Final Report

Volume 1 - Main Report

Dublin City Council
Fingal County Council

29 April 2005

Final Report

9M2793

A COMPANY OF

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Document title	Final Report Volume 1 - Main Report
Document short title	
Status	Final Report
Date	29 April 2005
Project name	Dublin Coastal Flooding Protection Project
Project number	9M2793
Client	Dublin City Council
Reference	9M2793

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ACKNOWLEDGEMENTS

The Dublin Coastal Flooding Protection Project is a landmark project for Dublin City and Fingal County Councils in terms of co-operation and consultation. Its inception and completion acknowledges that flooding, as a result of adverse coastal conditions, is a phenomenon that requires active management. Haskoning is proud to have been associated with this project. Without the invaluable contributions made by all those who have been involved to date, the completion and submission of the flood warning system would not have been achieved.

We would like to take this opportunity to thank all those who have provided assistance, together with those who have provided data and information for use. In particular we would like to extend our thanks to

- Dublin City Council
- Fingal County Council
- Department of Communications, Marine and Natural Resources
- Office of Public Works
- EPA
- ESB
- Dublin Port Company
- Dun Laoghaire Harbour Company
- Met Eireann
- UK Met Office
- DDDA
- The participants of the stage workshops
- Consultants who have provided data through other projects.

GLOSSARY OF TERMS

Admiralty Chart	Chart providing details of seabed bathymetry.
Armour Unit	Large quarry stone or concrete unit used as primary wave protection to a breakwater or revetment.
Barometric Pressure	Air pressure that can cause fluctuations in the sea level. Low barometric pressure will raise sea level, high barometric pressure will depress sea level.
Bathymetry	Topography of sea/estuary/lake or river bed.
Breakwater	A structure protecting a shore area, harbour, anchorage or basin from waves.
Coast Protection	The prevention of erosion of the coastline from the action of waves and tidal currents.
Demountable Defence	A moveable flood protection system that is fully pre-installed and requires operation during a flood event or a system that requires part installation into guides or sockets within a pre-constructed foundation.
Depression	A region of low barometric pressure
Earned Value	A generic performance measurement term used to describe the physical work accomplished in terms of financial worth accrued.
Earned Value Management	The process of representing physical progress achieved on the project in terms of a cost based measure, i.e. money, or in some cases man hours.
Ebb Tide	The period of tide between high water and the succeeding low water; a falling tide.
Embankment	A mound of earth or stone built to hold back water or to support a roadway.
Erosion	The wearing away of material by the action of natural forces.
Estuary	The region of a river that is affected by tides.
Extreme Tide Level	Water level resulting from the summation of normal astronomic tide levels and a storm surge residual created by meteorological conditions.
FINEL	Final Element a 2-dimensional numerical model for the computation of shallow water flow and transport processes in rivers and coastal waters. In the context of the project it was used to reproduce tidal currents and levels within the study area and to develop relationships for the transfer of surge conditions from offshore to nearshore.
Fishtail Groyne	A 'v-shaped' rock groyne that is constructed along a coastline to aid accretion of beaches.
Flood Alleviation	The prevention or reduction of the impacts of flooding due to the implementation of appropriate works or procedures.
Flood Defence	A formal flood protection system that is managed or controlled by an Operating Authority with responsibility for flood defence.
Flood Gate	A gate across an access that is temporarily closed during a flood event to prevent flooding.
Flood Tide	The period of tide between low water and the succeeding high water; a rising tide.
Flood Warning	An alert which activates a pre-defined set of procedures to warn the public at risk and where appropriate implement flood protection systems such as demountable defences or flood gates.
Flooding	The inundation of land by water through the action of the sea, rivers or meteorological conditions.
Foreshore	The part of the shore, lying between the crest of the seaward berm (or upper limit of wave wash at high tide) and the ordinary low-water mark, that is ordinarily traversed by the uprush and backrush of the waves as the tides rise and fall.
Gabion Wall	A wall constructed using a cubed hollow steel cage filled with earth and stones.
Harmonic Components	Components use to predict the movement of the tides .
Hydrodynamic Model	Numerical model used to simulate tidal flow and or wave conditions.

Hydrographic Survey	Survey undertaken in relation to the seas, lakes or rivers, usually includes survey of underwater bed levels, water levels, currents, suspended sediment concentrations and bed samples.
Hydrology	The study of the properties, distribution, and effects of water on the earth's surface, in the soil and underlying rocks, and in the atmosphere.
Inshore	Coming from the sea toward the land.
Meteorological	The science that deals with the phenomena of the atmosphere, especially weather and weather conditions.
Numerical Model	Computer generated model that uses numerical methods to solve the mathematical equations relating the physics of the processes to be modelled.
Offshore	The comparatively flat region of submerged land extending seaward from beyond the region where breakers form to the edge of the continental shelf.
Overtopping	Passing of water over the top of a structure as a result of wave runup or surge action.
Pallet Barrier	A demountable flood barrier consisting of galvanised steel supports holding a standard wooden Euro pallet.
Policy	A plan or course of action, as of a government, political party, or business, intended to influence and determine decisions, actions, and other matters.
Predicted Tide	Tides that have been calculated using the harmonic components derived from continuous observation of tides over the period of one year.
Probabilistic Analysis	In the context of this project, the process by which the probability, or chance, of an event occurring is determined.
Promenade	A public area set aside as a pedestrian walk.
Repointing	Re-grouting of blockwork of masonry walls to provide a watertight skin, to reduce the chances of collapse through undermining by river or seawater.
Residual Life	The remaining life of a structure, before any maintenance or replacement is required.
Revetment	A cladding stone, concrete or other material used to protect the sloping surface of an embankment, natural coast or shoreline against erosion.
Risk Breakdown Structure	A risk based breakdown which defines the risk categories within a project.
Seawall	A structure separating land and water areas to alleviate the risk of flooding by the sea, built along a portion of a coast to prevent erosion and other damage by wave action. Often it retains earth against its shoreward face.
Seiche	A short-period oscillation occurring in a harbour, bay or gulf. It can be caused by change in meteorological conditions, such as the passage of an intense depression or line squall or local topography. The period between successive waves may be anything between a few minutes and about two hours and the height of the waves may be anything from a few centimetres to a metre or even more.
Shelf Seas Model	A general purpose 3D baroclinic model developed by Proudman Oceanographic laboratory Birkenhead. Model area, resolution and input condition are the same as those used in the surge model. It is forced both atmospherically and ocean boundaried. Atmospheric forcing from hourly values of wind stress and pressure, provided by the NWP Mesoscale Model at the boundaries 15 tidal components and a radiated condition are imposed. It provides total elevation data from Time –6hrs to Time +48hrs. The grid cells are approximately 12km^2.
Standard of Protection	The protection offered by a defence in terms of an extreme return period event.
Strategy	A systematic plan of action.
Surge	A coastal rise in water level caused by changes in meteorological conditions. There are two methods of measuring surge:

Residual-surge defined as the difference between the actual water level (including tide and surge components) and the predicted (or hindcast) astronomical tide level at the same point in time. It represents the difference in levels without any correction or allowance for phase differences between the observed and predicted patterns.

Skew-surge defined as the difference between the actual high water (including tide and surge components) and the predicted high tide level.

SWAN SWAN is an acronym for Simulating Waves Nearshore, and is a third-generation wave model which computes random, short-crested wind-generated waves in coastal regions and inland waters.

Tide Rise and fall of the sea, happening twice each lunar day.

Topographic Survey The survey of a surface, including its relief and the positions of its streams, roads, building, etc.

Transfer Matrix A matrix of coefficients that transform input data to provide output, in the context of wave propagation the transfer of offshore wave patterns into the inshore using a matrix generated by a series of runs undertaken using the SWAN wave propagation model.

Wave Transformation The transfer of offshore waves to inshore.

Wave Transformation Model Numerical model taking into consideration the shallow water processes that transform waves as they propagate from offshore to the nearshore.

Weir A low dam or wall across a stream or river to raise the upstream water level. Termed fixed crest weir when uncontrolled.

Weiring The action of water flowing over a structure.

Work Breakdown Structure A task oriented detailed breakdown that defines the work packages and tasks to be undertaken. The grouping of the tasks within the WBS defines the total scope of the project.

Underpinning Material or masonry used to support a structure, such as a wall.

ZWENDL The one – dimensional hydraulic river model developed by Rijkswaterstaat in the Netherlands. Calculating water levels, discharges , salt concentrations and currents.

1 INTRODUCTION

1.1 General

In September 2002, Haskoning (formerly Posford Haskoning) submitted their proposal to Dublin City Council for the role of service provider on the Dublin Coastal Flooding Protection Project. In November of the same year they were identified as the preferred bidder by Dublin City Council, which preceded a period of negotiations. The negotiations concluded with the appointment of Haskoning in April 2003, as the service provider for the project. A pre-commencement meeting was held in Dublin on the 11th April 2003, which addressed the main stakeholders associated with the project. Following that meeting the project team mobilised in Dublin on the 12th May 2003 with substantial commencement of the project occurring on the 19th May 2003.

The report is presented in three volumes:

- Volume 1:- Technical Report and Executive Summary
- Volume 2:- Appendices
- Volume 3: - Drawings

In it is inevitable in reports of this nature, that a terminology is adopted which reflects the nature of the work undertaken. The wider readership of this report may therefore be unfamiliar with such terminology. Consequently, a glossary of terms used and found in this document is included to aid the reader in the understanding of the technical terminology.

Figures and sketches are presented in Appendices and are numbered sequentially to reflect the Chapter number in which they appear. For example, Appendix A contains Figures 1.1 to 1.8 all of which relate to Chapter 1, whilst Figures 9.1 to 9.15 relating to the content of Chapter 9 may be found in Appendix H.

1.2 Scope, Objectives and Project Aims

The Dublin Coastal Flooding Protection Project has been implemented in direct response to the extreme tide and flood event that was experienced across Dublin City and Fingal County during the 1st February 2002. This tide was the highest on record since 1922, being in excess of 1 metre above the predicted tide for that day. It caused extensive flooding and disruption at a number of locations across Dublin City and within Fingal County. The Dublin Coastal Flooding Protection Project is primarily aimed at addressing the risk from tidal flooding around the coastline and within the tidal reaches of a number of the rivers and canals. More specifically the project area encompasses:

- The coastline from the Martello Tower to the North of Portmarnock, to the east pier at Howth Harbour.
- The coastline from the Martello Tower on the South side of Howth Head to the Dublin city boundary at Merrion, including the Bull Island and the Dublin Port area.
- The tidal reaches of the River Liffey to Islandbridge Weir.
- The tidal reaches of the River Dodder to Ballsbridge Weir.
- The River Tolka to Annesley Bridge (subsequently Haskoning agreed to model up to Distillery Weir).
- The tidal reach of the Royal Canal to Strand Road.

- The tidal basin of the Grand Canal as far as the 1st lock.

A plan showing the extent of the above project area is presented in Figure 1.1.

Figure 1.1 - Project Extent

The main objectives and aims of the Dublin Coastal Flooding Protection Project are to:

- Undertake a strategic examination of the risk to Dublin from coastal flooding.
- Identify appropriate strategies and polices to combat and manage the risk.
- Identify short term urgent works on experience gained from the February 2002 event.
- Identify medium to long term options to reduce and/or manage the risk.
- Learn from the past.

In order to achieve these specific study objectives and aims, a number of study tasks and goals must be achieved. These include:

- Capture and analyses all relevant project data.
- Consult and liaise with all other DCC and FCC flood risk initiative projects.
- Carry out a public information campaign, including the creation of a web site
- Undertake a detailed asset condition survey of the coastal and tidal defences within the project area.
- Undertake a probabilistic assessment of existing tidal records.
- Undertake mathematical modelling for use in the development of a forecasting system.
- Identify areas at risk to coastal flooding and quantify the extent of those risks.
- Assess the impact of those flood risks identified.

- Identify risk reduction works and assess the merits of each to identify a preferred option(s).
- Develop preferred option(s) into work packages and prioritise.
- Investigate and provide a specification for the development of an Early Warning System.
- Identify a long term strategy for the area.

Each of the above specific goals and tasks are incorporated within the project programme and methodology to ensure that the overall project aims are achieved. The overall project and methodology has been broken down into four phases and more details of these, the project methodology and programme are presented in Chapter 4.

1.3 Sponsoring Authorities

The primary sponsoring authority for the Dublin Coastal Flooding Protection Project is,

- Dublin City Council (DCC).

In addition there are a number of co-sponsoring authorities to the project and they include,

- Fingal County Council (FCC).
- Department of Communications, Marine and Natural Resources (DCMNR).
- Office of Public Works (OPW).

An extensive list of project stakeholders has also been developed and they are regularly consulted and involved in the project as and when input from their particular field of expertise is required.

1.4 The Dublin Coastal Flooding Protection Project in the Context of the European SAFER Initiative

1.4.1 INTERREG III B

INTERREG III is an EU Community Initiative to promote transnational co-operation on spatial planning by encouraging harmonious and balanced development of the European territory. The overall aim is to ensure that national borders are not a barrier to balanced development and the integration of Europe and to strengthen co-operation of areas to their mutual advantage. **Interreg IIIB** represents Transnational co-operation on spatial development between national, regional and local authorities and a wide range of non-governmental organisations. The objective is to achieve sustainable, harmonious and balanced development in the Community and better territorial integration.

1.4.2 SAFER

Standing for Strategies & Actions for Flood Emergency Risk management, SAFER is an Interreg IIIB approved project comprising five partners:

- Gewasserdirektion Neckar, Germany (Lead Partner)
- Dublin City Council, Ireland
- Forestry Commission of Scotland, Scotland

- Federal Office for Water & Geology, Switzerland
- Ecole Polytechnique Federale, Switzerland

The SAFER Project is an innovative proposal to develop a best practice approach to flood risk management based on three themes:

Flood Hazard Information
- hazard maps, flood frequency determination etc.

Flood Emergency Response
- technical defences (barriers, soft defences, demountable defences.
- seamless systems from flood early warning; to call out and response; and recovery.

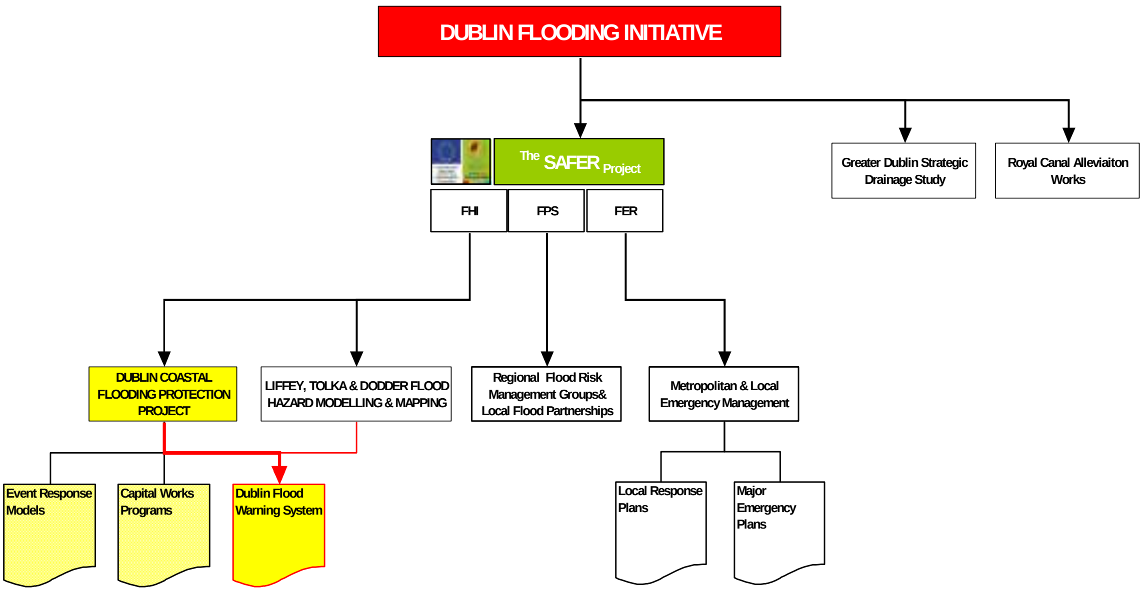
Flood Partnerships
- with national & regional government and agencies; & communities

The Dublin Coastal Flooding Protection Project (DCFPP) forms a major constituent of Dublin City's work on the SAFER project and also within an overall Dublin Flooding Initiative:

It provides, for the first time, the flood hazard information on Dublin's coastal flooding risk; this flood hazard information will enable flood risk management plans to be formulated and put in place. The study, mainly through its various workshops, has also fostered many of the working relationships with other regional and national stakeholders. This work has provided the first steps to identifying the flood partnerships.

The DCFPP has formed links with the other DCC flooding projects and is foremost in promoting an interim early warning system addressing coastal flooding.

The overall SAFER structure is shown in the flowchart below.



Flowchart 1.1 - The overall SAFER structure

1.5 Description of the Project Area

The extent of the project area has been outlined in Section 1.2 above and is shown on Figure 1.1. It includes both the coastal boundaries and also river and canal boundaries over their tidal reach. The nature of the coastline and river/canal boundaries and their respective quays and defences vary in type throughout the study area. An extensive inspection of the project area has been undertaken and the results presented in a database, which is discussed in more detail in Chapter 5 – Asset Survey. However, it is felt that a brief description of the project area should be given within the report and this is presented below. The text that follows should be read in conjunction with the Figures 1.2 to 1.8 and photographs presented in Appendix A.

1.5.1 Fingal County Council

i) Portmarnock to Baldoyle – Photographs A1 – A8 & Figures 1.3 & 1.4

The northern end of the study area commences at the Martello Tower, see Photographs A1 and A2 and Figure 1.3. Here, the coastline is defined by high promenade walls with some pedestrian access points (Photograph A3) leading to the beach in the village of Portmarnock. The study area then extends along Velvet Strand sand dune system which contains two golf courses. The sand dune system is a mature and well developed system although there is evidence of erosion in a number of places. This has been addressed on the seaward side over a number of lengths by placing old timber rail sleepers along the toe in a vertical position, see Photograph A3. On the Baldoyle Estuary side rock and building rubble has been placed over part of the length of the dunes to help protect them, although erosion on this side must be considerably less due to the sheltered nature of the estuary. Photograph A4 shows the Strand Road that runs out of Portmarnock along the northern end of the Baldoyle Estuary. Photograph A5 is taken at the roundabout just south of Portmarnock, at the junction of Strand Road and Coast Road, see Figure 1.3, where flooding of the road and roundabout occurred. The problem here has been reported as being due to drainage from the fields which becomes tide locked at the outfall into the estuary at this location. The water as a result gathers on the road and roundabout which are low in level. However in addition to this, a low spot between the wall and adjacent embankment was noted just off photograph A5, which could also have resulted in some overtopping at that location.

Continuing south along the Coast Road to Baldoyle, the coastline comes close to the road in a number of locations, with some places protected against erosion and others not, see Photographs A6 and A7. No significant defence structures were noted along this length and the road looks low enough for flooding to occur, indeed it is reported that the road and two adjacent properties were flooded. Photograph A8, see Figure 1.4, shows the North Fringe drainage scheme works which are currently underway in Baldoyle. As can be seen from the photograph, the scheme will provide a new promenade and seawall slightly seaward of the existing. However, the existing wall is also being maintained to act as a secondary defence that will be of considerable benefit from a flood protection point of view.

ii) Baldoyle to Howth Harbour – Photographs A9 – A11 & Figure 1.4

Photographs A9 and A10 show the sand-dune system, which fronts the properties along the Burrow Road east of the Sutton Golf Links. The dune system would provide a degree of protection against wave action along this length, although there was noted to be a number of vehicular access paths through the system which it is believed will act as flood paths to Burrow Road, see Chapter 3. In addition to the dunes the properties appear to have a wall around them but it is not thought to be flood defence in nature, although it would provide some benefit. Photograph A11 looks east along the port area from the western pier at Howth Harbour, which is the limit of the project extent on the northern side of Howth Head. This length of the project area contains wall and revetment type structures.

iii) Martello Tower, Howth Head South Side to Sutton Cross – Photographs A12 – A17 & Figure 1.4

At the location of the Martello Tower and for some distance along the coastline to the north west, the land is high and not at risk of flooding. However, there are a number of locations where coastal erosion is beginning to cause a problem, see Photograph A12. Whilst this does not pose a treat from a flood risk point of view as the ground levels are high in this area, continued erosion could eventually threaten the road. Beyond the junction with Strand Road and St Fintan's Road, the ground levels begin to reduce and the coastline consists of a vertical concrete and in places block wall, see Photographs A13 and A14 and Figure 1.4. A number of gaps exist in this wall to allow pedestrian and vessel access to the beach, see Photograph A14. Further west at the junction of Strand Road with Greenfield Road, the wall ends and is replaced by natural bank, which is in places protected by building rubble and shingle, see Photograph A15. A number of properties which lie on the southern side of Sutton Cross junction, have gardens that back onto the foreshore. However, the majority of these have some form of wall at their seaward boundary limit albeit they vary in type and condition, see Photographs A16 and A17.

iv) Sutton Cross to Kilbarrack Road (Fingal County Council Boundary) – Photographs A18 & A19 & Figures 1.4 & 1.5

Fingal County Council reported that there was no flooding along the length of Dublin Road as far as their boundary with Dublin City Council at the Kilbarrack Road. It is likely that this is a direct result of the protection that Bull Island offers this length of the project area against wave activity. Photograph A18 shows the start of this section at the western end of the Sutton Cross properties, see Figure 1.4. The typical land water interface over most of this length generally consists of a vertical quay or steep revetment type structure with low wall on top, see Photograph A19.

