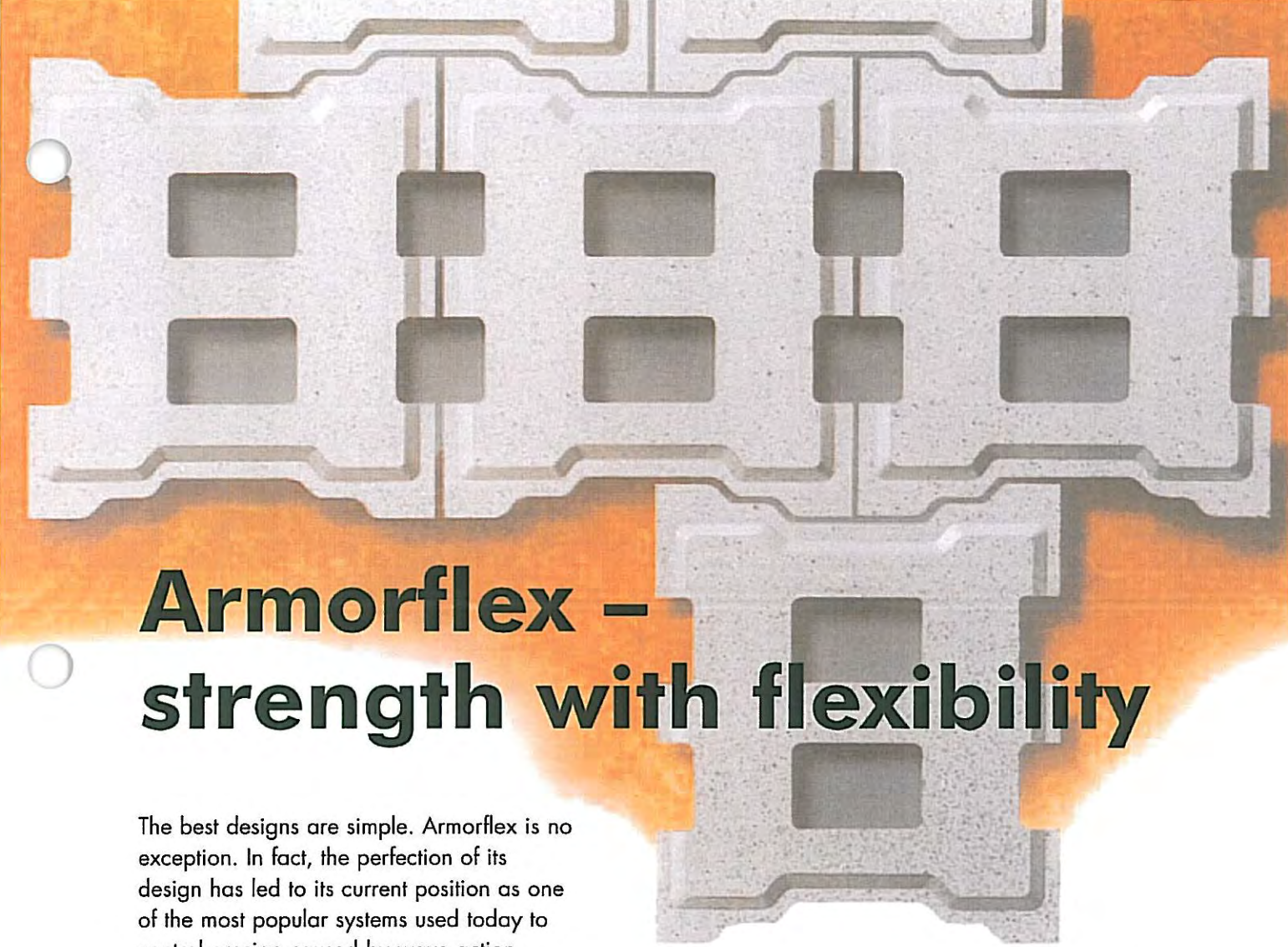
CHANNEL LINING
RIVERBANK PROTECTION
DRAINAGE DITCH LINING
PIPELINE PROTECTION
BOAT RAMPS
RESERVOIR SLOPE PROTECTION

LAKE SHORELINE PROTECTION
BRIDGE ABUTMENT PROTECTION
DYKES PROTECTION
DAM CRESTS AND SPILLWAYS
WEIRS AND OVERFLOW CHANNELS



BY
ARMORTEC™

EROSION CONTROL SOLUTIONS



Armorflex – strength with flexibility

The best designs are simple. Armorflex is no exception. In fact, the perfection of its design has led to its current position as one of the most popular systems used today to control erosion caused by wave action.