1.5.2 Dublin City Council

i) Kilbarrack Road to Wooden Bridge, Bull Wall – Photographs A19 – A21 & Figure 1.5.

From the Kilbarrack Road to the Bull Wall Bridge. The typical land water interface generally consists of a vertical quay or steep revetment type structure with low wall on top, see Photograph A19. No flooding was reported along this length and again this is likely to be as a direct result of the protection that Bull Island offers this length of the project area against wave activity.

ii) Bull Island – Photographs A20 – A 23 & Figure 1.5

Bull Island is a recently formed dune system which formed following construction of the North Bull Wall, see Photograph A20. The dune frontage around the island gives way to more low lying land behind, which consists of grazing marsh and golf courses, see Photograph A21. From the visual inspection, the standard of protection of this low lying area could be considered to be adequate for the general use of the land, i.e. the land does not appear to be getting flooded regularly yet it is not protected to a very high standard. With respect to Bull Island, it could be considered that the problem would be more of coast protection in nature, however, a number of recent studies to monitor the condition of the island would tend to indicate that the front face of the island is accreting rather than eroding. This can only be of overall benefit to the island. Nevertheless there are also a number of properties such as golf club houses and more particularly at the Bull Wall end, which could be at risk of flooding, see Photographs A22 and A23.

iii) Clontarf, Bull Wall to East Wall – Photographs A24 –A29 & Figures 1.5 & 1.6

A number of locations along this section of the study area were flooded, with the worst flooding occurring at the western end of Clontarf Road, where the road and a number of properties were extensively flooded. The nature of the coastline along this section consists for the majority of a vertical wall structure fronting a promenade type recreation area, see Photographs A24 and A25. Over much of this length there are additional secondary walls which vary in height and are located at varying distances back from the primary defence and which will act to trap flood water within the promenade area, see Photographs A24 to A27. At the western end the primary wall reduces in height and is replaced by a low rock revetment structure, Photographs A28 and A29 and Figure 1.6. This is clearly a vulnerable spot and it is likely that flood water weired directly over the revetment at this spot, indeed examination of photographs provided by local residents would tend to indicate this, see Section 3.3 for details. Flooding also occurred by water overtopping the wall at other locations further east, and it is thought that this was mainly due to wave overtopping. This can be confirmed following completion of the topographic survey and modelling aspects of the study. Problems also occurred in this location due to gaps in the secondary wall which exist for vehicular and pedestrian assess and also at bus shelters, see Photographs A26 and A27. Along the Alfie Byrne Road the coastline comprises a steep revetment backed by a grassy area, and further towards East Wall, a high earth bank. There were no reports of flooding along this length.

iv) River Tolka, East Wall – Photographs A30 – A32 & Figure 1.6

Photographs A30 and A31 show the stretch of the River Tolka immediately downstream of Annesley Bridge, which is the upper limit of the study, see Figure 1.6. A short distance downstream of the railway bridge shown in Photograph A31, the Tolka discharges into Dublin Harbour at the East Wall Business Park bridge. There were no reports of flooding along this section of the river, with the exception of some very localised flooding around a foot bridge immediately upstream of the railway bridge shown in Photograph A31. From the pictures it can be seen that the river walls are quite high, although it is likely that the February event must have come close to the top. However, there is noted to be extensive vegetation growing out of the bank walls, which could be of concern. The River Tolka Flooding Study Report undertaken as part of the Greater Dublin Strategic Drainage Study (GDSDS), suggests that there are no problems downstream of the Distillery Weir, although they do indicate that given sea level rise predictions the levels of the walls could become marginal and would require assessment. Between the John McCormack Bridge and East Wall Business Park Bridge, extensive reclamation works are underway as part of the Dublin Port Tunnel project. This work, whilst reducing the channel width at that particular location, has not reduced the width of the channel to a lesser extent than a number of existing restrictions located further upstream, see Photograph A32.

v) East Wall and Royal Canal – Photographs A33 – A36 & Figure 1.6

This area was quite badly flooded by water that escaped from the Royal Canal. At North Strand Road water initially over flowed both the left and right banks at the location of the lifting bridge, see Photograph A33, and then later as the tide rose over a considerable length of the canal banks. To the right the water escaped through a gap in the boundary wall and into Shamrock Cottages. To the left the water flooded the CIE land. Following flooding of the CIE land to the left, failure of a boundary wall (Photograph A34) caused by retention of this floodwater, resulted in extensive flooding of a small industrial estate on Ossery Road. In addition water escaped through a number of openings in the boundary of the CIE land and flooded an extensive area around Hawthorn Terrace and also Irvine Terrace. Details of the flood paths through those boundary openings is discussed later in Section 3.3. While the majority of the CIE boundary consists of wall structure, they are not flood defence in nature. In addition a number of opening exist some of which have been sealed temporarily and some permanently. Examples of these openings are a pedestrian access gate, now sealed, and palisade fencing, see Photographs A35 and A36. In addition to those areas that flooded, recent discussions with Kirk McClure Morton who are undertaking a separate flood assessment of the Royal Canal, highlighted an additional low lying area to the west of the canal around Seville Place, although this area was not affected in February 2002. This area has been investigated further in this project and has been shown to be at risk of flooding, see results of flood risk assessment presented in Chapter 15. Due to the extensive area of the CIE land and the uncertain nature of the boundary walls the best policy to adopt for this area is the prevention of flood water entering the canal at the source. Nevertheless it is felt that some canal bank strengthening and raising works should be undertaken in the short term and even as part of any proposed long term floodgate option.

vi) River Dodder – Photographs A37 – A46 & Figure 1.7

The River Dodder is tidal as far as the Ballsbridge Weir and flooding occurred in its lower reaches between London Bridge and Ringsend Road Bridge, see Photograph A37. Flooding occurred over both the right and left banks. To the left the flooding escaped at a low spot in the bank into an adjacent building site from where it later escaped and caused flooding in an area adjacent to the greyhound racetrack, see Photograph A38. To the right the river channel consists of masonry quay walls and water overtopped these and caused quite extensive flooding of a low-lying area in the region of Stella Gardens, see Photograph A39. This flooding was severe, as the land to the rear is considerably lower than the riverbank levels. In addition retreat of the flood water level in the river resulted in failure of a length of the bank, see Photograph A40, which shows the repairs to that failure. Upstream of London Bridge the river channel continues to consist of masonry quay wall on the right bank and low masonry walls, natural earth banks or a combination of both on the left bank, see Photographs A41 to A44. Over much of the masonry wall channel sides, there is considerable vegetation growth, which could cause structural problems in the future. The Ballsbridge Weir, immediately upstream of Ballsbridge, marks the upstream extent of the River Dodder within this project and is shown in Photographs A45 and A46.

vii) Sandymount Strand to Merrion – Photographs A47 – A52 & Figure 1.7

At the northern end of this section high water levels combined with wave action guided by a vertical wall at this location, resulted in a concentration of water at the junction of Sean Moore Park with Beach Road. Overtopping of the wall occurred causing flooding of the road and Marine Drive, see Photograph A47. In additional a number of pedestrian accesses in the sea wall at this location helped the release of water into this area, see Photographs A48 and A49. Over the reminder of the Strand Road frontage, there exists a car park and promenade area between the beach and the road. Over much of this length the promenade is fronted by a sandy beach and a low level revetment, see Photograph A50 and A51. A low masonry wall exists to the rear of the promenade along Strand Road over much of this length. However numerous gaps exist in the wall for access. It is likely that the revetment, which is low, was extensively overtopped flooding the promenade which in turn allowed flooding of Strand Road through the gaps. At Merrion a gap at the location of the railway crossing allowed high water combined with wave action to cause flooding of the road and railway line at this location, see Photograph A52.

viii) Grand Canal – Photograph A53 & Figure 1.7

The Grand Canal is tidal over a short length to the first lock structure. No records of flooding were highlighted for this area, however a number of properties line either side with windows backing onto the canal bank, see Photograph A53.

ix) River Liffey – Photographs A54 – A58 & Figure 1.8

The River Liffey is channelised over much of its length between the port and Island Bridge Weir. Through the centre of Dublin the channel sides consist of high stone walls, see Photographs A54 to A57. These walls were not over topped during the February 2002 event and consequently extensive flooding was not reported along the tidal length of the river. However, some flooding of the quays near the Guinness Brewery was recorded although this is believed to have been due to water backing up unflapped outfalls and not due to overtopping of the quay walls. In addition part of the board walk (Photograph A58) along the river wall was flooded and water levels came close to the pedestrian access points, which have been cut through the Liffey walls, although no flooding through these gaps was recorded.

x) Dublin Port Area North Side – Photographs A59 to A62 & Figures 1.6 & 1.7

The north side of Dublin Port consists of the main commercial port area. The Dublin Port Company reported that there was very little to no flooding on their land during the February 2002 event, although the water level did come within 0.5m of the top of most of their quays. However the implications of sea level rise may give rise to concerns for this area in the future. Much of the port area consists of quay related structures through the centre of the port and along the lower length of the Liffey, see Photographs A59 and

A60. On the northern side of the port facing Clontarf, the land water boundaries generally consist of revetment type structures, see Photographs A61 and A62.

xi) Dublin Port Area South Side – Photographs A63 and A64 & Figure 1.7.

The southern side of Dublin consists of some commercial port operations, but also two ESB power stations and DCC sewage treatment works. As with the north side there were no reports of flooding in this area during February 2002. Along the Liffey the frontage consists of container and dry bulk quays along the inner end and a number of jetties, intakes and outlets over the power stations and treatment plant frontage. Along the southern side the frontage consists of rock revetment structures of varying conditions along most of the frontage from the start of the Great Southern Wall back to Sean Moore Park, see Photographs A63 and A64.

2 FEBRUARY 2002 EVENT

2.1 General Description

On the 1st February 2002 an exceptionally high tide occurred at Dublin, which resulted in significant flooding throughout parts of the city and Fingal. The period around the end of January and beginning of February 2002, was a period of spring tide conditions associated with the full moon at that time. The highest tide predicted for Dublin port around that period was on the Thursday 31st January at 13.12 hours and was predicted to be 4.46m LAT (1.95m ODM). The actual tide recorded at that time was 4.69m LAT (2.18m ODM). The next highest tide was predicted for 14:00 hours on the 1st February at a level of 4.44m LAT (1.93m ODM). The actual highest level that occurred around that time was 5.46m LAT (2.95m ODM) at 14:30 hours. This was some 1.02m higher than the highest predicted value around that time based on the Dublin Port Tide Tables. Later analysis of the tidal records and predictions of the astronomical tide for that day based on this analysis would tend to indicate that the astronomical tide level was slightly under predicted in the port tables and that the surge was closer to 0.96m, see Chapter 9. Figure 2.1 shows a comparison between the predicted tide for Dublin and the recorded value for 31st January and 1st; 2nd & 3rd February 2002. The surge residual shown is the difference between the two and represents the observed surge for Dublin Port. A value of just over 1m can clearly be seen on the 1st February 2002 and is based on the actual less the Dublin Port tide tables predicted astronomical tide for the day.

Figure 2.2 shows the peak of the recorded tide in Dublin Port. It can be seen from the figure that there were two significant peaks. The first, having a level of 5.42m LAT, occurred around 13:40 hours, after which the tide level fell to around 5.28m LAT at about 14:08 hours, before rising again to its highest level of 5.46m LAT at 14:28 hours. Figure 2.2 also shows that the tide rose above 5.0m LAT (2.49m ODM) at around 12:43 hours and stayed above that level until 15:07 hours, a period of almost 2.5 hours.

From Figure 2.1, it can be seen that the peak of the surge event coincided more or less with the peak of a relatively high spring tide, and it is certain that this has resulted in such extreme combined tide levels. Whether a 1m surge is of itself a significantly unusual event is dealt with in the probabilistic analysis reported in Chapter 9, however, its coincidence with the peak of a spring tide would result in a significantly more unusual combined event.

The mechanisms resulting in such a surge in tide level are not fully clear although a better understanding is enabled as a result of the analytical work undertaken in Chapter 9. From the description of the weather conditions given in Section 2.2 around the time of the flooding, it is extremely likely that the surge component was driven by the extreme low pressure system which had been formulating to the northwest of Ireland over a period of days before the event. It was this low pressure system, which fell to a low of just 930mbar which was the driver for the high surge levels experienced at Dublin, even though the pressure at Dublin was in the region of 986mbar. The continuous low pressure system in the Atlantic generated conditions which propagated into the Irish sea in the form of a surge wave. This combined with strong winds, which around the 1st February were up to gale force from south to south westerly, increased the sea level significantly.

Rainfall and river flows were not significant contributing factors, see Section 2.4 below for description of conditions around that period.

2.2 Weather Conditions

The following description of the weather conditions for the 24-hour period, preceding and including the events of 1st February 2002, has been supplied by staff at Met Eireann from their Monthly Weather Bulletin No's 189 and 190, covering January 2002 and February 2002 respectively.

2.2.1 The General Situation

For several days preceding the 1st February storm surge event, there was an area of low pressure near Ireland causing stormy weather. On the 1st February a very deep depression (Figure 2.3) with a central pressure of 930hPa (930mbar) passed to the north west of the country. Bands of heavy and thundery rain, together with southwesterly gales, affected most parts of the country during the day (Figure 2.4). However, the most disruption was caused by the combination of very low pressure and the exceptionally high tides measured around the coastline, especially in the Irish Sea.

The worst affected area was Dublin, where severe flooding occurred after the highest tides measured in over eighty years caused sea defences to overtop and rivers and canals to overflow. Structural damage was also recorded as a result of the exceptionally high tides.

2.2.2 The Detailed Meteorological Situation

On Thursday 31st January 2002 a frontal trough crossed Ireland as a deepening depression approached from the Atlantic. Cold overnight with slight ground frost and a few showers, mainly on the northwestern coasts. A spell of heavy and persistent rain moved into the southwest early in the morning and spread across the country during the day, clearing to scattered showers and short sunny spells in the afternoon. Light to moderate southerly winds overnight became strong to gale force later with some severe gusts during the day.

Rainfall:	5mm – 10mm at many stations, with 17mm – 23mm in south and southwest.
Temperature:	maximum 9°C - 12°C, minimum 2°C - 6°C. Ground temperatures down to –4°C in Kilkenny.
Sunshine:	Nil – 1hour.

On Friday 1st February 2002 a storm depression to the northwest of Ireland moved steadily northeastwards, as its associated frontal trough crossed Ireland during the day. Winds were up to storm force at times, while rain was heavy in places, especially in southern areas. Rain cleared to showers later, some thundery, as winds eased slightly. Mild throughout.

Rainfall:	10mm – 20mm in many areas, over 30mm in south and southwest.
Temperature:	maximum 11°C – 13°C, minimum 4°C – 8°C. Ground temperatures 2°C – 7°C.
Sunshine:	Nil – 2hours.

2.3 Marine and Irish Sea Conditions

Data has been obtained for the Marine Institute M2 buoy which is located in the Irish Sea just off Dublin Bay. The buoy provides data on a number of meteorological and marine parameters, the main ones of which are atmospheric pressure, wind speed and direction, wave height and period. Data on the 1st February 2002 indicates that from about 0700 hours to 1500 hours significant wave heights were in excess of 3.5m and reached up to 4.4m on a number of occasions. The buoy does not record wave directions, however, this is likely to be similar to the wind direction around the time, which was predominately from a southerly direction. These wave heights, while not very extreme, are not insignificant.

The M2 buoy is located some distance offshore and therefore the recorded wave climate is likely to reduce as it propagates towards the coastline and into the bay. This is due mainly to wave refraction as the wave fronts interact with the seabed, wave breaking over the Kish and Burford banks, and shoaling as waves traverse the nearshore bathymetry. Nevertheless the conditions on that day are likely to have resulted in a nearshore wave climate that would have caused problems at a number of locations, particularly over the peak of the extreme tide, when water depths and hence wave heights would have been at their greatest.

2.4 Fluvial Conditions

Rainfall and river flows were not significant contributing factors to the flooding on the 1st February 2002. Rainfall data obtained from Met Eireann for Dublin Airport shows that only 1.8mm of rain fell in 3.5 hours on the 31st January and 10mm in 3.4 hours on the 1st February 2002. Such a low rainfall would not have contributed to the flooding on the day. Moreover, records for the month of January 2002 do not indicate any periods of intense or prolonged rainfall which would have contributed to significant runoff or higher than usual river flows.

For the Tolka, data from the gauge at Botanic Gardens indicates a mean daily flow of 4.54m3/s and a maximum daily flow of 8.26m3/s at 11:00 hours on the 01/02/2002. These values are not considered to be significant and indeed discussions with MCOS in respect of the Tolka Flooding Study, would suggest that these are of base flow magnitude. For the Dodder, data for the Waldrons Bridge gauge indicates a mean daily flow of 5.32 m3/s and a maximum daily flow of 10.5m3/s at 12:15 hours on 01/02/2003. Again this value is not considered significant in light of the extreme flow estimates presented in a number of study reports produced for the Dodder and chapter 13 of this report.

For the River Liffey no flow gauge just upstream of the tidal reach is available and so an accurate account of the flow through the centre of the city on that day is not known. However, a reasonably accurate estimate can be made based on information from gauge sources further upstream and other Liffey tributaries. Information obtained for the Liffey and its tributaries included daily average flow data for,

- Leixlip Dam 24.0 m3/s
- Rye Water 7.5 m3/s
- Griffen 0.7 m3/s

31.2 m3/s

This value for a river the size of the Liffey is not significant and indeed when compared with the design flow estimates presented in chapter 13, it is much lower than a 1 year flood event.

Therefore it can be concluded that rainfall and/or river discharges were not a significant contributing factor to the flooding that occurred on the 1st February 2002.

2.5 UK Met Office Surge Model

The UK Met Office has operated a version of the 2-dimensional storm surge model since 1978. The model was specifically designed to aid the prediction of storm surge and flood warnings around the coast of the UK. The model has been regularly updated, most recently in 2001.

The surge model[1] covers the area of the NW European Continental Shelf from 12oW to 13oE and 48oN to 63oN with a grid resolution 1/9o latitude and 1/6o longitude – a resolution which equates approximately to a grid 12½km x 12½km.

The model calculates storm surge elevations at the centre of each grid and horizontal and vertical water velocities at the centre of each boundary cell. The model is driven by winds and barometric pressures derived from the National Weather Prediction Mesoscale model (the atmospheric model from which weather predictions are made).

The surge model is run four times each day to generate tidal and surge elevations. For each prediction the model is run first without wind forcing effects to establish just the tidal elevations and then with the wind forcing effects to produce the total elevations. The difference between the two elevations is the surge elevation.

The model will produce reliable forecasts of surge levels in water that is less than 100m in depth which is the case through out the area covered by the Dublin hydrodynamic modelling.

The UK Met Office has provided data from their storm surge model to enable validation of the tidal hydrodynamics model work to be undertaken as part of this project. For more details of this particular task, refer to Chapter 12. Output from the surge model has been purchased for the periods 31st January 2002 – 1st February 2002 and 10th – 11th March 2001. The data from the first has been used to ensure that the tidal model is able to reproduce the water level that led to the flooding on 1st February 2002. The second data set has been obtained to validate the calibration of the hydrodynamic tidal model. This is described in detail in Chapter 11.

Figure 2.5 shows a plot of the bathymetry of Dublin Bay and surrounding area and also the location of UK Met Office node point which is used within the study and for which the

[1] McArthur J (2001). Comparison of Shelf Seas Model and Surge Model water level predictions. Ocean Applications Internal Paper No. 38.

above data has been obtained. The predicted tide curve and surge elevation for these two nodes have been plotted on Figure 2.6 and compared with the measured tide curve at Dublin Port. While the two nodes are some distance offshore in relation to the Dublin Port tide gauge and hence there would be some distortion as the tide curve propagates into the bay, the comparison is nevertheless good. It can be seen that the model was predicting a 1m surge more or less coinciding with the peak of the actual tide curve.

The UK Met Office has also provided a series of power point plots from their surge model showing the development of the surge water level in the period leading up to and after the high tide experienced on 1st February 2002. Figure 2.7 is a reproduction of the surge predictions for 1400hrs and 1500hrs respectively from a model run at 00:00 hours 31st January 2002. These plots show that for an area around the Dublin region of the Irish Sea, the predicted storm surge was between 0.8m – 1.0m.

In Figure 2.8 a comparison of the UK Met Office Shelf Seas and Storm Surge Models at the two node points is presented against the predicted and observed tide and observed surge at Dublin Port. Whilst the UK Met Office predicted surge elevations are not identical to the observed at Dublin Port, they are reassuring similar. Whilst the observed is less smooth, the peaks are located in relatively similar locations and are of relatively similar order of magnitude.

The above observations provide encouragement in the proposed methodology which intends to use this surge model as the basis for establishing a forecast system for Dublin through the current study.

3 FEBRUARY 2002 FLOODED AREAS

3.1 Introduction

Using reports compiled by Fingal County Council and Dublin City Council staff on the flooding around the time of the 1st February 2002 event, and through subsequent discussions with those staff, a number of areas have been identified as having been subject to some form of flooding or disruption. These flood areas are described below. It should be noted that the text below and the flood areas identified on the figures presented in Appendix C, represent those that are understood to have been affected during the February 2002 event and are not thus indicative flood risk maps. The areas affected include:

3.2 Fingal County Council

3.2.1 Portmarnock to Baldoyle

i) Coast Road

- The road and roundabout at the junction of Coast Road with New Road and Strand Road Portmarnock, was flooded, although no properties were flooded at this location. The problem here has been reported as being due to drainage from the fields which can become tide locked where the outfall discharges into the estuary. As a consequence the water backs onto the low-lying road and roundabout. However, during a site inspection a low spot was noted where the masonry wall meets an earth embankment south of the roundabout on the Coast Road. This may have been a contributing factor to the flooding that resulted at this location on the 1st February 2002, see Figure C1.2 and Photograph A5. In October 2004 this location was again flooded. Inspections during that flood event indicated that the road flooded before escaping through the gap mentioned above. This would tend to indicate that either the water was escaping through the masonry boundary wall or backing up through the outfall at this point, or some combination of both. Nevertheless, all three mechanisms mentioned above will need to be address if future flooding is to be alleviated.

- A stretch of the Coast Road south of the Mayne River was flooded. The mechanism of flooding along this location was not confirmed, however the road level at this location appears low and at one spot is protected by a gabion wall with a concrete post fence, see Photograph A7. A survey undertaken by Fingal County Council (FCC) after the flood indicated the highest point along the gabion wall to be around 2.7mODM with the road being lower at around 2.5mODM. The tide level recorded in Dublin Port was 2.95mODM and therefore it is highly probable that the water simply weired onto the road at this point and flowed back along the road towards the Mayne River, see Figure C1.2 and Photograph A7. Having said that the road was again flooded at this location in October 2004, albeit to a lesser extent and on that particular occasion water did not over top the gabions. Inspections during that flood event revealed that water was also seeping through the old masonry wall and therefore the condition of the wall also required careful attention as well as the defence level.

- Two properties, Nos 1 and 2, were flooded on Coast Road. These properties are located adjacent to the flooded section of road mentioned above, see Figure C1.2. While it is likely that water may have entered the properties from the road, the FCC survey also indicated the ground levels along the rear of these properties to be low in places. It is therefore likely that water flooded them from a number of directions. In addition there were reports of effluent in the flood water which could have been due to water backing up their septic tanks.

3.2.2 Sutton

i) Burrow Road

- Flooding was noted on Burrow Road in the region of the Avalon Apartments. Whilst a number of gardens were flooded, no reports were received that properties were flooded along this road. The water is believed to have penetrated the dune system and flowed down an access road leading from the beach on to Burrow Road, see Figure C1.3.

ii) Sutton Dinghy Club

- The Sutton Dinghy Club was extensively flooded. The club is relatively low lying and the flooding resulted primarily from the collapse of a boundary wall fronting the club, see Figure C1.3.

iii) Strand Road

- Strand Road was flooded for some distance from its junction with Greenfield Road.
- Property No's 2 to 18 had their gardens flooded but it is believed the properties themselves did not flood.
- Property No 1 was flooded in addition to its gardens.

The extent of the above flooding is presented on Figure C1.3.

The frontage at this location is relatively exposed to wave activity. A block and concrete wall exists along much of this frontage, however there are several openings along the wall to allow pedestrian and vessel access to the foreshore, and indeed, the wall at one of these access points was damaged during the February event. This has since been repaired, see Photograph A14. In addition to the access points, a number of drainage holes exist at regular intervals along the wall. Towards the junction with Greenfield Road, the wall ends and is replaced by a lower bank armoured with building rubble. This bank is much lower than the top of the block wall. It is likely that the main source of flooding at this location was due to wave action through the openings in the wall and over the bank. The gardens of the properties along Strand Road are lower than the road and so the road was the main flood defence element along this length.

iv) Greenfield Road & Sutton Cross

- Property opposite the church, was flooded following failure of the property seawall.
- Other properties on the seaward side of Sutton Cross had their gardens flooded.

The flooding along this location is shown on Figure C1.3. Only one property was flooded following collapse of the boundary wall that backs onto the foreshore. The flooding mechanism along this frontage is primarily as a result of wave action being driven up the beach on the enhanced tide. The location is still relatively exposed, being just outside of the protection offered by Bull Island.

3.3 Dublin City Council

3.3.1 Dublin City Council – North City Flooding

v) Clontarf Road

Flooding occurred along this road at three locations, see Figure C1.4. These include,

- A length of road adjacent to the junction with the Alfie Byrne Road. As well as flooding of the road, a number of properties were flooded along this length, see Photograph C1 and C2 and flood map Figure C1.4.
- A length of road adjacent to the Clontarf Baths.
- A length of road adjacent to Bull Wall.

Flooding at the first location, near Alfie Byrne Road was principally as a result of water overtopping the low-lying primary sea defence, i.e. where the concrete promenade wall is replaced by a much lower revetment structure. Water at this location weired directly onto the linear park causing extensive flooding. This is evident from Photograph C2, where no trace of the revetment can be seen beyond the flooded cars. However, a short distance to the east Photograph C1 clearly shows the top of the promenade wall. A number of properties were flooded at this location and the situation was made considerably worst by passing vehicles, see Photograph C3.

Flooding at the latter two locations was as a result of waves overtopping the higher promenade seawall. The exact reasons for overtopping at these locations are not fully clear. However the baths project perpendicular from the promenade wall and this has resulted in the build up of a shingle beach to the eastern side which could have aided the run-up of waves that were concentrated at this point. Near the Bull wall initial visual observations indicate that bed levels are slightly lower, thus allowing larger waves to reach the wall. A more likely mechanism in this location is that wave action is focused along the Bull Wall and concentrated in the corner, resulting in increased sloshing at that point. While the road was badly flooded at these latter two locations, there were no reports of significant flooding in any premises.

Further on site observation during the flood event of October 2004, when flooding of parts of the promenade and Clontarf Road occurred again, revealed that the mechanisms mentioned above do play a part in the overtopping of the walls. However, it also revealed the importance of another mechanism that of Mach Stem wave overtopping. This occurs when incident wave direction to a structure (mainly vertical or near vertical structures) is between 20 and 40 to the perpendicular. Under these conditions the wave can run along the length of the wall building in height before overtopping in large quantities. This was very evident along the Clontarf Frontage from between Vernon Avenue and The Clontarf Baths. West of the baths the waves were considerably lower and to the east of Vernon Avenue the waves overtopping wave also quite significant but due more to “normal” overtopping and not Mach Stem.

3.3.2 East Wall – Royal Canal

i) Shamrock Cottages

- The floodwaters escaped from the Royal Canal at the location of the lift bridge and began to flood the CIE land over the right bank of the canal, see Figure C1.5 and Photograph C5. As the water level increased, it escaped through a low spot in the CIE boundary wall, into Guilford Place and down to Shamrock Cottages. A significant number of properties were flooded in this area up to a depth of 1m. Photograph C7 shows part of the clean up operation.

ii) Ossary Road

- The floodwaters escaped from the Royal Canal and entered CIE property over the left canal wall. Photograph C4 shows the floodwater moving up the railway track on the north west side of Strand Road. Initially the floodwater came through the lift bridge and as the water level continued to rise it overtopped the left bank of the royal canal for some considerable distance, see Photograph C5. This added extensively to the volume of water in the CIE land and caused the collapse of a boundary wall immediately downstream of the Dart Railway Bridge along Ossary Road. As a consequence of the collapse there was extensive flooding of the Ossary Road Business Park, with water reaching up to 1.5m deep in places, see Photograph C6.

iii) Blyth Avenue and Adjacent Area.

- The flood water that entered Blyth Road and surrounding area again originated from CIE land which had been flooded from the Royal Canal, see Figure C1.5. In this case the water made its way into Blyth Ave. through an access door in the CIE boundary wall, see Photograph A35 which shows the subsequent sealed door. In addition a palisade fence a short distance further along the boundary wall, also allowed floodwater to escape into this region. The flooding extended to many roads beyond Blyth Ave. and included, Church Street, West Road, Church Road, Caloden Road, St. Mary's Road, Moy Elta Road, Fairfield Avenue, Killan Road, Russell Avenue and Hawthorn Terrace. All of this area is extremely low and most was flooded to depths in excess of 0.5m.

Discussions with relevant stakeholders highlighted the above as the main sources of flooding in this East Wall region. However, discussions with a local resident of West Road at the time of recent site inspections have identified what could have been an additional flood path into this area. On the day of the flood, water was observed flowing down West Road from the northern end which, it was suggested, had come through the railway underpass to West Road from Ossary Road. This is at the opposite side of the flooded area to Blyth Ave. From the observations that day it was not clear how the water could have accessed from that direction.

Later inspections of the CIE land and discussions with their representatives indicated that water had made its way down the railway line between Ossary Road Business Park and the flooded residential area, to a low spot between West Road Bridge leading to the underpass and the Dart railway line bridge. From the on site inspection it is clear that water would then have entered an unused car park to the right of that track that exits at the underpass. A manhole cover level was checked at the underpass and found to be 1.68m ODM, which is considerably lower than the flood level and so it is very probable that this was indeed an additional flood path. Furthermore the industrial estate which lies opposite the location of the collapse wall at Ossary Road, backs onto this railway line and car park. It was noted that its boundary with the rail line consists of old corrugated iron sheets with many holes, and so a direct flood path through this estate is likely to have been an additional direct route. Indeed it is highly likely that the flood path(s) described above were as direct and equally responsible for the flooding of this area as those at Blyth Ave. This possible flood path is highlighted on Figure C1.5.

iv) Abbercorn Terrace and Irvine Terrace

- Water escaped from the CIE land through the CIE boundary wall on the opposite side of the track at the site of the sealed doorway, see Figure C1.5. The boundary fence consisted of a palisade fence set on a very low wall. The flood water escaped into both Abbercorn and Irvine Terraces. Approximately 20 properties were flooded in this region to a depth of 0.5m.

Initially the primary blame for the flooding in the CIE land and adjacent areas was placed with the lift bridge. However, while this was indeed likely to have been where flooding of the CIE land commenced, it is unlikely that sufficient water could have escaped through this location to cause the extensive flooding of the CIE land and the adjacent areas. Recent inspections of the lift bridge would tend to further support this idea. It is believed that while flooding commenced at that location, as the water levels rose they overtopped a considerable length of the left bank of the canal, considerably increasing the volume of water within the already flooded CIE land. Indeed Photograph C5 in Appendix C would tend to confirm this, as no sign of the top of the canal wall can be seen. When the flood water eventually subsided the considerable volume of water within CIE land was held there by the canal walls for some time after, all the while continuing to discharge into the flooded areas mentioned above.

3.3.3 East Wall - Other

i) East Wall Road

- There were reports of some localised flooding around the footbridge across the River Tolka adjacent to the East Wall road. However this was not extensive and did not flood any properties, see Figure C1.5.

3.3.4 River Liffey – Left Bank

i) North Wall Lighthouse

- It was reported that the North Wall Lighthouse and tip of the North Quay extension at Alexandra Basin, were flooded. This flooding was not extensive and the extent is shown on Figure C1.8.

ii) Quays at Spencer Dock

- It was reported that some slight localised flooding occurred over the Liffey Quay walls at the entrance to the Royal Canal. This was reported to be only around 30mm deep, see Figure C1.5.

iii) Boardwalks

- A number of the boardwalks between O'Connell Bridge and Gratten Bridge were flooded. The flood water came to within 100 mm of the lowest access gap in the Liffey quay walls but no flooding of the quays occurred, see Photographs C11, C12 and C13 and Figure C1.6.

iv) Wood Quay

- Some localised flooding was reported along Wood Quay and this was generally thought to be as a result of water backing up through drains, see Photograph C9 and C10.

v) Wolfe Tone Quay

- Flooding to a depth of approx. 300mm occurred on the road along Wolfe Tone Quay, see Figure C1.7. It is believed that this come through openings or gullies in the quay walls and was not as a result of overtopping of the quay walls, see Photograph C8. These openings have since been sealed.

3.3.5 Dublin City Council – South City Flooding

i) River Dodder - Stella Gardens

- Water overtopped a considerable length of the right bank of the River Dodder causing extensive flooding of the Stella Gardens and surrounding areas, which are at a much lower level than the river, see Figure C1.8. This was the most extensive flooding on the southern side of the city, with floodwaters in excess of 1m deep along many of the roads and within a significant numbers of properties. The main areas flooded included, Fitzwilliam Quay, Dermot O'Hurley Avenue, Aikenhead Terrace, Irishtown Road, St. Brendans Cottages, The Square, Summerfield and Fitzwilliam Walk. Photograph C16 shows also the flooding of the basement car park at the Fitzwilliam Quay Apartments. When the peak of the flood water receded, part of the right bank collapsed, see Photographs C14 and C15. Fortunately this collapse did not result in a complete breach through this section of the river bank, otherwise much more extensive and severe flooding could have occurred. This breach has since been sealed with sheet piling.

ii) River Dodder - South Lotts

- Water escaped from the River Dodder through a low spot of the left bank just upstream of the Ringsend Bridge and into an adjacent building site, Photograph C17 shows the location. From here it made its way into South Lotts Road and down to South Dock Road, Doris Street and Gordon Street, see Figure C1.8. Approximately 40 properties were flooded to a maximum water depth of around

400mm. Since the event a concrete patio wall has been constructed as part of the development, however a small low spot still exists between the earth bank fronting the dog track and this wall which could leave the area vulnerable to further flooding.

iii) River Dodder - Newbridge Avenue

- Just downstream of New Bridge on the River Dodder adjacent to Lansdowne Road, water escaped through a number of viewing gaps in the wall which are located on the right bank within a small park. The water flooded a small area of Newbridge Avenue and connecting laneways, flooding a number of garages, gardens and approximately 4 properties, see Figure C1.8. The flood water reached a maximum depth of about 450mm. At the time of writing these gaps had not been sealed.

3.3.6 Sandymount

i) Marine Drive and Drummond Avenue

- At the location were Beach Road meets the Sean Moore Park, high water levels combined with wave activity resulted in overtopping of the seawall and the escape of water through a number of gaps in the seawall at this location, see Figure C1.9. The Beach Road was flooded, which then flowed into both Marine Drive and Drummond Avenue, both of which are at a lower level. These gaps have since been blocked, however access will need to be restored in the future. Both roads were flooded to depths of about 600mm and in all approx. 20 properties were flooded. It is believed that waves are the predominant mechanism for flooding at this location. It is likely that waves tend to run along a vertical wall at this location when the tide is in and become focused into this corner. Ground level information obtained following completion of the topographic survey will help to confirm this risk. If this is revealed to be the case, then in conjunction with improved access openings, a possible option would be to reduce or disrupt the wave action before it reaches this location. This could be achieved by construction of a number of fishtail groynes perpendicular to the wall and a detached low level breakwater. This would have the effect of breaking up the waves and would also encourage the build-up of material to enhance the beach at that location which would further promote wave breaking.

i) Beach Road and Strand Road

- The majority of Strand Road and Beach Road between Merrion Gates and Marine Terrace was flooded, see Figure C1.9. Over much of this length the frontage consists of a revetment structure fronting a promenade, with car parks with a secondary wall along Strand Road. At the Beach Road end the frontage consists of a vertical wall. Over the complete frontage there are 18 openings in either the vertical or secondary wall, leading onto Beach and Strand Roads (not including the two at Marine Drive and the one at Merrion Gate). Waves would have overtopped the revetment causing flooding of the promenade before escaping through these gaps onto Beach and Strand Roads. At the vertical wall frontage on Beach Road, access steps lead down onto the beach and these cause a protrusion which extends perpendicularly out from the wall on the seaward side. It is likely that this protrusion would catch wave action resulting in the projection of water vertically and cause overtopping of the wall. Indeed this mechanism has been noted even at times of lesser tidal elevations. Photograph C18 shows one of these opening sandbagged, but some spray and water can still be seen. After flooding the main Strand and Beach Roads, the water then extended down a number of side street including, Seafort Avenue, Newgrove Avenue, St John's Road, Sydney Parade Avenue, Gildford Road and St. Alban's Park. In all around 20 properties along the coast were flooded directly.

3.3.7 River Liffey – Right Bank

i) The East Link Bridge

- Flooding occurred on the road on the southside, east of the Toll booth, see Figure C1.8. At the peak water depths reached 400mm.

ii) Sir John Rogerson's Quay

- There were reports that the Sir John Rogerson Quay experienced minor overtopping of the quay wall, see Figure C1.8. The local drainage system was confirmed to have adequately coped with the resulting flooding.

iii) Victoria Quay

- Flooding occurred on the road along Victoria Quay resulting in the quay being closed to traffic for a period of time. While the water level came close to the top of the quay walls, it is believed that the main floodwater came through openings or gullies and was not as a direct result of overtopping of the walls, see Figure C1.7.

3.3.8 Merrion

i) Merrion Gates

- Flood water due to high tide and driven by wave action escaped through an access gap at the Merrion Gates and onto the Road and Dart line, see Figure C1.9. The Dart was closed and at its peak the flood water reached a depth of around 600mm at the gap and 1.2m on the lowest spot on the road. The gardens of about 21 properties adjacent to the area were flooded but the number of properties which were directly flooded was kept to two thanks mainly to the release of water into manholes by a DCC gang who were working nearby.

4 DATA COLLECTION

4.1 Introduction

One of the fundamental requirements of work within Phase 1 of the DCFPP was the collection and collation of reports and data from past and existing projects and studies that might have relevance to the project. This background information is an essential part of the project process, and often provides the study team with a wider appreciation of the important issues as well as an understanding of what further investigations and/or additional data would facilitate the progress of the project. In reality the collection of data has continued throughout the project.

Dublin City Council began this process at an early stage prior to the award of the contract, with the provision of a table of known data, Table 8.1, forming part of the Request for Proposal (RFP). The information in “Table 8.1”, as it has continued to be known, has formed the basis for further data collection within the project. An extensive list of additional data was collected during Phase 1, and this data has been recorded and presented in a follow-up table – “Table 8.2”. The data has been obtained through consultation with all the major stakeholders for the study and contains details of historic and current projects in the form of drawings, reports, digital data, etc.

Both Table 8.1 and Table 8.2 are presented in Appendix D. The tables have been enhanced to include a review section and provide brief comments on the mains points of interest. Details of the main elements of data and their benefit within the DCFPP are discussed in Section 4.2 and 4.3 with respect to Tables 8.1 and 8.2 respectively.

4.2 Proposal Data – Table 8.1

Table 8.1, in Appendix D.1, comprises data that was complied at the proposal stage of the project by Dublin City Council. The table has been set up to summarise each document or piece of data including the title, author, and client along with further contact details as appropriate. As part of the project it was necessary to review these documents to decide their relevance to and highlight the important issues. The results of this process are summarised in the last three columns of Table 8.1.

Much of the data collected includes bathymetry, photographs, initial flood reports from February 2002, digital maps, data and drawings, collected through liaison with the stakeholders involved with the Project. It is considered that this system, together with the brief comments presented in the table is sufficient to indicate the relative significance of each to the project.

Many of the reports and maps in Table 8.1 provide information on the flooding that occurred on 1st February 2002. This information has been used extensively to build up the flood maps that have been produced and are presented in this report.

In addition there is much data of relevance to the numerical modelling work that has been used to validate the modelling process and results. This data includes reports on the February 2002 event, water level, wave data and previous studies on the Rivers throughout Dublin.

4.3 Post-Project Award Data – Table 8.2

Table 8.2, in Appendix D.2, has been compiled using the same format as in Table 8.1. The information in Table 8.2 has been collected throughout the project, through a series of joint and individual meetings with the main stakeholders. As with Table 8.1 all the documents have been summarised, reviewed and commented on with respect to their relevance to the project. The data in Table 8.2 comprises a wide variety of information and includes wind and tide data, bathymetric and survey data, reports and studies, development and policy plans, photographs, maps, hydrological data.

Several of the documents obtained are studies and reports that address the effects of flooding of the three rivers within this study, namely the Dodder, Liffey and Tolka. In particular a number of recent studies have been completed for the Tolka. Of particular relevance are the data and results available from the River Tolka numerical modelling study, whose results have been used in the assessment of areas at risk. The integration of these results with the findings of the DCFPP study will ensure a holistic approach to flood alleviation for Dublin City.

During the construction of the Dublin Bay Pipeline project extensive pre and post construction bathymetric surveys were carried out. These surveys together with those made available from the Dublin Port Company, and the survey of the foreshore undertaken specifically for this study, have been of significant help in creating the numerical wave and tidal hydrodynamic models. The records of the tide gauge at the Lighthouse have been of direct relevance to the probabilistic analysis of tides and meteorological surges. Chapter 9 contains a more detailed discussion of the latter.

In addition interviews with stakeholders have led to a wider appreciation of the issues relevant to identification and development of strategy and policy guidance. Chapter 16 details the outcome of these developments and summarises short, medium and long term policy and strategy objectives.