A machine-compressed cellular concrete block, Armorflex has a unique interlocking shape – which provides great strength when laid, yet makes it remarkably flexible during installation.

Armorflex mats are manufactured and assembled in our UK factory, where the

blocks are linked longitudinally by means of galvanised wire cables or polyester ropes. The horizontal linkage is made by means of an interlocking stretcher bond.

The made-up mats are then delivered to the site where suitable lifting equipment is provided.



But what about the environment? "Concrete can't be considered when a 'green' solution is desired." Nothing could be further from the truth. One of the great benefits of Armorflex is the way its open structure can actively encourage plant growth when used on canal and river banks. This not only acts as a secure binding agent but can also hide the original structure below a lush cover of vegetation. Armorflex can therefore provide a truly green solution.

With an array of important features, Armorflex is the right solution for a wide range of erosion control problems.

Quality

The blocks are produced using sophisticated, purpose-built machinery to ensure the consistent high quality commensurate with its specification: sulphate-resistant to Grade 2, for excellent durability in service.



Design features

Up to 35% of the surface area can be open by making two rectangular cells within the blocks. This permits free drainage of ground water, thus preventing a destabilising build-up of back-pressure behind the revetment and permits the establishment of indigenous vegetation.

The unique double taper on the sides of the block also greatly enhances the 'friction interlock' when the revetment is blinded with the specified granular mixture.

(Refer to H.R. Wallingford Revetment systems against wave attack - a design manual)





The mats are fully interlocked transversely, so adjacent mats can be butted against each other without discontinuity. Furthermore, mats may be linked both longitudinally and transversely for more aggressive situations. A range of mat weights, from 140 – 435 kg/m², enables the optimum weight to be chosen for each design requirement. Solid blocks are also available.

Easy installation

Laying is comparatively quick, 150 m² per hour for a 4-man team plus crane can easily be achieved.



Armortec does more than provide you with the product. Our Field Engineers will help you select the right product for your particular situation and will assist with on-site advice during the installation.

Moreover, Armortec supplies a wide range of geosynthetic fabrics enabling the best possible solution to be chosen by the designer every time.

Plenty of help

Extensive research has been carried out in the Netherlands, the USA and the UK and design recommendations can thus be made on the basis of extensive trial data and experience.



Armorflex – a quality pedigree

Armortec has carried out extensive research into wave and open channel flow conditions on Armorflex in the United States and the Netherlands. Design manuals and computer programs are available to assist in the proper Armorflex block selection for your hydraulic conditions. Design recommendations can thus be made on the basis of specific research data and sound engineering principles.

Since the early 1980's, Armortec has initiated and participated in a wide range of research projects to evaluate the performance of Armorflex, including the following:

1. Tetratex model tests – California, USA.
2. Lelystad field trials, Netherlands – Rijkswaterstaat Directorate of the Zuiderzee Project, 1982.
3. Wave attack tests, Report No. M1910 – Delft Hydraulics Laboratory, 1982.
4. Hartel Canal Trials – Rotterdam Public Works Laboratory and Delft Soil Mechanics Laboratory.

5. River Waal Breakwaters, Arnhem – Rijkswaterstaat, 1983.
6. CIRIA Report 116 Design of reinforced Grass Waterways.
7. Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow, Colorado State University 1988-2002.

Design assistance

A computer program produced by the Armortec Corporation can be utilised to give valuable assistance at the design stage as well as providing 'in-use' statistics which demonstrate the product's merits.