Since completion of the Phase 1 report (for project management purposes the study was separated into 4 phases) significant amounts of additional data have been collected and used within the project. Table 8.2 has been updated to catalogue all of this additional data. The data collection exercise within the project has continued right up to the end of the project and indeed new sources of data are been made available to the project even as this report is being completed.

Through Table 8.1 and 8.2, a complete record of all the data obtained, researched and used within the project has been and is continuing to be kept. The tables will form in effect a "Table of Contents" for that data library which can then be easily retrieved to the source and furthermore used to justify its use or otherwise within the project.

4.4 New Surveys

4.4.1 Commissioning New Surveys

Whilst a substantial body of information was available to the project team (cp. Table 8.1 and 8.2 in Appendix D), there was a need for further topographic and bathymetric surveys to augment the existing information. One of the principal objectives of the survey was to obtain digital data from which to produce accurate and reliable plans of the existing seabed and surface features, and to provide base data for the construction of numerical models.

Once the existing information had been collected and reviewed, the scope of the topographic and bathymetric surveys was set. The areas identified topographic and bathymetric surveys of:

a) the River Dodder between the River Liffey and Ballsbridge Weir;
b) the River Liffey between Sean Heuston Bridge and Islandbridge Weir;
c) Royal Canal between the River Liffey and Strand Road; and
d) a topographic survey of the land water interface.

Where the frontage consists of a hard defence, the surveys captured levels at the landside toe, the crest and the seaward toe, where it was safe and practicable to do so. Along the rivers the survey also included the riverbanks, crest elevations and promenade/road levels.

As a consequence of the increasingly widespread use of remote sensing techniques such a LiDAR (Laser induced Direction and Range) for the collection of survey information, the tender documents were drafted in a manner that would permit tenderers to submit alternatives based on a remote sensing approach, if this could be shown to offer advantages to the project. Tender documents were therefore prepared and issued on 9th July 2003 covering the above scope.

Tenders were invited from six survey companies:

i) BKS Surveys Ltd – Coleraine, N. Ireland;
ii) Ois Surveys Ltd – Stockton-on-Tees, UK;
iii) Land Surveys – Dun Laoghire, Ireland;
iv) Longdin & Browning Surveys – Stevenage, UK;
v) Deep Surveys – Amsterdam, Netherlands; and
vi) Infoterra Surveys – Leicestershire, UK

Following initial pre-tender discussions with several of the survey companies above, an initial return date of 17th July was set. This was subsequently extended to 23rd July 2003 following requests from tenderers for more time to prepare their bids.

4.4.2 Topographic Survey

A topographic survey was conducted using real-time Digital Global Positioning Systems (DGPS). Surveys of the coastal roads, strategic roads in the surrounding areas, and the port area, were carried out using a car-mounted DGPS module. Where the survey was extended to include coastal defences, details such as crest elevation, toe level, slope, beach and foreshore profiles, and promenade levels etc., a hand held DGPS unit was employed.

The DGPS network was linked initially to seven new Permanent Ground Markers (PGMs), although this was later increased to ten with the inclusion of three passive Ordnance Survey stations in Malahide, Howth and Dun Laoghaire to facilitate greater accuracy across the network.

4.4.3 Bathymetric Survey

Cross sections were taken along the River Dodder at a minimum of 100m between centres. At bends in the river, the frequency of the cross sections was increased to 50m between centres. Bank levels were taken, as well as bridge soffit levels and profiles.

The River Liffey was well served by surveys from the port upstream to the Sean Huston Bridge. Between the bridge and the Island Bridge weir, a new survey was required.

5 ASSET CONDITION SURVEY

5.1 Structure of Asset Condition Survey

An important element of any coast protection or flood alleviation study is a detailed knowledge of the condition of the coast protection or flood defence assets. These assets can be either natural or man made. In addition to acquiring data for site specific projects, the monitoring of their condition is also an important element in the long-term management of these assets and the coastline and it is important to have continuous records to aid this management process. To that end Haskoning has developed an Asset Condition Survey approach for a number of their clients, which collects these records and presents them an easily retrievable and user friendly database system thus promoting simple and effective management of their assets.

This Asset Condition Survey approach has been used for the project area within the Dublin Coastal Flooding Protection Project and will aid the appropriate assessment of the risk from coastal flooding arising from the poor or deteriorating condition of the assets. This information in the form of a database has been passed to Dublin City and Fingal County Councils to aid them in the future management of their coastline. The Asset Condition Survey database was an early deliverable of the project. Since its completion it has been used to identify several areas where priority should be given to the upgrading or maintenance of the existing defences.

The Asset Condition Survey format consists of the,

a) collection of relevant data through site inspections of the project area;
b) classification of coastal areas around the coastline and discrete defence units within those areas;
c) entry and storage of recorded data in a suitable database; and
d) preparation of a user manual to facilitate use of the database.

The site inspection surveys consisted of walking the coastline fronting the study area, the three rivers (Liffey, Dodder and Tolka), the canals, and categorising the area into discrete defence units. Each defence unit was split according to type and or condition. For instance one unit may include a rock revetment, whilst the next may include a sea wall. This method of splitting the lengths of coast and rivers assets means that each defence unit is assessed on its merits, whilst the consequences for work and alteration are considered in a holistic frame of reference with respect to management of the whole coastline.

Each defence unit has been assigned a specific identification number and a complete range of data has been collected for each which includes:

- Location
- Type and function
- Essential elements, e.g. quay wall, revetment etc.
- Length
- Description
- Condition, e.g. Poor-Fair-Good.
- Residual Life, e.g. <5 year, 5 -10 year, >10 year.
- Digital Photograph

- Sketch.
- Details of other features, e.g. outfalls, floodgates etc
- Nature of hinterland.
- Nature and condition of foreshore.
- Level, to be added on completion of topographic survey.

The above information aids the assessment process in respect of their condition and in making valued judgements regarding work required.

From a management perspective the database may be used to include information on any essential repairs or routine maintenance, and documentation of expenditure on any stretch of defence. The database can then be checked and updated at regular intervals to ensure that regular maintenance, if required, is undertaken and a history of the work recorded.

5.2 Information Collected

For the Dublin Coastal Flooding Protection Project, the asset survey inspections were undertaken during May and June 2003 by Haskoning Engineers experienced in this type of inspection.

During these inspections, the project area was categorised and data as detailed in Section 5.1 above was collected for each defence unit identified. In addition any immediate or long-term actions have been identified, prioritised and recorded. The purpose of each element or defence, and the assets it protects are also recorded.

5.3 Presentation of Data

The Asset Survey Database for Dublin City and Fingal County uses a Microsoft Access based system design by Haskoning for asset management. The database has been set-up so that it can be used as a tool to monitor the condition and function of the river and coastal defences in the Dublin and Fingal areas, assisting in the management of both individual stretches and the coastline as a whole. A user manual for the Asset Survey Database has been prepared and a copy given to DCC and FCC with the installation of their databases.

The information collected is entered into the Defence Details screen shown on Figure 5.1. This screen gives a general overview of the information available for each defence unit.

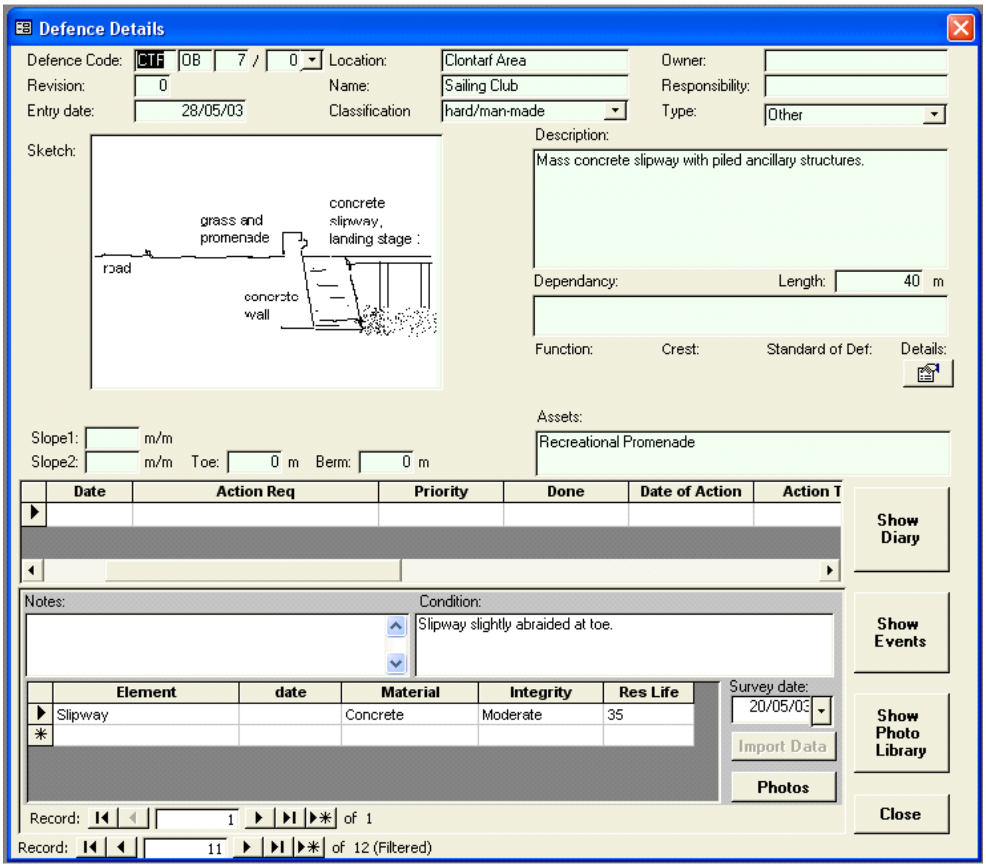


Figure 5.1 - Defence Details screen

From Figure 5.1 it can be seen that the main interface of the database presents a range of data for each element including a sketch and description of the specific defence. The condition, substance and residual lifetime of each element of the defence is briefly described. Overall this is intended to give a basic understanding of the form of defence, its condition and the assets it protects. Notes can also be made within the database to identify any work or maintenance that may be required to improve the condition and stability of the defence. Using this tool, work on the defence can be prioritised.

From this screen access is available to a number of other windows that provide further information. One of these is the Photograph library, see Figure 5.2, which allow the user to view the specific asset thus providing a visual aid to enhance the detail already provided and for the identification of a specific type of defence and its surroundings.

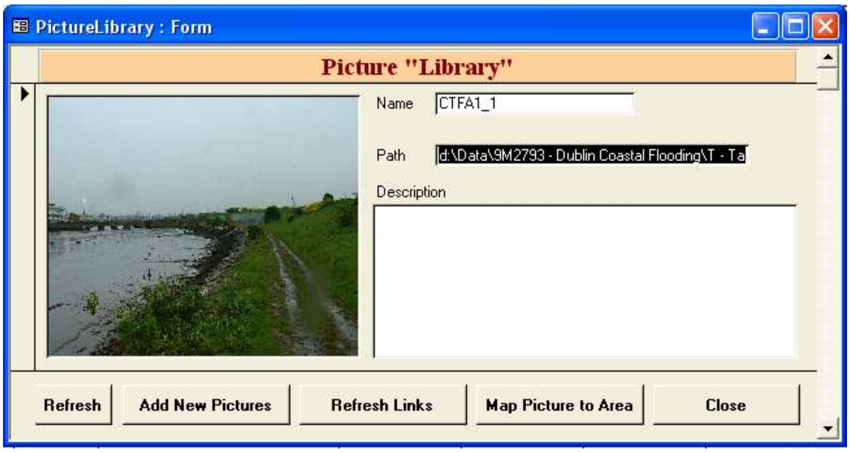


Figure 5.2 - Photograph library screen

Once all the data has been entered different reports can be obtained, for future surveys and to assist in the management of the assets. These reports include:

General Overview

This is essentially a report showing all the information that was initially included into the database. Allowing easy reference for future surveys and revisions to the database.

Detailed Action

This report highlights the types of works that are required and their priority to maintain a good defence mechanism. The work can be clarified into individual sections for the scheduling of any such works.

Diary

This report allows the user to input and classify any maintenance or capital works that have been carried out, see Figure 5.3. This function can also be used to highlight studies in the area, or revisions to the asset survey that have taken place. Keeping the database up to date is essential.

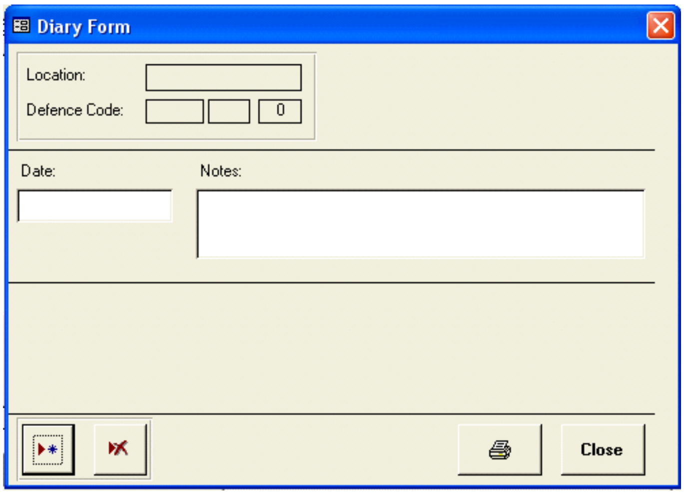


Figure 5.3 - Diary form screen

Ideally the database should be used as a working document, collecting the history of each defence to provide a record of how the condition and type may change and to include maintenance and repairs details such that the information can be used as an aid to justify future capital expenditure.

Screen dumps showing the “Defence Details” for as selected number of the defence units where actions are highlighted are also presented in Appendix E2.

The database was used to identify a list of actions across the study area and an emergency works report was produced to address the most urgent of these actions. A table showing these actions is presented in Appendix E1.1 and E1.2, for DCC and FCC respectively, and details of the works to address these actions is included on Chapter 17.

5.4 Extension of Asset Management Database

5.4.1 Background

Following the completion of the Asset Management Database for the Dublin Coastal Flooding Protection Project, it was agreed that the database should be extended to cover the remaining length of the Fingal County Council (FCC) coastline, providing a full definition of the coast from Sutton through to the Balbriggan.

The main coastal inspection was undertaken in October 2003, with further re-examination of critical areas being undertaken during November. Each defence type encountered during the inspection was photographed. A series of maps showing the defence type and length accompanies the database.

A key issue in extending the database was the manner in which it would be maintained and updated. Whilst it is clearly important for information to be shared between Dublin City Council and Fingal County Council, it nevertheless more appropriate for each authority to have a separate database covering their own area of operational management. In line with this decision, information relating to FCC’s southern coastline between Sutton and Portmarnock, initially included within the Dublin asset database, was extracted and combined with that data held in the new, extended database. The complete database was installed on FCC's computer system in May 2004.

Clearly the two databases (that covering the Dublin frontage and that for Fingal) are totally compatible, and it has been recommended that as information is updated by each authority with respect to their own individual areas, that data is exchanged as a means of maintaining that important awareness of each authority's management practice.

5.4.2 Inspection of the Fingal County Coastline

A thorough visual surface inspection was undertaken of all structures on the coastline, as well as all soft frontages. Only in areas where there was a continuation of hard cliffs or in areas where it was impractical to gain access to a frontage due, for example, to access being limited even at lower states of the tide, was the coast not inspected in detail. In such areas, and as far as was practicable, visits were made to various points along the frontage to ensure that there was no significant variation.

Although the visual inspection was thorough, it represents only a snapshot of the condition of the defences at the time they were conducted. Assessment of condition and performance of structures or systems (such as dunes) are solely based on experienced coastal engineering judgement. In particular, it has to be appreciated that estimates of residual life of a structure, which depends both on future maintenance and on an understanding of the specific rate of deterioration of a structure, is still only an estimate. As the database is used and updated, the quality of the initial assessment of the residual life is improved. Over time a picture emerges of how the individual structures perform and where resources will be required for their management.

The database has allowed an assessment of the composition of the coastal defence assets. This is summarised in Table 5.1 below.

Table 5.1 - Summary of the Fingal Coastline

Principal Features	Number of individual sections	Total length. km.
Soft natural coastline (dunes, marsh)	77	32.7
Soft managed coastline (managed dunes)	27	4.2
Hard natural coastline (hard cliffs)	34	20.0
Manmade defences (walls, revetments)	342	30.3

The inspection included the open coast and all estuaries up to the effective tidal limit.

The coast has been divided into general areas for convenience of locating and then into individual units along the coast. The division of units is primarily made on the probable need for different management practice. For example, generally divisions have been made with respect to different structures i.e. seawall, revetment or embankment, or different materials used in construction such as masonry, concrete, or gabions. In addition, where evident and appropriate, the divisions reflect different exposure conditions or, apparent rates of deterioration.

6 PUBLIC INFORMATION CAMPAIGN

6.1 Format of Campaign

Being able to engage with the population of Dublin City Council and Fingal County Council in a positive and productive manner was an early objective of the Dublin Coastal Flooding Protection Project. The trauma suffered by members of the public during flood events such as February 2002, can only be overcome by a combined programme aimed at:

a) raising the awareness of the public to the risks posed by flooding;
b) promoting the options available to protect properties at risk;
c) raising awareness amongst the public of the responsibilities of the City and County Councils;
d) promoting dialogue among residents groups to hear their concerns and expectations; and
e) promoting the project.

The initial format proposed for the public information campaign consisted of a number of elements and included,

- Leaflet Survey
- Web-site
- Public Information Campaign

The construction of a web site was an early deliverable, as was the leaflet campaign, although delivery of the information leaflet and questionnaire was delayed until early 2004. Examples of the leaflet and questionnaire are included in Appendix F.

6.2 Leaflet Survey

A public information leaflet was prepared and delivered to members of the public via a leaflet drop during April 2004. The leaflet was accompanied by a questionnaire, the purpose of which was to gauge the public perception of the emergency response in relation to the February 2002 event, as well as to obtain additional confirmation of the details surrounding that event. Offering a prize draw encouraged public response. A copy of the leaflet and questionnaire is included in Appendix F.

6.3 Web-site

Although the content and format of the web site was substantially completed early on in the project, delays were experienced as a result of technical concerns raised by Ordnance Survey Ireland (OSi) on the use and integrity of maps and aerial photographs.

Access to the site itself is via the web address http://www.floods.eu.com . It is intended that the site will become part of a wider portal addressing flood mitigation initiatives undertaken by Dublin City Council and Fingal County Council.

6.4 Public Information Meetings

Wider consultation with members of the public will be considered once the development of options has reached the stage whereby comments can be invited in a constructive manner. The format of the meetings is currently under discussion, although an open forum or exhibition is considered to offer the most flexible approach. Meetings will be scheduled in the four main areas.

In addition to the proposed open exhibition format, meetings have been held with residents at Sandymount and Ringsend on 1st and 2nd June 2004 respectively. The meetings were arranged by Dublin City Council in response to the concerns raised in the completed questionnaires received by DCC. The meetings were well attended and have provided a valuable means of engaging the public in the consultation process.

6.5 Responses

Responses to the questionnaire were received by Dublin City Council over the months April and May. A total of approximately 1900 leaflets and questionnaires were delivered. From this a total of 292 questionnaires were received. This represents a return of 15%.

An analysis of the questionnaires has been undertaken by Dublin City Council. The results have been summarised and are included in Appendix F.

6.6 Workshops & Stakeholder Meetings

A series of workshops were arranged as part of the strategy to keep stakeholders informed of progress. The workshops were also arranged in such a manner as to engage the stakeholders in debate and discussion about pertinent issues relating directly to the study.

The workshops were held on the following dates:

- Workshop No1. – 5th September 2003
- Workshop No2. – 19th February 2004
- Workshop No.3 – 27th April 2004
- Workshop No4. – 21st September 2004

A summary of each of the workshops is provided below.

6.6.1 Workshop No.1

The morning session of the workshop focused on the initial findings of the DCFPP team, and included Project Familiarisation, Data Collection and initial modelling. This was done in the form of a presentation of the Phase I Report. The presentation also highlighted the areas that where known to have experienced flooding as a result of the February 2002 event.

In the afternoon, the workshop turned its attention to a discussion of three questions posed by the project team; namely:

Question 1 - What are trigger criteria? The consensus among was that triggers can take several forms. These would be criteria such as the high tide levels, fixed defence levels, fixed levels above which warnings/actions are implemented, flows into Ringsend Treatment Works and different events/conditions that 'triggers' actions.

Question 2 – What factors would influence a trigger criterion? Factors that may influence a trigger(s) might include scale of consequence and its location, the condition of sea/river defence, whether strategic infrastructure is affected, excedence and joint probability of predefined safety thresholds, insurance impacts, social and personal disruption and the resources available for the event. These factors are closely linked to the flood maps.

The above are only a summary of the factors considered and a wider discussion also took place as a consequence. It was generally accepted that whilst it was possible to manage some of the risks, not all could be wholly removed or mitigated. This led to the conclusion that a maintenance and capital works programme should recognise this when drawing up budgets.

Question No.3 – Of the above criteria, which are the most important? The workshop discussed this question from the perspective of the needs, expectations and modes of the future potential Early Warning System. The group considered that the most important factor would be education; both of the Council operations staff and of the members of the public. It was agreed that the programme of education should take place before any warnings are issued i.e. before any EWS goes live.

Other points that were raised as a result of the general discussion suggested that:

- A crisis plan should be drawn up to inform emergency services and general public.
- All parties that are involved in building and using the EWS should practice.
- Public awareness should be raised through public exhibitions, the use of the media, incorporating responses from the media in how warnings should be presented.
- The early warning system should match the resources that are available through a phased response and staggered lead in times, to allow the emergency services to plan their responses earlier and to adapt to the changing situation.

6.6.2 Workshop No.2

The second workshop focused on the development of the elements of a Flood Warning and Flood Forecasting system and introduced the experience gained from the UK Environment Agency flood warning system that was set up in the late 1990's.

The initial presentation highlighted the main and important components of any flood warning system i.e. detection, flood forecasting, warning, dissemination and response. It also showed how the responsibilities and roles had been developed pre and post 1995.

To date the Environment Agency (EA) has published performance standards for a two hour warning, but the operational lead in time for coastal warnings is significantly more than two hours. This warning also depends on the nature of the event. With tidal and some fluvial events, a much greater warning time can be, and is given. However, on steep rivers which respond very quickly to short, high intensity bursts of rainfall, the warning is generally not better than two hours.

The response to this presentation and the short presentation made by Mr Wass (an invited speaker) of the UK Met Office, generated considerable constructive discussion.
An up-date on the progress of the Flood Policy Review undertaken by the Office of Public Works was presented by the representative from OPW. The Flood Policy Review report was published in November 2004. Recommendations include:

- a more strategic and proactive approach to flood management;
- control of future risk, through Flood Hazard Mapping and development planning;
- raising awareness of flood risk;
- the use of structural measures for high level risk;
- an emphasis on non structural measures for lower levels of risk;
- public preparedness through a campaign of education implementation of flood warnings.

The remainder of the workshop involved the attendees moving into three smaller groups to discuss the four out of five important elements that form an Early Warning System, which are Flood Forecasting System, Flood Warning, Information Dissemination and Response.

Some of the important parts of a flood forecasting system included a 24 hour system operation with a 24 hour information and helpline. This would require good lead in time accuracy through good interpretation of the data available, allowing allocation of proper resources in the potential Flood Risk Areas that have been identified beforehand.

The flood forecasting system leads directly to issuing some form of flood warning. This warning will be part of an action plan drawn up prior to such and event, and would include a response team, using the critical elements and resources from local council organisations, the emergency services, local area wardens. Each of these will be given a specific form of the flood warning and each of those involved will have been trained so that each person can exert a certain responsibility when needed.

However the information from the flood warning will need to be disseminated in some form to specific agencies, the emergency services and the general public. This can be done using the local and national media, in the form of television and radio warnings, 24 hour telephone helpline, dedicated alarms, church bells and community Wardens.

It is essential that prior to any event that a public awareness and education campaign is undertaken forming flood partnerships between different hierarchies within the flood warning chain. That way ensuring that at each level the responses and feedback will be communicated. This will be essential during the feedback period after the potential or actual event.

Once the warning has been issued the response to it can be in several different ways, the response can be low, medium or high impact. Sometimes monitoring the early warning can be sufficient, issuing further warnings as the event pans out. Communication is the key element allowing a co-ordination of the response or levels of response.

Prior to any event taking place a response plan should be set up so that each responder knows their level of response, should a warning be issued. This can include local authorities, emergency services, infrastructure agencies, and community responsibilities.

6.6.3 Workshop No.3

The third workshop was undertaken as a result of the second focus group. One of the elements discussed in the second focus group was Policy and Strategy. It was felt by those who attended that further discussion was required to investigate the current flooding policies in place and the future policies that would be required as part of the flood protection strategy that would need to be built up as a consequence of the Dublin Coastal Flooding Protection project.

The initial session of the workshop was a presentation explaining the process of disseminating the information from current Policy and Strategy using International Best Practice documents. The presentation focussed on how the documents available highlighted the policy areas that are currently missing and would need to be addressed at some level, be it national, regional or local.

From all the International Best Practice documents four main areas were selected and their policy were extracted from them; these included the scope of policy on flood protection and prioritisation of Strategy for Coastal Defence, MAFF, UK, 1993. The second was the institutional and legal frameworks of Fundamentals on Water Defences, Technical Advisory Committee on Water Defences, from the Netherlands, 1998.

Thirdly the development and risk of Planning Policy Guidance Note 25 (PPG25):, from the Office of the Deputy Prime Minister, UK, 2000 in addition to the planning, precautionary principle and sequential testing. Finally the National Flood Insurance Program from the United States of America focussing on the relationship with insurance companies through an integrated approach.

Using these fundamental policies as guidance a ‘wish list’ of policies was drawn up. These were then categorised to show how well they had been documented to date. In respect of a communications policy there is often public awareness and consultation during schemes, however there is no clear policy that strategises the processes that are needed for communication at all levels.

As a regional perspective shoreline management plans and catchment management plans have been set up, but need to be collated and co-ordinated nationally. Climate change policies are available, though they are mainly aimed at rainfall, sea level rise, and as these areas are fairly recent, the policies are fairly vague and require a more detailed look over time.

At present in Ireland there is no planning for development policies available, within the public domain. This policy will need to address the importance of flood plains, precautionary principles and sequential testing, leading to a risk based approach for future developments. Again with flood warning systems, flood forecasting there are no clear policies, but it is an area that is beginning to pique interest and whilst local actions may take place during emergency situations, there is no national government or local authority policies available. Also included in this policy could be guidance for building developments that are wholly or partly flood resistant. This would significantly reduce the risk of flooding in these built up areas.

The policy for flood defence measures can be spilt into a number of areas, these could include urban sea defence, urban river defence, rural sea defence, existing rural river defence and drainage schemes. However, as part of the flood defence measures policy, new rural river defence and drainage schemes will need to be taken into account, so that inadvertently there is no increase to the existing areas at potential flood risk.

Further policies on the existing maintenance of flood defences and monitoring of water levels can potentially feed into the flood warning system and thus become part of that particular policy. Also policies on management and funding were included under post project evaluation, along with research and development.

One area that requires careful negotiation is the relationship with insurers. Potentially insurers can legally stop flood damage insurance cover or raise premiums. Therefore policy on legal obligations need to be dealt with through an integrated approach with either national government or local authority

Prior to setting up new policies or reviewing existing policies a consultation needs to be undertaken with the major parties involved with policy, at local government level as a minimum. These bodies can include DCC, Fingal CC, Dublin Port, Dept. of the Marine, OPW, etc, but this is not considered an exhaustive list. These bodies, along with others need to ensure the implementation of emergency response procedures through local or national policy.

As part of the GDSDS study policy guidance regarding new development; environmental management; inflow, infiltration and exfiltration; climate change and use of basements are expected, and can be reviewed as part or the stakeholder policy consultation mentioned above.

The policies above were classified into 4 different categories:

Category 0 - no defined policy or documentation
Category 1 - some form of policy but not formally documented
Category 2 - some form of documented policy
Category 3 - comprehensive policy that is fully documented

It was felt that none of the policy areas have been fully documented, whilst some are in a state of flux, either being reviewed, or reliant on a review pending. The majority of the present polices were in category 1. Much of the work was undertaken but without a formal policy.

Two of the major policy areas which are of concern to the DCFPP are that of planning of developments and flood defence. To date both of these policies are classified in category 2. It is felt that in respect of the DCFPP these policies should be taken forward prior to another major flood event occurring in the future.

In regard to a planning policy for Dublin those attending thought that the policy guidance should include planning control in respect of flood risk and that there should be recognition of uncertainty. There should also be an obligation on authorities and developers to check whether a proposed development will create/increase potential flood risk even in areas which are not currently at risk from flooding. In addition the policy may need to be a regional level, with guidance at a national level, filtered and detailed further as it reaches a local level.

Regarding the policy statement on Flood Defence for Dublin the workshop considered the policy should focus in respect of protection of life (first priority) and then property. Indicative priorities for protection (e.g. existing urban development in high risk areas, new development in low risk areas), will need to be set up. Also the policy needs to encourage the promotion and undertaking of works within a broad geographic context (catchment, coastal cell) and be mindful of protecting the environment.

6.6.4 Workshop No.4

The fourth workshop was aimed at arriving at a list of factors that needs to be considered in the assessment of the options.

Again work was carried out in separate groups. In the following the results are given of each group.

Group 1		
	1	Benefit / Cost ratio
	2	Aesthetics - Landscape architecture
	3	Robustness
	4	Assets at risk / population at risk
	5	Ancillary uses - integrated into the community
	6	Environmental statement / EIA
	7	Adaptability / Flexibility
	8	Maintenance & Operational Costs
	9	Time to Construct
	10	Time to respond to an emergency (ref Demountables??)
	11	Policies - should it ever be considered.
	12	Decision support models to evaluate options - multi criteria decisions support
	13	Resources are the internal resources adequate (see also 8)
	14	Land Ownership - Responsibility Transfer
	15	Disruption

Group 2	
IMPACT Factors	Life at Risk (e.g. basements) Properties at Risk Strategic Assets At Risk No Prop Unit Value Time to Evacuate & Nos (NB)
COMMUNITY & PUBLIC STAKEHOLDERS Factors	Lobbying by Area / Representatives / Stakeholders History of flooding Regional Competition for Funds Capitalise on Other Advantages Insurance Industry view Part of Integrated policies
ENVIRONMENTAL Factors	Environmental Issues Visual Impacts
DELIVERY OF SCHEMES	Time to deliver (Statutory/ Legal / Procurement issues) Actual Delivery (Progressive Improvement) Cost of Proposals Cost benefit

Group 3		
System:		*Pros / Benefits* Compensate for, or eliminate Negative impacts. Economies of Scale Reduces Disruption Amenity / Environmental Enhancement
		Cons. / Disadvantages Distracts focus from the main problem (either flood alleviation or amenity schemes) Additional Costs Additional implementation timescale (public consultation / statutory permissions) Health & Safety Confusion of Public Added environmental impacts e.g. reclamation!
Conclusions:		
	1	Added Value should be sought
	2	Don't compromise the solution or primary scheme objective
	3	Seek to minimise & optimise all other disadvantages.

Based on the previous an overall checklist was derived regarding factors that need to be considered in option assessment. The list comprises the following:

OVER RIDING PURPOSE	
	Assets at risk / population at risk
Primary Defence	History of flooding Areas at Risk Standards of Defence Life at Risk (e.g. basements) Properties at Risk Strategic Assets At Risk Level of Protection Decision support models - primary coastal / estuarine fluvial Time to respond to an emergency (ref demountables)
Flood Emergency Management	Don't compromise the solution or primary scheme objective Time to Evacuate & Nos (NB) Robustness Disruption
VALUE ADDED	Full Life cycle costing Cost sensitivity Benefit / Cost ratio Ancillary uses - integrated into the community Maintenance & Operational Costs Added Value should be sought Capitalise on Other Advantages Seek to minimise & optimise all other disadvantages.
ENVIRONMENTAL	Environmental statement / EIA requirements Environmental Issues Aesthetics - Landscape architecture Visual Impacts Adaptability / Flexibility
BUILDABILITY	Time to Construct Time to deliver (Statutory/ Legal / Procurement issues) Actual Delivery (Progressive Improvement)
MISCELANEOUS	Land Ownership - Responsibility Transfer Policies impacts Resources are the internal resources adequate Lobbying by Area / Representatives / Stakeholders Regional Competition for Funds Insurance Industry view

7 OVERVIEW OF INTERNATIONAL BEST PRACTICE

7.1 Introduction

Throughout history, flooding has represented a threat to both urban and rural economies alike. The instances of flooding are on the increase, as are the damages suffered as a consequence. Recent examples include the severe flooding in Prague in 2002, in the UK during Easter 1998 and the autumn of 2000, in Dublin on 1st February 2002, and Moray in Scotland in 1997 and 2002.

In the Republic of Ireland, responsibility for flooding policy falls to the Office of Public Works. The Republic of Ireland government conducts its statutory responsibilities through the Office of Public Works under the Arterial Drainage Act 1945 in respect of river drainage and flood relief. The Arterial Drainage Act 1945 was amended in 1995 to allow the Office of Public Works to address localised flooding problems particularly in urban areas.

The implementation of policy in England and Wales is through the Environment Agency via a programme of capital, maintenance and operational schemes. They also have a responsibility to provide advice to planning authorities relating to flood risk. In Scotland, the local authorities undertake this function, whereas in Northern Ireland, the responsibility rests with the Rivers Authority. In addition to the Environment Agency, internal drainage boards and local authorities have delegated powers to carry out flood defence works on watercourses and coastlines.

By contrast to both Ireland and the UK, flooding policy in the Netherlands has developed out of a need to protect the low-lying land from inundation by the tide. With almost 65% of the country at risk from daily inundation coastal defences are, as a result, designed to a 1:10,000 year return standard. The economic consequences of flooding are potentially disastrous. Responsibility for the development and implementation of flooding policy at a national level rests with the Rijkswaterstaat (Ministry of Public Works). In the Netherlands the standards of defence are set down in legislation. It is the responsibility of the local water boards to raise funds for the maintenance and operation of the defences. However, if the standard of defence is raised, then under current policy arrangements the Rijkswaterstaat will provide capital funding to meet the costs of improving the defences to meet the new standards. Thereafter, maintenance is devolved down to the local water boards. More recent developments are aimed at moving the consideration of defence standards from a deterministic approach to one based on a reliability approach.

7.2 Towards the Development of Best Practice

Best practice in Europe and in the UK has developed in response to the unique pressures that exist in each country and the needs to deal with the increasing incidents of flooding. A review is given here of information that is of relevance to the Dublin Coastal Flooding Protection Project. These are necessarily focused on practices outside Ireland, since it is these practices which will inform the development of flooding policy for Dublin City Council.

The purpose of the review given here, is to identify elements of international practice that will:

a) facilitate the strategic appraisal of flood mitigation options;
b) inform the development of policy and strategy within Dublin City Council; and
c) provide the foundation for the long term development of an early warning system.

7.3 Overview – Key Documents

As part of the review of International Best Practice enquiries were made of both stakeholders and those within Haskoning. These enquiries were to ascertain what reports and documents regarding International Best Practice were available.

Certain documents reviewed will undoubtedly prove more useful within the Dublin Coastal Flooding Protection Project (DCFPP). In the light of this a summary table has been set up in Appendix G. The summary table has been used to highlight those texts that are considered to be most useful for the project, as opposed to those which have historical or anecdotal references. A brief summary of some key documents is given below.

The Benefits of Flood and Coastal Defence: Techniques and Data for 2003 - Flood Hazard Research Centre, Middlesex University.

The manual centres around ways of expanding on advice given in the Flood and Coastal Defence Project Appraisal Guidance Notes (FCPAGN) described below. Using the original methods from the preceding manuals: Benefits of Flood Alleviation (Blue), Urban Flood Protection Benefits (Red) and The Economics of Coastal Management (Yellow). The 'Multicoloured Manual' expands and updates the Cost Benefit Analysis (CBA) methods, and uses more recent data to guide the user to a more informed decision.

The manual explains in detail the types of land use and how each use and value may influence the way that CBA may be assessed and how the final Cost Benefit Ratio may be viewed. It does not however give examples of fluvial and coastal flood alleviation options. The manual gives an objective view on how potential value could build up, and how this data may be assessed against the cost of land loss and schemes.

This report is useful to the DCFPP because it gives guidance on the uses and relevance of cost benefit analysis data, including new methods and will assist in the Benefit Cost calculations that will need to be undertaken later in the study. Also it enables the reader to use a methodical approach to decisions made on strategies and schemes.

Flood and Coastal Defence Project Appraisal Guidance (FCDPAG) – DEFRA, MAFF.

The Flood and Coastal Defence Project Appraisal Guidance (FCDPAG) aims to provide best practice advice to those involved in the preparation of strategies and schemes. This guidance encourages high quality of decision-making supported by a rigorous of options so that the most appropriate scheme or strategy is proposed.

The guidance is set up into different manuals, enabling each one to read individually or as part of a process. These are:

FCDPAG1 – Overview

This document provides little detail but is a good starting point for those wishing to use the FCDPAG Manuals, but have no former experience of them.

FCDPAG2 – Strategic Planning and Appraisal

This report approaches flood and coastal defence schemes from a strategic route. This includes identification of problems and key issues, establishing the aims and objectives of the specific scheme, gathering data in the form of consultation, surveys, etc, appraising scheme options using economic, technical and environmental criterion, selecting the preferred strategy or policy for the area, then compiling and recommending a plan.

FCDPAG3 – Economic Appraisal

This report uses the methods of Benefit-Cost analysis to guide the user into undertaking an economic evaluation of any options that may have been highlighted through and studies or projects involving flood alleviation.

FCDPAG4 – Approaches to Risk

This guide explains what is considered a risk and how best to evaluate and address the risks involved in any flood alleviation scheme. It sets out clearly the methods of identifying potential problems/risks and how best to overcome them.

FCPAG5 – Environmental Appraisal

This publication seeks to advise on how to take account of environmental objectives and sustainability of scheme designs. To improve project appraisal by drawing attention to the different techniques available for environmental evaluation, including monetary and non-monetary.

These report are useful to the DCFPP because they provide a baseline to the methods that will be used throughout the project. This guidance provides a step by step approach to each section of strategies and appraisals of options that may come from the protection project.

Planning Policy Guidance 25: Development and Flood Risk – ODPM, DEFRA

Planning Policy Guidance 25, National Planning Policy Guidance 7 – Planning and Flooding, and Technical Advice Note 15 - are planning policy manuals that have been put together by the England, Scotland and Wales planning departments respectively. The idea behind each is to provide guidance on the use of flood plain areas and areas prone to flooding in the development of urban conurbations.

These documents summarise the responsibilities of various parties in the development process, encouraging where possible schemes that reduce flood risk. They intend that flood plains be used for their natural purposes, continue to function effectively and are protected from inappropriate development. The guidance also outlines how flood risk issues should be addressed in regional planning guidance, development plans and in the consideration of planning applications.

At the time of writing the Irish planning guidance on developing in flood plains was under development by the Office of Public Works and central government and was awaiting approval. A review of the recommendations of this guidance will be undertaken and included within the final report.
These reports are useful to the DCFPP because they will give planning guidance on development in flood plain areas, and how the potential flood risk areas can be avoided. The methods used will develop policies for Dublin to highlight areas that could be at continued flooding risk within the Dublin project area.

Flood forecasting and warning best practice - Baseline Review. RD Publication 131 – DEFRA & EA

This document was written as the baseline information for the Environment Agency's 'Flood Forecasting and Warning Research Programme'. It provides information to support development of the flood forecasting and warning service, guidance for the policies and programme of the National Flood Warning Centre (NFWC). This study expands the knowledge gained by the Easter Flood Actions (EFAs) and Changing Needs in Flood Defence Review (CNFDR).

This report can be used in the Phase 3 flood-forecasting element of the DCFPP. It will provide assistance in setting up flood forecasting and early warning systems to reduce the risk of flooding in Dublin.

UK Climate Impacts Programme 2002 Climate Change Scenarios: Implementation for flood and coastal defence: Guidance for users. R&D Technical Report W5B-029/TR - DEFRA & EA

This research report provides guidance on the use of UK Climate Impacts Programme (UKCIP) climate change scenario information within the flood and coastal defence community of England and Wales. It refers to input and derived hydraulic parameters within coastal and rivers and the economic decisions involved therein.

This report takes the data originally recommended by DEFRA and reviews this data in the light of the UKCIP02 information helping organisations to assess how they might be affected by climate change, so they can prepare for its impact.

It offers information in extreme sea level rise, extreme rainfall, windspeed, adjustments in Mean Sea Level, etc.

This document will be useful for the modelling part of the DCFPP.

Arterial Drainage Act 1945

This Act set out the roles, responsibilities and procedures necessary for the implementation and subsequent maintenance of a scheme relating to the drainage and improvement of land by the execution of works of arterial drainage.

It is important to know the existing legislation within Ireland and understand what procedures need to be adopted. Whilst the Arterial Drainage Act does not appear to specifically cover tidal or coastal flooding, it may be applicable in tidal sections of the Dublin Rivers and thus may warrant consideration. Furthermore future strategies and policies for Dublin regarding flooding should consider the overall picture and relate to issues of both a coastal and fluvial nature and therefore needs to understand existing legislation.

Arterial Drainage (Amendment) 1995 Act

This Act updated the 1945 Arterial Drainage Act to allow investigation of flooding and the undertaking of works on a more localised basis particularly in urban areas. Specifically to look at the impacts on a watercourse as opposed to catchment wide.

The importance of this Act to the DCFPP is similar to reasons stated above for IBP 33.

Review of Cost Benefit Procedures for Flood Relief Schemes – Goodbody Economic Consultants, 2001

To highlight and compare the different Irish methods available for assessing flood relief schemes.

This report uses the Flair (Multicoloured Manual IBP1) and the PPPs (Purchasing Power Parities) to indicate how flood relief methodology and where the pricing indexes differ, but can be compared.

IBP39 Climate Change, Studies on the Implications for Ireland – Department of the Environment, 1994

The studies in this report examine the impact of climate change on; agriculture, forestry, flora and fauna, hydrology and fresh water resources, coastal areas due to changes in mean sea level rise and fisheries. However it does not claim to clarify whether the 'greenhouse effect' will alter Irelands climate, instead it assumes the Irish climate will change.
This report will be used throughout the project as a baseline source of information for the modelling and flood risk impact elements, as well as information for other elements.