Installation couldn't be faster or easier

Prior to laying Armorflex, the subgrade must be profiled to line and level and compacted where necessary. As the mats are articulated and will adapt to any profile, great care is necessary at this stage. A geosynthetic fabric is then laid on the profiled surface.

The mats are delivered to the nearest hard road on 40' trailers containing 60 to 180m² per load, with the geosynthetic and the lifting beam delivered on the first vehicle. On suitable sites the delivery can be phased so that mats can be offloaded directly from the road vehicles onto the prepared subgrade.

If the revetment has a gradient of 1:2 or steeper, the mats may slide on the geosynthetic fabric until the system settles and vegetation begins to grow. Temporary anchorage can be achieved by means of wooden pegs driven at 0.6m centres along the top of the mats. Permanent anchorage for special installations can be achieved using ground anchors and non-corroding cables.



Mats subject to wave attack should be blinded with a 5mm to 30mm sand/gravel mixture with a D₁₅ of 5mm. Above the normal waterline, mats may be topsoiled and seeded to give a 'green' effect.

Filling volumes are as follows:

Armorflex 140	- 0.02 m ³ per m ²
Armorflex 180	- 0.03 m ³ per m ²
Armorflex 220	- 0.04 m ³ per m ²
Armorflex 165	- 0.01 m ³ per m ²
Armorflex 215	- 0.012 m ³ per m ²
Armorflex 305	- 0.015 m ³ per m ²
Armorflex 435	- 0.023 m ³ per m ²

Product Advice

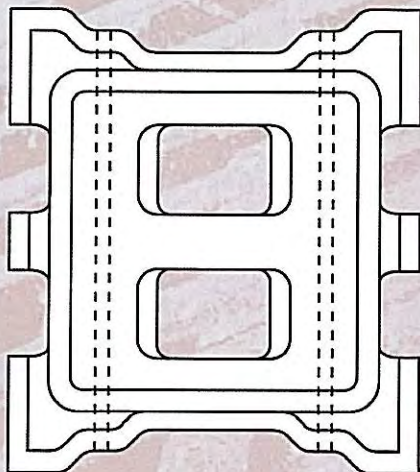
Our Field Engineers will help you select the right product for your particular installations and will assist with on-site advice during the installation.



Summary of benefits

Armorflex is a flexible, interlocking matrix of concrete blocks of uniform size, shape and weight connected by a series of cables which pass longitudinally through preformed ducts in each unit. **Armorflex** is installed over site-specific filter fabric on a prepared surface. **Armorflex** revetment systems combine the favourable aspects of lightweight blankets and meshes, such as porosity, flexibility, vegetation encouragement and habitat enhancement with the nonerodable, self-weight and high tractive force resistance of a rigid lining.

Armorflex has proven to be an aesthetic and functional alternative to dumped stone riprap, gabions, structural concrete and other heavy-duty, durable erosion protection systems. **Armorflex** is easy to install, therefore, can dramatically reduce overall project costs. More specifically, when compared to other revetment systems, life-cycle costs have been reduced because **Armorflex** is a permanent system and saves on subsequent maintenance expenses.



**Resistant to
wave attack**

**Prefabrication at
works for fast site
installation**

**Underwater
installation
possible**

**Economical and
easy to handle**

**Advanced design –
interlocking blocks
and double-taper
for greater
strength**

**Different weights
and sizes
available for
optimum design**

**Encourages
plant growth –
environmentally
acceptable**

**Mats can be lifted
and re-used**

Resistant to frost

**Flexible: will
accommodate
variations in
profile**

Specifications

Concrete cube strength: 50 N/mm² at 28 days
Sulphate resistant to Class DS-2.
Cables: galvanised or polyester.
Mat sizes 6 x 2.4m, 6 x 1.2m, other lengths available.
(see also detailed specification sheet)

Cellular block mats

Armorflex 140

L x B x H: 340 x 400 x 85mm
Block weight: 17.5kg each
Unit weight: 140kg/m²
Open area (max.): 35%