7.4 Site Visits

As part of the project a study tour was proposed to undertake visits to other international sites and agencies from which the project could draw upon the latest international best practice.

A site visit was organised by RH to visit the Thames Barrier in June 2004 to see the facility, but more importantly to discuss the forecast system and response procedure which the barrier employs. This was particularly relevant to the DCFPP's proposed surge forecasting model since the Thames Barrier warning system utilises the UK Met Office Storm surge forecast model, which is also to be used to provide input forecast information to the Dublin and Fingal Flood Forecasting System.

During the meeting access was gained to the barrier operational control room and the system and procedure used to implement a closure discussed in some detail with the project team and representatives from DCC. The visit provided a good insight into the operational use of a system similar to that proposed for Dublin including the sensitivities of its use and the importance of an appropriate operational response team to read, monitor and interpret the incoming information with respect to recorded data on the ground.

Furthermore it was agreed that the actual study tour itself be carried out only after first experience has been gained from the use of the early warning system. At the moment of completion of this report this was scheduled for September 2005. The first thoughts regarding the programme of this study tour is to share experiences regarding early warning systems with the city of Rotterdam and / or Dordrecht in the Netherlands. The final programme is to be determined in the next few months.

7.5 Links to Strategy and Policy

Within Phase 1 the Overview of International Best Practice mainly concentrated on documents from the UK and Ireland and a brief summary of those documents identified in the context of the DCFPP and their possible use. In the stages of the project that followed on from Phase 1, the review of IBP was carried forward as part of the work undertaken in relation to the investigation into strategy and policy, see chapter 16. In that section further review work is undertaken in relation to key documents.

8 REVIEW OF MEAN SEA LEVEL

8.1 Background

In Chapter 9, Section 9.3, an overview of the "tide" at Dublin Port has been given. This describes the influence of the components of the astronomical tide, observed seiches, surges and developments in mean sea level. In the remainder of this chapter a more detailed consideration of mean sea level is set out, based on trends in sea level rise derived from the data collected as part of the study.

In addition to historic sea level rise, a detailed review of international best practice in respect of future predictions of sea level rise has been undertaken and a discussion on this topic is presented in Section 9.6.3. In general, this chapter, Chapter 8, is aimed at reaching some firm conclusions on the issues of historic and predicted sea level rise for recommendation and use within the DCFPP.

The starting point for this work was a detailed review of historic sea level rise information from the following:

- Review of DCFPP MSL tide data presented in Chapter 9 to determine an actual figure for SLR from the data plotted in Figure 9.5, Appendix I.
- Review of historical results from other data sources, reports and studies.

This was then followed by a detailed review of the latest guidance on and best practice documents in relation to predicted sea level rise. This review included:

- Latest UKCIP02 guidance
- Inter-governmental Panel on Climate Change - 2001 guidance
- GDSDS climate change policy document and recommendations.
- Other Irish and UK best practice documents, papers and studies.

A quick overview of all the reports and information sources covered and the results is presented in Table 8.1 below. This is followed by a more detailed presentation of results and discussion of the most important sources of information in respect of both historic and predicted sea level conditions within Sections 8.2 and 8.3 respectively. From these discussions some conclusions and recommendations in respect of sea level rise for use within the DCFPP are presented in Section 8.4.

All figures 8.1 to 8.6 mentioned in this chapter can be found in Appendix H.

Table 8.1 - Review and Summary of Literature on Sea Level Rise

Author	Document/ Report	Source Data	Historic Sea Level Rise	Predicted Sea Level Rise
John Sweeney, NUI for EPA	Climate Change, Scenarios & Impacts for Ireland	Statistical downscaling technique on output from Hadley GCM.	Global SLR - 10-20cm over past century. (**1-2mm/yr**)	Global SLR – 50cm during 1990-2100. (**4.5mm/yr**)
B.E. McWilliams, EPA, for Dep. Of Environment	Climate Change Studies on the Implications for Ireland	IPCC, Climate Change – The IPCC Scientific Assessment 1990.	Suggested present (1990) global mean SLR to be 1mm/yr. **Carter analysis Dublin data 1938-1980 = +0.3mm/yr**. Valentin analysis Dublin data 1938-1951 = **+0.5mm/yr**.	18cm 1990-2030. (**4.5mm/yr**) Scenario based on IPCC guidelines. Also looked at best and worst case of 2.25 & 7.5 mm/yr. respectively.
Brady Shipman Martin	Coastal Zone Management A draft Policy for Ireland	Latest IPCC guidance at time of writing, pre 1997!	N/a	50cm by 2100 (**4.5mm/yr**, assuming IPCC 1990))
Robert J.N. Devoy UCC	Implications of Accelerated Sea Level Rise for Ireland	Historic data – Woodworth & Jarvis, 1991 (POL). Predicted – Various papers.	**Dublin - +0.24±0.34 mm/yr**	40cm to 2100, (**4mm/yr**)
D. Richardson , DEFRA, Paper	Flood Risk – the impacts of climate change	Latest guidance at time of writing (pre April 2002), of IPCC & UKCIP.	N/a	**4.5mm/yr**, (South west & Wales = 4.5 + 0.5 for land rise = 5mm/yr)
DEFRA/EA	UK Climate Impacts Programme 2002, Climate Change Scenarios: Implementation for Flood and Coastal Defence: Guidance for Users.	UKCIP, April 2002	N/a	**5mm/yr** for South West and Wales (as DEFRA current practice above).

Author	Document/ Report	Source Data	Historic Sea Level Rise	Predicted Sea Level Rise
McCarthy Acer & MCOS for DCC	Dublin Bay Project, Appendix A – Dublin Still Water Level Study	Table of IPCC(1995) projections of Global MSL	N/a	Estimates show nonlinear increase. Based on **"best guess "** estimates the following is inferred, 200mm increase 1990-2050 (average yearly value 3.33mm/yr) & 490mm increase 1990-2100 (average yearly value **4.45mm/yr**)
		What the Authors Recommended for use in Study	N/a	Upper end of range value for year 2060 = 473mm, (average yearly value of **6.75mm/yr**)
McCarthy Acer & MCOS for DCC	GDSDS Regional Policies – Volume 5 Climate Change	UKCIP02	N/a	300 to 400mm by end of 21st century around Ireland. (**3-4mm/yr**)
		Guidance from NUI Maynooth.	N/a	NUI assessment of 8GCM gives average of 480mm by end 21st C. (4.8mm/yr). Should consider drop in land at Dublin measured at 0.3mm/yr. Recommend range 400mm to 480mm is used. (**4-4.8mm/yr**)
		Recommendation of policy	N/a	440mm to 2080 (**5.5mm/yr**)

Author	Document/ Report	Source Data	Historic Sea Level Rise	Predicted Sea Level Rise
UKCIP02	Climate Change Scenarios for the UK.	Low Emmissions (B1) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2080s 23cm (**2.87mm/yr**) 9cm (1.125mm/yr) 48cm (6mm/yr)
		Medium-Low Emissions (B2) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2080s 26cm (**3.25mm/yr**) 11cm (1.375mm/yr) 54cm (6.75mm/yr)
		Medium-High Emissions (A2) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2080s 30cm (**3.75mm/yr**) 13cm (1.625mm/yr) 59cm (7.375mm/yr)
		High Emissions (A1F1) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2080s 36cm (**4.5mm/yr)** 16cm (2mm/yr) 69cm (8.63mm/yr)
IPCC (2001)	Climate Change 2001: The Scientific Basis, Chapter 11	Scenario – **B1** (UKCIP02 – **L**) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2100 0.31m (**2.8mm/yr**) 0.09m (0.82mm/yr) 0.57m (5.18mm/yr)
		Scenario – **B2** (UKCIP02 – **ML**) "**Central**" or best estimate "Low" estimate "High" estimate	N/a	To 2100 0.36m (**3.22mm/yr**) 0.12m (1.05mm/yr) 0.65m (5.9mm/yr)
		Scenario – **A2** (UKCIP02 – **MH**) "**Central"** or best estimate "Low" estimate "High" estimate	N/a	To 2100 0.42m (**3.82mm/yr**) 0.26m (2.32mm/yr) 0.74m (6.72mm/yr)

Author	Document/ Report	Source Data	Historic Sea Level Rise	Predicted Sea Level Rise
IPCC (2001) (cont'd)	Climate Change 2001: The Scientific Basis, Chapter 11	Scenario – **A1F1** (UKCIP02 – **H**) "**Central"** or best estimate "Low" estimate "High" estimate	N/a	To 2100 0.49m (**4.45mm/yr**) 0.19m (1.68mm/yr) 0.875m (7.95mm/yr)
PSMSL - POL	PSMSL Web site database of MSL values	MSL values for Dublin from 1938 to 1996. MSL data provided to POL by DPC.	**0.23** ± 0.3 **mm/yr**	N/a
Royal Haskoning.	DCFPP	MSL trend – Analysis of 15 years of tide data from DPC obtained for DCFPP.	Yearly MSL data analysis – **0.287mm/yr** Monthly MSL data analysis – 0.358mm/yr	-
		PH analysis of MSL values for Dublin from 1938 to 2001 taken from PSMSL website database. MSL data provided to POL by DPC.	Yearly MSL data analysis – **0.193mm/yr** Monthly MSL data analysis – 0.114mm/yr.	-
		Predicted SLR – review of latest guidance and IBP documents. Recommendation for use in DCFPP.	-	**4.15mm/yr** to 2100 for general design use.

8.2 Historical Sea Level Rise

8.2.1 General

Trend lines were initially established from the mean sea level values calculated using the tidal data obtained from the Dublin Port tide gauge, along with estimates of sea level rise over the period of the data produced. During a review of other sources of information on historical sea level rise for Dublin, it emerged that the Permanent Service for Mean Sea Level (PSMSL) holds a database of mean sea level values for numerous ports around the world, including Dublin. This data set was obtained from their web site and analysed for comparison with the results of the MSL data obtained for the DCFPP.

Finally the historic SLR results from a number of other sources for Dublin have been reviewed and compared with both analyses.

8.2.2 DCFPP – Tidal Data

As presented in Section 9.3.3 and Figure 9.5 (Appendix I), values of MSL were calculated for the tide data obtained from the Dublin Port records for the DCFPP (see Section 9.2). These results have been reproduced in Figure 8.1 for the MSL values for yearly and monthly analysis. Linear trend lines have been added to the data and equations for these determined and presented in the figures. From these equations estimates of the change in mean sea level have been made over the period of the data obtained.

For the yearly analysis the earliest complete year of data obtained for the DCFPP was 1943 and the most recent 2002. From the analysis of the trend line between these two periods, a sea level rise of 1.72cm was determined providing an annual average sea level rise in the order of 0.287 mm/year.

For the monthly analysis the earliest complete month of data obtained for the DCFPP was November 1924 and the most recent May 2003. From the analysis of the trend line between these two periods, a sea level rise of 2.86cm was determined providing an annual average sea level rise in the order of 0.358 mm/year.

The results of both these analyses are indicating relatively low estimates for sea level rise over the last century in comparison to the general view given that global sea level has risen by between 1 to 2 mm/year over that period and certainly much smaller than any estimates of predicted future sea level rise.

8.2.3 Permanent Service for Mean Sea Level – Tidal Data

The PSMSL is responsible for the collection, publication, analysis and interpretation of sea level data from a global network of tide gauges, one of which includes the tide gauge at Dublin Port. The data for Dublin Port gauge has been retrieved from the PSMSL's web site database and analysed for use and comparison within the DCFPP. The PSMSL indicate within the web site that they receive the MSL data directly from the authority responsible for the tide gauge (referred to as metric data) and do not themselves calculate the values from tide records provided to them. They do, however, carryout a number of standard reliability checks on the data when received and, where possible, convert it to a uniform world wide datum known as “Revised Local Reference” (RLR). The data can only be converted to this if the relationship between RLR and the local datum has been established for the gauge on site. Both metric and RLR data is available on the PSMSL website for Dublin.

For the purpose of analysis in the DCFPP all data has been converted to local datum (LAT) to aid comparison across data sets. Both monthly and yearly MSL data was available. Each data set has been plotted and linear trend lines fitted in order to evaluate the respective rates of sea level rise. Yearly values of MSL are available from 1938 to 2001 with the exception of 1997 and 1998. These values are plotted in Figure 8.2 and from analysis of the trend line between these two periods, a sea level rise of 1.26cm was determined providing an annual average sea level rise in the order of 0.193mm/year. The second graph presented in Figure 8.2 compares the plot of the

PSMSL data against the PH yearly MSL values. It can be seen from the plot that the PH yearly MSL values are higher than those of the PSMSL, with the smallest difference being 29mm and the largest 93mm. These differences are probably down to the method of calculating the MSL value from the data set and the differences are not unduly large. The value of sea level rise is smaller than that obtained from the PH data however the PSMSL data set is more complete. Notwithstanding the differences referred to above, it is interesting to note that the gradient of the lines are similar, indicating that the rate of rate of sea level rise is the same in both data sets.

Similarly monthly values of MSL were available from 1938 to 2001, with only a number of months missing in the years 1997 and 1998. These values have also been plotted and are presented in Figure 8.3. A linear trend line has been fitted to the data and the trend line analysed across the period of the data set to provide an estimate of sea level rise. From this analysis a sea level rise of 0.74cm was determined providing an average annual sea level rise of approximately 0.114 mm/year. The second graph in Figure 8.3 shows a comparison plot of the monthly PSMSL data with that of PH values. In general the PH monthly values are again higher than those from the PSMSL data set. The differences are, for the most part less than 100mm. However for a number of the month the PH MSL values are lower than the PSMSL values and on a number of occasions the difference is between 100 and 200mm. The value of sea level rise obtained from the PSMSL monthly data set is much smaller than that for the PH data set which was presenting a value of 0.358mm/year. This could be due to the fact that:

- the PH data set is more complete in recent years which could have introduced a bias on the trend line; or
- the monthly MSL values seasonally biased and only a limited number are available within the early years of the PH data set. If these data values related to a month for which the MSL was particularly low this could have the effect of showing an increased SLR trend line. Therefore it is considered that the trend line for the PSMSL data is more realistic.

Nevertheless the range of values obtained across both data set provided estimates of historic sea level rise for Dublin of between 0.114 to 0.358 mm/year with a mean of 0.236mm/year. This is considerably lower than the current world consensus which suggests that global sea level has risen by between 1mm - 2 mm/year over the last century and very much lower than estimates of future sea level rise over the coming century. In addition to the raw data being available on the PSMSL web site, a table of MSL secular trends obtained for the RLR data sites across the world was also available. For Dublin the trend had been calculated from a data set between 1938 and 1996 and provided a mean sea level rise of 0.23 mm/year with a standard error of ±0.3mm/year and a standard deviation of the residual variability about the fitted trend line in mm of 38.3. This value falls within the range of values calculated from both the PH and PSMSL data sets.

8.2.4 Comparison with Other Work

In order to provide a better feeling for the reliability of the estimates of SLR for Dublin made within the DCFPP analysis, a review of work undertaken for other projects was carried out and comparisons made. The comparison can be undertaken at two levels. The first is with estimates given in reports of global SLR and the second is with estimates of SLR for Dublin arising from the analysis of the port tide data.

At the first level comparison two reports provided statements on global SLR:

i) the report by Sweeney (2003) – "Climate Change, Scenarios and Impacts for Ireland" commented that global sea level rise was in the order of 0.1/ to 0.2m over the last century.
ii) McWilliams (1990) - "Climate Change Studies on the Implications for Ireland" suggested that present (1990) global sea level rise estimates were in the order of 1mm/year.

The second report also reported estimates of sea level rise for Dublin that were as a result of work undertaken by Professor Carter, as well as earlier work by Valentin. Valentin carried out an analysis of the Dublin data from 1938 to 1951 and estimated SLR values of +0.5 mm/year. Carter carried out an analysis on a longer data set from 1938 to 1980 and presented estimates for Dublin in the order of +0.3 mm/year. Carter's estimates in particular are of the same order of magnitude to those obtained as part of the DCFPP research.

A third paper by Devoy (2000) - "Implications of Accelerated Sea Level Rise for Ireland" the rate of sea level rise at Dublin is estimated to be +0.24mm/year to ±0.34mm/year. The data source quoted within the paper was through work undertaken by researchers at Proudman Oceanographic Laboratory (POL) and is therefore believed to be the same as that obtained from the PSMSL secular rise table, discussed earlier.

It should be pointed out that the sea level rise estimates obtained from the analysis of the Dublin Port tide records are relative sea level rise estimates i.e. the values are relative to the land on which the gauge is located and consequently include any rebound/or subsidence of that land due to recovery from the last ice age.

The GDSDS climate change report states that the land at Dublin is dropping and this has been measured at a rate of 0.3mm/year. If this is indeed the case then it would tend to indicate that actual SLR at Dublin is closer to zero, with the fall in land accounting for most if not all of the relative sea level rise observed at Dublin.

8.3 Future Predicted Sea Level Rise

8.3.1 General

A detailed review of the latest guidance on climate change, with particular regard to sea level rise, has been undertaken. This has included a review of the latest UK Climate Impacts Programme, 2002 (UKCIP02) and Intergovernmental Panel on Climate Change, 2001 (IPCC, 2001) recommendations as well as a review of other relevant international best practice and reports, studies and research within Ireland. From the results of the review a detailed discussion on the information is presented and some recommendations made in respect of values of sea level rise to be used within the DCFPP and by DCC and FCC in respect of future design considerations.

8.3.2 Research and Reports in Ireland

A number of reports and research papers relevant to Ireland have been reviewed for guidance on SLR. The guidance in each is presented in the summary table 8.1, however the most relevant of these include,

- Climate Change Studies on the Implications for Ireland, Department of Environment 1990.
- Climate Change, Scenarios and Impacts for Ireland, Environment Protection Agency 2003.

The first, Climate Change Studies on the Implications for Ireland, was prepared over ten years ago and presented a number of scenarios for use based on IPCC guidelines at that time (IPCC, 1990). The second, Climate Change, Scenarios and Impacts for Ireland, has been completed more recently and presents recommendations based on a review of international work and work undertaken at the National University of Ireland (NUI) on a regional downscaling technique of GCM's for Ireland. A comparison of both will give an understanding of how guidance and research has changed over the last decade.

The 1990 report presented three scenarios for consideration. Only the best (central) estimate is considered here for discussion. For that scenario recommendations were given to consider a SLR of 180 mm from 1990 to 2030. This equates to an annual average rise of 4.5mm/year.

The recent work undertaken by NUI suggests a likely global sea level rise in the order of 0.5m will occur during the period 1990 to 2100. This value is also considered to be likely around the Irish coastline and equates to an annual average rise of 4.5mm/year.

This suggests that guidance on recommendation for predicted SLR has not changed over the last decade.

8.3.3 Latest Guidance in the UK

The latest guidance for the UK is based on the recommendations of the UKCIP02. This guidance is presented in a R&D technical report produced for the Environment Agency (EA) and the Department for Environment, Food & Rural Affairs (DEFRA) entitled "UK Climate Impacts Programme 2002 Climate Change Scenarios: Implementation for Flood and Coastal Defence: Guidance for Users." The recommendations for Wales are presented here as they are considered to be most relevant to the Dublin Area. A value of 5mm/year has been recommended for use around Wales. This includes a value of +4.5mm/year for sea-level rise and a further +0.5mm/year to allow for land level movement. This recommendation has not changed from the previous guidance given by the Ministry of Agriculture, Fisheries and Food, 1999 (MAFF, 1999).

It is considered that this recommendation reflects the higher emissions scenarios considered by the UKCIP02 and IPCC01, see review of scenarios below.

8.3.4 UK Climate Impacts Programme 2002 (UKCIP02)

The latest guidance form the UK Climate Impacts Programme, was published in April 2002 in a report entitled Climate Change Scenarios for the United Kingdom: The UKCIP02 Scientific Report. The report presents the results of the latest global climate models used by the Hadley Centre, of the UK Met Office, for each of four scenarios based on the latest global emissions scenarios published in the Special Report on Emissions Scenarios (SRES) by IPCC in 2000. These four scenarios include:

Low Emissions (B1)	Clean and efficient technologies; reduction in material use; global solutions to economic, social and environmental sustainability; improved equity; population peaks mid-century.
Medium-Low Emissions (B2)	Local solutions to sustainability; continuously increasing population at a lower rate than A2; less rapid technological change than B1 and B1.
Medium-High Emissions (A2)	Self-reliance; preservation of local identities; continuously increasing population; economic growth on regional scales.
High Emissions (A1F1)	Very rapid economic growth; population peaks mid-century; social, cultural and economic convergence among regions; market mechanisms dominate. Reliance on fossil fuels.

No one of the above scenarios is any more likely to occur than the next as they are each dependent on choices made by society and as such no probabilities or likelihood of occurrence can be assigned to each. The guidance given by the UKCIP02 report recommends that all four climate change scenarios be used in any impact assessment, with impacts and implications of a minimum of two contrasting scenarios being evaluated.

With this in mind, the recommendations of UKCIP02 in relation to globally-averaged sea-level rise using the Hadley Centre models for each of the above scenarios relative to a 1961 – 1990 average baseline period, for three 30-year periods – the 2020s, 2050s and 2080s, are reproduced in the table below.

Table 8.2 - UKCIP02 Global-Average Sea-level Change (cm) Recommendations

Scenario	2020s (cm)	2050s (cm)	2080s (cm)
Low Emissions (B1)	6 (4 – 14)	14 (7 – 30)	23 (9 – 48)
Medium-Low Emissions (B2)	7 (4 – 14)	15 (7 – 32)	26 (11 – 54)
Medium-High Emissions (A2)	6 (4 – 14)	15 (8 – 32)	30 (13 – 59)
High Emissions (A1F1)	7 (4 – 14)	18 (9 – 36)	36 (16 – 69)

Note: The values in brackets are the ‘low’ and ‘high’ estimates for each scenario taken from the IPCC range associated with the same SRES emissions scenario, with the HadCM3-derived values adopted as ‘central’ estimates.

Considering the GASLC results to the 2080s with a base-line period of 1961-1990 (i.e. 90 years), it is possible to determine the annual average sea level rise value over that period and the results are presented below.

Table 8.3 - Annual Average Sea Level Rise Values (mm/yr) Based on UKCIP02 Recommendations

Scenario	Estimate Range		
	Low	Central	High
Low Emissions (B1)	1.0	2.56	5.33
Medium-Low Emissions (B2)	1.22	2.89	6
Medium-High Emissions (A2)	1.44	3.33	6.6
High Emissions (A1F1)	1.77	4	7.66

The values of global sea level rise presented above for the UKCIP02 scenarios are generally smaller than those recommended in the UKCIP98 reports, particularly so at the upper end of the range. For example the UKCIP02 High Emissions scenario for the 2080s provides a range of 16 to 69 cm, compared to a value of 99 cm quoted for the same scenario in UKCIP98. The UKCIP02 report suggests that the differences between both reports are considerably less for the other three scenarios.

The reason for the differences is attributed mainly to improvements in model representation of both the ice melt and ocean heat uptake, with the sea level rise being less sensitive to global temperature change than previously thought. This arises also despite the UKCIP02 recommending larger global temperature increase compared to those of UKCIP98.

8.3.5 Intergovernmental Panel on Climate Change 2001 (IPCC(2001))

The latest guidance form the Intergovernmental Panel on Climate Change, was published in 2001 in a report entitled Climate Change 2001 The Scientific Basis. The report presents the results of a comprehensive range of Atmosphere-Ocean General Circulation Models (AOGCM's) for six scenarios which includes the four presented earlier with a further two adaptations on A1 for varying levels of reliance on fossil fuels (A1T & A1B). Each of the scenarios used are based on the latest global emissions scenarios published in the Special Report on Emissions Scenarios (SRES) by IPCC in 2000.

For comparison purposes with the UKCIP02 only recommendations for the results of the B2, B1, A2 and A1F1 scenarios have been reproduced below. These results have been inferred from Figure 11.12 in Chapter 11, Changes in Sea Level, of the IPCC(2001) Climate Change 2001 report. It should be noted that the results of the IPCC01 work presented in Figure 11.12 show global average sea level rise from 1990 to 2100. The results indicate a non linear trend as with the UKCIP02 results.

Table 8.4 - IPCC(2001) Global-Average Sea-level Rise (cm) Results

Scenario	2020 (cm)	2050 (cm)	2080 (cm)	2100 (cm)
Low Emissions (B1)	5.5	13.5	23.5 (9 – 48)	31 (9 – 57)
Medium-Low Emissions (B2)	5.5	14.5	26 (11 – 54)	35.5 (11.5 – 65)
Medium-High Emissions (A2)	5	14	29 (13 – 59)	42 (15.5 – 74)
High Emissions (A1F1)	6	16.5	35 (16 – 69)	49 (18.5 – 87.5)

The results presented above which are not in brackets represent the average of seven AOGCM's for each scenario, or the 'central' estimate in the overall range. The values in brackets for 2080 and 2100 represent the estimate range ('low' and 'high') for all the AOGCM's, with the values for 2080 taken as those referred to by UKCIP02 for the same year and those for 2100 taken from the outermost limit lines shown on Figure 11.12 of the IPCC01 report. In summary for the complete range of AOGCM's and SRES scenarios, the IPCC01 predicts a global sea level rise of between 0.09 to 0.88 metres over 1990 to 2100, with a central value of 0.48 metres.

The IPCC(2001) 'central' values for the year 2080 presented above, are very similar to those presented by UKCIP02 for the same period. In deed the main difference between UKCIP02 and IPCC (2001) is the fact that UKCIP02 only presents estimates to the 2080s while IPCC (2001) estimates to the end of the century. Estimates over the period 1990 to 2080 are in fact very similar as shown in Figure 8.4. In fact the UKCIP02 curves for the higher scenarios are marginally higher at 2080 than the IPCC (2001). The graph indicates that there will be acceleration in annual SLR rates over the second half of the century and in particular the last two decades which has an impact on the annual averages values when calculated over the period 1990 to 2100 compared to 1990 to 2080, as presented in Table 8.5 below.

Table 8.5 - Annual Average Sea-level Rise Values (mm/yr) to the Period 2080 and 2100 Based on IPCC(2001) Recommendations

Scenario	Based on Values to 2080 (90 years)			Based on Vales to 2100 (110 years)		
	Estimate Range			Estimate Range		
	Low	Central	High	Low	Central	High
Low Emissions (B1)	1.0	2.61	5.33	0.82	2.82	5.18
Medium-Low Emissions (B2)	1.22	2.89	6	1.05	3.22	5.9
Medium-High Emissions (A2)	1.44	3.22	6.6	1.41	3.82	6.72
High Emissions (A1F1)	1.77	3.89	7.66	1.68	4.45	7.95

The AASLR values to 2080 are very similar to those of the UKCIP02 presented in Table 8.3 as would be expected from the results presented in Figure 8.4. If SLR to the end of the century is considered, then AASLR values increase by more than 0.5mm/year for the higher emissions scenarios. If such values are considered for design, they will certainly provide a conservative estimate for sea level change over the early to mid period of this century. However, if they are not realised then flood defence structures and indeed other development structures could be considerably over designed in the long term.

8.3.6 Dublin Regional Studies

i) Dublin Bay Project

For the Dublin Bay Project a Still Water Level Study was carried out to determine appropriate design water levels for structures around the shoreline of Dublin Bay. The study recommendations were based on the latest IPCC guidance at the time which was IPCC(1995). The report presents a graph of the IPCC(1995) projections of Global MSL from 1990 to 2100, however it does not say which, if any, scenario these represent.

The graph shows a non-linear rise and the results to 2100 indicate sea level rise with a central ‘best’ estimate of 49 cm (4.45mm/year), and ‘low’ and ‘high’ estimates of 20 cm (1.82mm/year) and 86 cm(7.82mm/year), respectively.

The study then goes on to recommend that, based on a design life to 2060, the ‘high’ estimate to that year should be considered as the appropriate design consideration, given uncertainties in the IPCC(1990) projections. This value was 473mm and provides an annual average sea level rise to 2060 of 6.75mm/year. This is extremely conservative and ranks with the ‘high’ range estimates for the higher scenarios of both the UKCIP02 and IPCC(2001).

ii) Greater Dublin Strategic Drainage Study (GDSDS)

As part of the GDSDS a number of policy documents were produced. One of these dealt with climate change issues on a wide context and was entitled ‘Regional Policies – Volume 5, Climate Change. The report was aimed at providing recommendations for design criteria for drainage works in respect of incorporating aspects of climate change for such parameters as Temperature, Rainfall, Sea level and Ground Water. The report then goes on to discuss implications for drainage design and provides recommendations for design.

Whilst the completed report has been reviewed, only the relevant sections on aspects of sea level rise have been reviewed in some detail and are discussed here.

In respect of the climate change debate, emissions scenarios and climate change models, the report suggests that its conclusions reflect the recommendations of the UKCIP02 work for the Medium-High scenario and that a design horizon of the order of 80 to 90 years should be considered in respect of climate change.

Section 3.4 of the GDSDS climate change policy report deals with ‘Sea Level’ and makes a number of recommendations in respect of extreme water levels and future sea level rise. It states that by the end of the 21st century the UKCIP02 report suggests sea level rise around Ireland will be in the order of 300 to 400mm. The climate change policy report then goes on to suggest that there are some concerns within Ireland at the

NUI, that this estimate is not conservative enough and their (the NUI) evaluation of 8 GCM's suggests a value more in line with 480mm to the end of the century. This is very similar to the IPCC(2001) central estimate for the complete range of AOGCM's and SRES scenarios as presented in their 2001 report.

The policy report also considers land subsidence at Dublin and suggests a value of 0.3mm has been measured for Dublin, but does not confirm where this figure comes from. The report suggests that taking a more precautionary approach a figure for sea level rise and land subsidence over the next century of between 400 and 480mm might be more appropriate. The policy report also considers the impacts of sea level rise beyond the end of the century and in particular the impact on strategically important assets. For such assets it recommends that a value of sea level rise in the order of 1 metre be considered.

In Chapter 4 of the GDSDS policy document, 'Recommendations for incorporating climate change issues into drainage design criteria', the report concludes that the results of the Hadley Centre HADRCM3 model and the Medium – High (A2) SRES scenario should be used for climate change policy. It recommends that projections to 2080 should be made for all infrastructure design unless design lives are short and that this should consider an assumed sea level rise to 2080 in Greater Dublin of 440mm, including land subsidence.

In respect of the above recommendations it should be noted that the projection of 440mm, 410mm (4.55mm/year from 1900 to 2080) without land subsidence, is in excess of the UKCIP02 central estimate for even the High scenario. Indeed this value of 410mm is in excess of the IPCC01 projections to 2080 for the High scenario and fits better with their Medium-High projection to the end of the 21st century, see Table 8.5 above.

In Figure 8.5 and 8.6, the recommendations of the GDSDS are compared to the UKCIP02 and IPCC (2001) recommendations, and are presented as AASLR values estimated by period, for the Low, Medium-Low, Medium-High and High scenarios.

8.4 Dublin Coastal Flooding Protection Project Findings

8.4.1 General

In this section an overall discussion on the topic of sea level rise will be presented based on the information and results presented in the preceding sections. From this discussion some recommendations will be made in respect of sea level rise methodology for use within the DCFPP and by DCC and FCC on future projects where sea level rise should considered.

8.4.2 Discussion

Overview

A detailed review of the latest guidance from both UKCIP and IPCC has been undertaken, together with other relevant guidance, studies and reports for the UK and Ireland. The subject is quite complex and the amount of information available is extensive and as such different conclusions and interpretations of the results are inevitable. The work and recommendations of the two leading international experts on the topic, the UKCIP and IPCC, do not point to or attempt to provide a definitive answer on climate change or sea level rise, but present varying scenarios, all of which are considered possible but each without a level of probability of occurrence associated with them. They recommend that the user interpret the impact of a number of the scenarios in any given assessment. However, this inevitably leads to inconsistent interpretation of the results and an “erring on the side of caution” with the results for the most extreme scenarios often being recommended for use, in light of uncertainties in the scenarios.

Whilst such interpretation will lead to conservative designs, they could result in considerable over design, particularly if the higher estimates of SLR are chosen and later not realised, leading to visually obtrusive and uneconomic solutions. Similarly if designs are based around SLR estimates for the lower scenarios then flood defences and other structures could be under designed, if the higher SLR estimates are shown to be applicable through further research in the future. Indeed recent guidance on SLR has produced lower estimates than the pervious guidance given despite the fact that predictions of an increase in global average temperatures have been made since pervious estimates. Climate change is something that happens over a long period of time and given the time scales over which it is likely to occur, the time over which research into the topic has been undertaken is very small indeed. Therefore it will take a much longer period of time to develop a full understanding of the facts and confirm estimates and recommendations made through continued monitoring and research. Indeed the wide ranging recommendations of both the UKCIP and IPCC tends to reflect this through use of the range of scenarios considered.

Haskoning’s approach to climate change, and in particular recommendations for sea level rise on the DCFPP, reflects this and the recommendations of the UKCIP02 to consider the impact of other scenarios.

UKCIP02 and IPCC (2001)

When considering issues of climate change most data sources refer back to the work of either the UKCIP or the IPCC. One might expect that the results of the UKCIP would be most applicable to Ireland given the location. However, the policy document of the GDSDS considers that the recommendation of the UKCIP02 for sea level rise might not be conservative enough. When a comparison of the results of the UKCIP02 to the IPCC (2001) is made, as presented in Figure 8.4, it can be seen that both recommendations are in considerable agreement, with the main difference being that IPCC predicts SLR to the end of the century while UKCIP only predicts to 2080. Therefore one must not simply compare the predictions made towards the end of the century for both the UKCIP and IPCC without considering the prediction truncation date.

The results presented in Figure 8.4 show a non linear trend and indicate that sea level rise accelerates over the second half of the century particularly for the higher emissions scenarios. From the results presented in Figure 8.4, it is likely that had UKCIP02 provided predictions to the end of the century, then these would not have been considerably different from those provided by IPCC (2001). The figure also indicates that care must be taken when considering Annual Average Sea Level Rise (AASLR) values, as these can vary considerably over the century given the non linear nature of the rise. That said Haskoning’s approach recommends the use of AASLR values, as it is felt that these can be more readily applied to a design life than simply stating that a value of X metres should be considered to a given date. However, the limitations or boundary conditions surrounding the use of the AASLR values should clearly be stated in the context of how they have been defined. DEFRA guidance for example recommends sea level rise considerations in this way, but does not clearly define how these have been derived or the limitations on their use, i.e. up to which period they are valid.

Figures 8.5 and 8.6 show a number of bar charts which present the results of the UKCIP and IPCC as annual average values for each of the four scenarios and for varying periods of time over the century.

From the figures it can be seen that AASLR values increase considerably for the periods towards the end of the century. The figures also demonstrate that the AASLR values obtained from the UKCIP02 and IPCC (2001) are very similar for the three 30 year periods up to 2080 and also if calculated over the period 1900 to 2080. It demonstrates that whichever AASLR value is chosen for design it is only valid up to the end of the period over which it has been calculated. For example if a design AASLR value was chosen based on the results over the period 1990 to 2080 for the Medium-High scenario, then the annual average values used would be conservative over the first half of the century to 2050 and by 2080 they would have realised the full sea level rise as predicted to that point. However, their use to design to 2100 would be inappropriate if the predictions of the IPCC to the end of the century are valid.

DCFPP Recommendations

Following the research undertaken as part of the project and the results presented in the earlier sections, it is concluded that the results of either the UKCIP02 or the IPCC (2001) are similar and would be applicable for use in design considerations for Dublin and Ireland provided they are considered in the context and time frame to which they have been made.

For the purposes of strategic consideration Haskoning consider that the impact of climate change and sea level rise should be considered up to the end of the century, and that the impacts of at least two scenarios should be considered, with the baseline scenario Medium–High chosen to reflect a slightly conservative but not over conservative approach. This does not mean that structures should be constructed to a level to reflect SLR to the end of the century, more that the design should be robust enough to consider accommodating such sea level rise beyond a structures design life or in a phased approach as appropriate.

To this end annual average sea level rise values over the period 1990 to 2100 should be considered based on the results of the IPCC (2001) work and a further allowance made for the 30mm of land subsidence noted within the GDSDS. This approach will provide a conservative estimate of SLR for design purposes over the first half of the century, while providing an allowance for the predicted value of sea level change to the end of the century.

Based on the IPCC(2001) work and the M-H scenario to the end of the century, an annual average sea level rise of 3.85mm/year (rounded up from 3.82, Table 85) should be used. With the allowance for land subsidence this results in a total relative sea level rise of 4.15mm/year. This value is applicable for use up to 2100.

This value has been plotted on the bar charts presented in Figures 8.5 and 8.6, for comparison with the AA values for each of the four scenarios. The recommendations of the GDSDS have also been presented for comparison.

Whilst this figure is quoted it is recommended that the sensitivity of any design should be checked for another scenario and annual average values (rounded up from those in Table 8.5) of 2.85, 3.25 and 4.45 mm/year should be used for the Low, Medium-Low and High scenarios respectively, with an appropriate land subsidence value.

8.5 Recommendations

The main recommendation of the DCFPP in respect of sea level rise can be summarised as follows,

- Should consider the use of the Medium-High scenario as a baseline minimum for design.
- Should consider the impacts of at least one other scenario in the design of any flood defence or other structures.
- Should base recommendations for design to meet prediction of sea level rise to the end of the century.
- Should consider an annual average sea level rise in all designs of 4.15 mm/year, applicable to the end of the century. This includes an allowance of 0.3mm/year for land subsidence.
- Should consider designing and constructing flood defences using the above value in a phased approach to allow for review and accommodation of changes to predictions pending future research into and realisation of actual SLR values.

9 PROBABILISTIC ANALYSIS

9.1 Background

To properly understand the significance of the tides that occurred on 1st February 2002, an analysis of the tide records from Dublin Port has been carried out.

Dublin Port has almost continuous paper records from their pen plotter dating from 1923 up to 2000. In January 2000, the port installed a new tide gauge at the Alexandra Pier Lighthouse that provides a digital record of the tide levels. These records together provide an invaluable source of data extending back almost eighty years.

Sufficient records have been obtained to ensure that an appropriate and reliable level of analysis can be undertaken. This includes a continuous period of records for correlation analysis with meteorological parameters and further periods of data extending back over the eighty years for long term trend analysis. A complete list of the data that has been extracted together with brief details of the procedures used, is presented in Section 9.2.

The probabilistic analysis of the tidal records has a number of important purposes. The decomposition of the tide into its respective astronomical and surge components is the first step in being able to predict future tide levels, both at the port and along the coastline which makes up the frontage of the study.

In the approach for the probabilistic analysis the correlation between successive time series of water level records are examined. Five years of continuous records are separated into discrete time series of one year each. For each time series of measured water level the correlation between wind speed, air pressure and measured river discharge is investigated.

The next step in the process is to carry out an harmonic analysis to produce the time series of the harmonic tide level. The difference between the harmonic water level and the measured water levels at Dublin Port is the residual water level i.e. that part of the tide height that is as a result of the combined effects of the wind, and barometric pressure.

Finally the records selected from the eighty year period are analysed to give an insight into the trends in sea level rise over the long term.

9.2 Data Collected

The vast majority of the records collected from Dublin Port exist in hard bound copy which makes subsequent use and analysis of the data much more difficult. The most recent three years of data (2000 to present) was available digitally, although there were several gaps in the data set which were filled in using digitised paper records of the tide that existed for the same period. Prior to 2000 the data was manually extracted from the paper tidal trace. From September 1982 to the present day these records were stored in hole punched lever arch files and so were relatively easy to extract and photocopy followed by subsequent digitising. However, prior to September 1982 the records were bound in large books, which made copying of the data much more difficult. An added constraint was that Dublin Port would not permit the bound copies to be removed from the Port offices.

A number of methods of extracting the data from these bound sets was considered and included digitising them locally, unbinding them photocopying and rebinding them, all of which would have to be undertaken within the Port. However, the method finally chosen was to digitally photograph the required records. The photographs were then manipulated to close to A3 size, printed out and digitised as before. This method was chosen because it was the most efficient and the least damaging to the actually Port records. Subsequently the remainder of the Port records have been photographed and stored on CD ROM. These have not, however, been included in the analysis reported in this chapter.

The data used in the analysis is listed in Table 9.1 below.

Table 9.1 - Selected Data

Year	Complete Date Range	Data Format
2000 - 2003	1st Jan 00 - 30th May 03	Digital tide recorder
1997 - 1999	1st Oct 97 - 31st Dec 99	Paper records
1995	11th Oct – 13th Nov	Paper records
1993	1st Jan 93 - 31st Dec 93	Digital image of paper record
1988	1st Jan - 31st Dec	Digital image of paper record
1983	1st Jan - 31st Dec	Digital image of paper record
1981	17th Nov – 24th Dec	Digital image of paper record
1974	11th Jan – 11th Feb	Digital image of paper record
1973	1st Jan - 31st Dec	Digital image of paper record
1968	1st Jan - 31st Dec	Digital image of paper record
1954-1955	1st Nov – Jun 30th	Digital image of paper record
1945	2nd Nov – 21st Dec	Digital image of paper record
1943	1st Jan - 31st Dec	Digital image of paper record
1933	4th Feb - 4th Mar	Digital image of paper record
1925	7th Feb - 21st Mar	Digital image of paper record
1924/1925	29th Nov – 17th Jan	Digital image of paper record

9.3 The Dublin Port Tide Gauge

9.3.1 General

Variations in the tide level recorded at Dublin Port tide gauge are due to a variety of factors. These include:

a) fluctuations in the astronomical tide;
b) variations in mean sea level (MSL);
c) the occurrence of seiches;
d) meteorology; and
e) discharges from the River’s Liffey and Dodder

Of the above, the main force that causes variations of the water level at Dublin Port is the astronomical tide. The astronomical tide is generated by the attraction forces of the moon and the sun and to a much lesser degree, other planets, and by the geometry of the Irish Sea and Dublin Bay. Methods of predicting the astronomical tide are well established. Changes in the astronomical tide (if any) normally have a long time scale. Analysis of old records of the tide may show any of such variations in the past. These changes can be very relevant for the study on high water levels. It is possible that the probability of occurrence of high water levels over time changes because of changes in the astronomical tide. Verification of this is an important part of the water level statistics, and is covered in more detail in Section 9.3.2.