Armorflex 140S

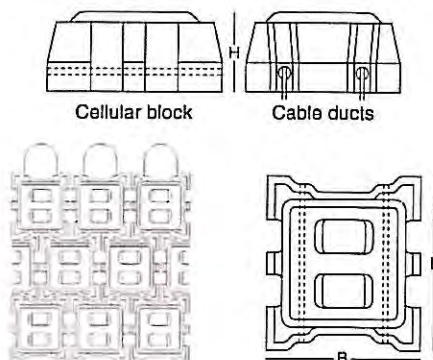
L x B x H: 340 x 400 x 90mm
Block weight: 19kg each
Unit weight: 150kg/m²
Open area (max.): 35%

Armorflex 180

L x B x H: 340 x 300 x 120mm
Block weight: 17.3kg each
Unit weight: 180kg/m²
Open area (max.): 35%

Armorflex 220

L x B x H: 340 x 300 x 150mm
Block weight: 21.9kg each
Unit weight: 225kg/m²
Open area (max.): 35%



Solid block mats

Armorflex 165

L x B x H: 340 x 400 x 85mm
Block weight: 20kg each
Unit weight: 160kg/m²
Open area (min.): 6%

Armorflex 215

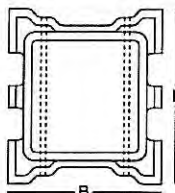
L x B x H: 340 x 300 x 120mm
Block weight: 21kg each
Unit weight: 220kg/m²
Open area (min.): 6%

Armorflex 305

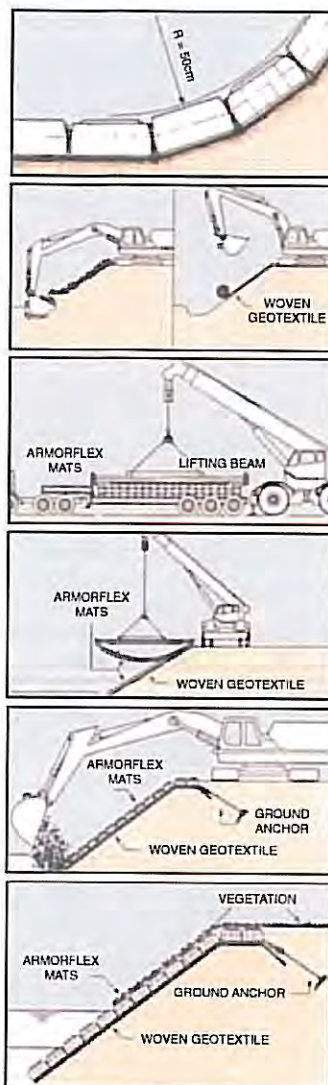
L x B x H: 340 x 300 x 150mm
Block weight: 26.7kg each
Unit weight: 285kg/m²
Open area (min.): 6%

Armorflex 435

L x B x H: 340 x 300 x 225mm
Block weight: 41kg each
Unit weight: 435kg/m²
Open area (min.): 6%



Also refer to:- Revetment Systems against wave attack – a design manual. Hydraulics Research, Wallingford.



Wave attack data

The following table is based on Delft Hydraulics Laboratory Report M 1910 and is intended to offer a guide to designers. Designs for specific projects can be prepared using the report itself. The table assumes the following data:

- Revetment slope 1 in 3.
- No reinforcement effect from cables (assumed corroded).
- Revetment blinded with sand/gravel mixture after installation.
- Irregular wave pattern $T_p = 3.75$ secs (most damaging period found in tests).

Armorflex type	Significant Wave Height H_s	Maximum Wave Height H_{max}
140 Cellular	1.0m	1.2m
180 Cellular	1.3m	1.5m
220 Cellular	1.6m	1.8m

NB. Run-up Cellular blocks mats can reduce wave run-up by 30% as compared with smooth revetment. This may enable a reduction in the height required for the protected area.



Concrete Products (Lincoln) 1980 Ltd
Verside, Skellingthorpe Road, Saxilby, Lincoln LN1 2LR
T: 01522 703681 F: 01522 704233
info@armortec.co.uk
www.armortec.co.uk

AUTHORISED AGENT



0800 5878309

www.water-lines.co.uk
info@water-lines.co.uk

Orchard House, Glebe Road, Huntingdon
Cambridgeshire PE29 7DL