It is also important to understand the impact of the behaviour of MSL over the course of time. Climate studies indicate that changes in MSL have occurred in the past and the trend is for this to continue in the future. For any study on coastal flooding must also be considered alongside isostatic and eustatic changes in the land mass. In Dublin it is expected that the changes in land level over the last decades are limited. In Section 9.3.2 the historical development of the MSL is discussed, whilst in Section 9.6 the anticipated sea level rise is presented based on a consideration of "international best practise".

Visual inspection of the paper records of the tide at Dublin Port shows that seiches are present in Dublin from time to time. Seiches are relatively short water level fluctuations that behave like tides, but with a much shorter period (typically 1 hour). They are more commonly caused by the geometry of a harbour basin, but in the case of Dublin is influenced more by the geometry of Dublin Bay. The wavelength of a seiche (which is related to the period) corresponds with the dimensions of the basin (or bay) where it occurs. For this reason, the resonance frequency at which the seiche occurs is unique to the site. Thus if some disturbance enters the basin or bay with a frequency that corresponds to the resonance frequency, the seiche starts to develop and continues for as long as the disturbance continues.

Other influences that affect the development of extreme water levels involve the local meteorology, in particular the barometric pressure and wind strength. For example there is a direct correlation between barometric pressure and water level. A high barometric pressure pushes the water level down, while a low barometric pressure allows the water level to rise. Moreover a low barometric pressure occurring away from the site may generate a surge that travels some distance to the site, thereby adding to the local effects. **This was the case with the flooding of 1st February 2002, where the low pressure that existed to the north east of Ireland generated a surge that travelled around the north of the island and down into Dublin Bay.**

Wind and wind generated waves also cause a temporary change of the water level. Local wind shear at the water surface generates a water level gradient in the direction of the wind, resulting in different water levels at the coast to those out in the Irish Sea. Furthermore the effect of wind strength is not limited to the generation of a localised surge. Wind fields and other weather patterns in the ocean may generate a surge that travels away from the meteorological disturbance that has generated it and, as a result, can cause increases in water levels at the site without there being any significant wind activity present

The last component in the list above relates to the effects of river discharge on the water level. High river discharges are known to increase water levels along water courses. From the records obtained of discharges along the rivers Liffey, Dodder and Tolka, the study has considered the effects on the water level at the Dublin Port tide gauge.

Each of the above are examined for their contribution to and influence on the tide levels at Dublin. Where possible historical trends are derived from the data to help understand what changes have occurred over the last 80 years.

9.3.2 Historical Development of the Astronomical Tide

The astronomical tide is the major component contributing to the variation in water levels at Dublin Port. The predicted tide is published annually in the Dublin Tide Tables, which in turn are supplied by the Proudman Oceanographic Laboratory in Bidston, UK.

To illustrate this, it is useful to compare the highest and lowest predicted tides. In 2003 the highest predicted tide level was 4.47m above Lowest Astronomical Tide (LAT) occurring on March 20th, April 18th and October 26th. For the same year the lowest predicted level was 0.15m above LAT, occurring on March 19th and April 17th. Hence between the periods 19th – 20th March 2003 and 17th – 18th April 2003 the water level varied by more than 4m within a period of twenty four hours.

The driving force behind the tide is the astronomical tidal motion. This motion consists of a large number of sinusoidal components with their own amplitude, frequency and phase. See Appendix I. The complete list contains more than 100 components. In Figure 9.3.1 for the 12 full years (January – December) that were digitised the phase and amplitude of the 37 most relevant components is given. In the theory of tides, the different components are addressed by a code of letters and numbers. Thus the contribution from the Moon is indicated by the letter ‘M’ and the number ‘2’ indicates that the tides are semi-diurnal i.e. two tides per day. By inspection from Figure 9.3.2 it is evident that the greatest contribution to the tides at Dublin is as a result of the influence of the gravitational pull of the moon. This component reflects the main tidal cycle with a period of about 12.5 hours.

In Figure 9.3.2 the behaviour of the amplitude of the M2 component is given for the digitised monthly data from 1923 to 2002. For the years that were completely digitised all 12 monthly values are shown in this figure. The annual average of the M2 component has been calculated for the years digitised. Between 1943 and 2003, the annual average amplitude of M2 ranges from 1.32m to 1.36m, with a typical variation of 0.10m being exhibited in any one year.

The distribution of the annual M2 components suggests that there may be some sinusoidal fluctuation with a large period (about 100 years), however, there is insufficient evidence to confirm such a trend on the basis of the digital records analysed. All paper records analysing improve this.

The phase of the M2 component is illustrated in Figure 9.3.3. The arrangement of this figure is similar to that of the previous figure. On a month by month basis the variations in phase are in the order of 10 degrees (between 320o and 330o). There is one exception found: the year 1933. The phase of the (single) month that was digitised in this year drops to approximately 310°. For the M2 component a phase shift of 10 degrees is approximately equivalent to a shift in the time of high water by 20 minutes. The reason behind the deviation in 1933 could not be traced.

Notwithstanding the above, the overall conclusion arising out of the analysis of the harmonic constituents is that the magnitude and phase of the components as they relate to the tides at Dublin Port, has not changed significantly in the 80 years preceding 2003. **Thus, for the purposes of evaluating and predicting extreme water levels the major contribution of the astronomical tide (as demonstrated by the M2 component) can be considered to be constant.**

9.3.3 Historical Development of Mean Sea Level

MSL is usually described as a tidal datum that is the arithmetic mean of hourly water elevations observed over a specific 19-year cycle. This definition averages out tidal highs and lows caused by the changing effects of the gravitational forces from the moon and sun. In this study MSL is determined per year. So, it must be kept in mind that fluctuations related to the 19 year cycle may still be present in the annual MSL data.
The observed Mean Sea Level (MSL) is a relative level i.e. it gives the local land level relative to the local sea level. Therefore any changes in MSL over the years will reflect the combined effects of sea level rise and local land rise (or subsidence). Mean Sea Level (after corrections have been applied for changes resulting from the shift from Old Port Datum to that of Lowest Astronomic Tide) has been analysed in a similar fashion to the tidal components. The datum change referred to above – a difference of 0.23m – occurred in the mid-seventies. The results of the analysis after correcting for the change in datum are shown in Figure 9.3.4. The result shows a variation in MSL of 0.5m when viewed month-on-month, which is due largely to seasonal effects of meteorological conditions such as the effects of wind and barometric pressure. Figure 9.3.4 also includes the trends in MSL with the meteorological effects filtered out. In this situation the difference in MSL is reduced to 0.3m i.e. 2.32m LAT to 2.52m LAT.

Although the study has not used the complete eighty years of data in the analysis, a trend is nevertheless apparent from the plots. Linear trend lines have been plotted against both data sets included in Figure 9.5, with the result that an annual rise in MSL of 0.3mm/year is deduced.

A more detailed investigation into the results of historic rise of MSL, including comparisons with other work and data sets, is presented in Section 9.6, together with a detailed review of international best practice guidance on predicted sea level rise over the next century.

9.3.4 Seiches

Seiches have been observed very clearly on the Dublin Port tidal records. They are defined as periodic fluctuations of the water level with a period from a few minutes to a few hours.

Inspection of the tide plots show that seiches are recorded quite frequently by the Dublin Port tide gauge. For example, on 1st of February 2002 the tide record showed a double peak occurring on the rising tide, with the seiche occurring before the actual high tide.

A detailed analysis of the tide records for the period 2000 – 2002 was undertaken using the data collected from the digital tide gauge. An example of the result of the analysis is given in Figure 9.3.5, in which the water level trace for the first week of February 2002 is plotted. It is clearly seen that during the afternoon of 1st February 2002, a seiche occurred prior to the main high tide, having an amplitude of approximately 0.1m. In the same figure a severe seiche can be observed two days later during the morning of 3rd

February 2002. On this occasion the amplitude of the seiche exceeded 0.4m, although the seiche itself coincided with a low tide and therefore did not have any deleterious impact. During the afternoon high tide the seiche was still active, although the amplitude at high water had reduced to between 0.10m to 0.15m.

Figure 9.3.5 shows clearly that in the afternoon of February 3 the peak of the astronomical tide and the seiche coincide. This ‘matching’ of peaks results in unexpected behaviour of the water level. The rise of the water level after 12.00 hrs develops more or less ‘normally’. However, suddenly, the rise stops and the level remains almost constant for more than 30 minutes, before increasing suddenly by 0.40m over approximately 15 minutes.

The presence of the seiche in the tidal trace appears to occur randomly, in that there are no immediate explanations to be found in the examination of the meteorological conditions at the time. Any correlation with meteorological parameters will be discussed in Section 9.4.

For those three years (2000 – 2002) for which data was available in electronic format, a ‘seiche parameter’ was defined, as being the maximum seiche height of the day, after mathematical filtering of the data. Height is defined here as the difference between the local top and the following low, or twice the amplitude. Figure 9.3.6 illustrates the ‘seiche’ parameter for each of the three years for which the digital tide records were available. The gaps denote periods for which there were no records. From these figures the following can be concluded:

- Under ‘normal’ daily conditions the seiche parameter is approximately 0.10m. This level should be seen as ‘noise’ generated by the mathematical approach, rather than an actual seiche. Seiches only become significant if the height exceeds this level of about 0.10m.
- Significant seiches, heights more than 0.10 m, mainly occur in winter.
- In terms of seiches, the 1st February 2002 was not an extreme event; however, the surge that occurred on 3rd February 2002 was the maximum seiche recorded during the period 2000 - 2002.

Table 9.2 lists the highest twenty seiche heights for the period 2000 – 2002 are listed. The water level at the actual moment of occurrence of the maximum seiche of the day is also given. This table shows that the seiche may be present during any phase of the tide (at low, intermediate and high water levels). It is also noted that the seiche occurring on 1st February 2002 was not particularly high as is evidenced by its absence from the top twenty seiche heights.

Table 9.2 - Extreme seiche heights 2000-2003

Date	Time	Seiche (m)	Water level (m+LAT)
03-02-02	09:10	0.89	1.43
08-02-00	23:50	0.65	3.97
26-02-02	04:20	0.60	1.43
25-11-00	19:40	0.59	3.13
10-03-00	23:50	0.58	2.63
13-03-00	02:10	0.52	2.18
13-12-00	06:20	0.52	1.06
11-03-00	23:50	0.52	1.96
27-10-02	07:40	0.47	1.81
26-01-00	23:50	0.45	2.19
17-05-02	08:10	0.45	1.14
09-03-02	16:10	0.41	1.52
01-04-01	00:30	0.41	1.96
31-10-01	23:30	0.40	3.93
09-03-02	17:40	0.40	2.21
08-11-01	14:20	0.40	2.43
11-03-00	02:10	0.40	3.30
08-03-00	23:30	0.39	3.33
09-03-00	00:30	0.39	3.72
13-03-00	00:00	0.39	1.69
Top-20 extreme seiche heights 2000-2002 (3 years)			

9.3.5 Surge

In the context of this study, surge is defined as the observed high water level minus the hindcasted high water (based on astronomical components only)

To determine the occurrence of surges, all digitised data (see Section 9.2) was used, including the years with a single month of digitised data. The surge is determined as the observed high water level, minus the hindcasted high water level. The latter is based on the astronomical components found for 2002 combined with the annual average MSL of that specific year.

For example on February 2, 2002 a level of 5.46was reached at 14:40 and the hindcasted (astronomical) high water for the same tide is 4.50m. Note: this hindcast high water level has resulted from the analysis of the 2002 tidal records and is slightly larger than the predicted tide for the day given in the DPC tide tables. The resulting surge height is 96 cm. The hindcasted high water did not occur at the same time. The time difference does not effect the surge height.

The height of the extreme surges is given in the table below. It shows the 'top-20' surge list, together with the recorded total water level at the same time. It appears that the surge of February 1, 2002, is on the fifth position in the top-20. Highest surge occurred at 02.40 hrs on November 30, 1954, with a height of 1.28 m.

In the last two columns of the table the data is ranked in a different way. The 'top-20' of the water level is given there, together with the accompanying surge. Here, although the February 1, 2002 water level was the maximum the surge component was only the 5th highest identified.

Table 9.3 - Extreme surge heights and water level

Nr	Ranking on surge level				Ranking on water level			
	Date	Time	Water level (m)	Surge (m)	Date	Time	Water level (m)	Surge (m)
1	30-11-54	02.40	4.95	1.28	01-02-02	14.40	5.46	0.96
2	20-10-43	16.40	4.56	1.06	21-01-33	11.40	5.19	1.01
3	21-01-33	11.40	5.19	1.01	29-01-24	12.20	5.07	0.70
4	19-10-43	15.00	4.61	0.95	24-12-99	00.30	5.04	0.85
5	01-02-02	14.40	5.44	0.94	24-01-81	12.40	5.01	0.69
6	24-12-99	00.30	5.04	0.85	15-01-95	00.00	4.99	0.68
7	01-02-83	01.50	4.82	0.78	14-01-95	22.50	4.97	0.66
8	31-01-43	07.10	4.36	0.75	30-11-54	02.40	4.95	1.28
9	01-02-88	11.20	4.70	0.71	12-12-00	23.50	4.93	0.67
10	27-11-54	12.30	4.74	0.71	10-01-93	13.00	4.92	0.49
11	05-11-00	18.30	4.09	0.70	10-03-01	12.00	4.89	0.46
12	29-01-24	12.20	5.07	0.70	03-02-02	15.50	4.86	0.55
13	01-01-01	03.10	4.17	0.70	19-03-88	12.40	4.86	0.34
14	24-01-81	12.40	5.01	0.69	22-12-68	13.30	4.85	0.58
15	15-01-95	00.00	4.99	0.68	05-01-74	13.40	4.85	0.45
16	12-12-00	23.50	4.93	0.67	25-11-00	22.40	4.83	0.61
17	01-02-24	02.50	4.63	0.67	26-11-99	13.50	4.83	0.60
18	14-01-95	22.50	4.97	0.66	25-09-88	23.10	4.82	0.36
19	20-10-98	23.10	4.61	0.64	11-03-01	12.40	4.82	0.34
20	08-12-54	10.00	4.75	0.63	01-02-83	01.50	4.82	0.78
Top-20 extreme surge heights and extreme water levels								

It must be noted that the levels in this list may deviate slightly from the data that is used in other reports and that was derived from other sources. This has to do with the fact that the analysis presented here is done on the digitised records, and partly on the digital output of the recorder at Dublin Port. Both time series have a 10 minutes interval. It is always possible that differences of a few centimetres occur between this way of analysis and manual interpretation of tidal curves. Also, the fact that annual calculated MSL values are applied in this analysis may cause some deviation from other data sources. The surge levels may further be influenced by the choice of the astronomical components that is applied for the determination of the astronomical tide.

A detailed analysis of the extreme surges, and the resulting probability distribution of extreme water levels at Dublin Port, is given in Section 9.5.

9.4 Correlation between Surge and Other Meteorological Parameters

9.4.1 General

This chapter investigates the relationship between the residual water level fluctuations at Dublin Lighthouse (after eliminating the harmonic effects such as the astronomical tide) and wind speed and direction, barometric pressure and river discharge. Section 9.4.2 deals with the meteorological parameters of wind speed, wind direction, and barometric pressure, whilst Section 9.4.3 considers the potential impact of the discharges from the rivers Liffey, Dodder and Tolka.

9.4.2 Correlation between Surge and Meteorological Conditions

The following data was available for the correlation analysis:

Wind data: Wind data (speed and direction) was obtained from Dublin Port covering the period April 2000 - July 2003 with a 10 minute time step, averaged for 10 minutes. The observations were supplied for the anemometer located at the top of a 42.5m mast at the southerly end of Breakwater Road South. The information is part of the Vessel Traffic Control system. In the original data files the wind is presented in knots. For the purposes of the analysis the wind data has been corrected to wind speed in m/s at a height of 10m above MSL. An exponential wind profile distribution was assumed. In 2000 a substantial quantity of the wind records are without wind directions and therefore are of limited use. The data for the years 2001 and 2002 are suitable for the correlation analysis.

Barometric pressure: Hourly corrected barometer pressure records from Dublin Airport in units of 0.1hPa for the period 1998-2002. Since the analysis period is taken to be similar to the available wind data, only the period 2001-2002 is considered.

Water level: Digital readings of the water level observations at Dublin Lighthouse in 10 minutes intervals is available. The analysis of correlation requires data from all three data sets to be concurrent. For this reason only the water level data for the years 2001 and 2002 were considered. On further inspection the water level records of 2001 show some gaps in January/February and in July, whereas the year 2002 is complete.

Thus, based on the concurrency of the data sets and the completeness of the records the correlation analysis was carried out for the full calendar year of 2002. For this year the relation between water level (surge) and meteorological conditions has been investigated in a combined correlation analysis.

In this analysis a best fit relationship is determined between water level (surge) and a function of the following type:

$$\text{surge} = a\,(W_N)^x + b\,(W_E)^y + c\,bp$$

where:

W_N: north component of wind speed (northward is positive) [m/s]

W_E: east component of wind speed (eastward is positive) [m/s]

bp: barometric pressure fluctuations relative to mean air pressure [hPa] a, b, c, x, y: coefficients

The correlation is determined using a polynomial regression function with a least squares fit. A MATLAB program has been applied for this analysis, which uses as input the time series of all parameters. Each time series exists of 46,914 values with a time step of 10 minutes. If (small) gaps occur for one of the parameters, the same gaps are also introduced in the time series of the other parameters.

For preset values of the coefficients x and y, the output of the analysis gives the values of the coefficients a, b, c (and hence the water level related to wind and air pressure effects). Also the correlation coefficient is given.

Various values of the preset coefficients x and y were applied (1.0, 1.5 and 2.0) and the correlation coefficient for each of these coefficient was determined, with the best fit correlations being chosen as the representative values for the coefficients 'x' and 'y'. The analyses concluded that:

a) a value of 1.0 for the 'x' and 'y' values gave the highest correlation;
b) number of values where the moving average is based on: for both wind and air pressure, the selected average is based on 14 hours (7 hours before till 7 hours after).
c) time shift: the correlation is best when the water levels are correlated with wind and air pressure that occurred 7 hours before

Summarizing item b and c, the observed water level has been correlated with the average wind and air pressure of the preceding 14 hours. The resulting coefficients with maximum correlation coefficients are:

a = 0.0216
b = -0.0038
c = -0.01
x = 1.0
y = 1.0

The mean air pressure, as measured in 2002, is 1001.41 hPa

Hence the function given above can be re-written using the best fit correlation coefficients as:

$$\text{surge} = 0.0216\ W_N - 0.0038\ W_E - 0.010\ bp$$

This function is applicable to determine the contribution of wind and barometric pressure on the water level in Dublin Port.

For example, applying the above relationship to real values:

Wind speed - 15m/s
Wind direction - south east
Barometric pressure – 991.4hPa

Before substituting these values into the above equation, the wind speeds are resolved to their northerly and easterly components. The equation then reads as:

$$\text{Surge} = 0.0216\ (10.6)^{1.0} - 0.0038\ (-10.6)^{1.0} - 0.01\ (..) \quad = 0.354\text{m}$$

An example of how the relation works out in practice can best be seen if real values are introduced, for instance with the following preceding (14 hour) average wind and air pressure of :
>> wind speed 15 m/s
>> wind direction SE
>> barometric pressure 991.4 hPa
The input parameters for the surge function become then:
>> windN = 10.6 m/s
>> windE = -10.6 m/s
>> bp = -10.0 hPa

The resulting surge height is: 0.0216 x 10.6 – 0.0038 x (-10.6) – 0.010 x (-10.0) = 0.354 m.

From the factors 'a' (being +0.0216) and 'b' (being -0.0038) can be seen that the north component of the wind has more influence on the water level at Dublin than the east component. Southern and eastern winds contribute to water level rise, the opposite directions cause reduction in water level. The effect of southerly wind is much stronger than the effect of easterly wind.

The result of the correlation analysis regarding air pressure is that there is a direct relation between the water level and air pressure fluctuations. The fact that the computed value of coefficient c (being -0.010) is negative means that a rise in air pressure corresponds with a decrease in water level (the water is pushed down then) and vice versa.

The value of coefficient c corresponds to a linear relation with the pressure at the water surface: 1 hPa of air pressure corresponds with the weight of 0.01 m of water.

In Figure 9.4.1 the time series of each of the input parameters in the analysis are given, as well as the water level surge before and after correction for the meteorological effects. It can be seen that the weather parameters explain about half of the surge that occurred on the 1st of February 2002.

The other half can not be explained in this way. It is still possible that the remaining part of the surge is also caused by meteorological conditions, but from a remote location (for instance meteorological effects at the Atlantic Ocean may give a surge that travels away from its origin and enters into the Irish Sea).

9.4.3 Correlation between Surge and River Discharge

Fresh water discharges from the rivers Liffey and Dodder may have an impact on the water levels in the harbour and therefore on the water levels recorded by the tide gauge in Dublin Port. In addition to the influx of water, density differences between the fresh river water and the saline water can also affect the water level.

The discharges of the rivers Liffey and Dodder have been correlated with the water level surge at the Dublin Port tide gauge for the year 2002. Two cases were examined; one using the full surge height and the second using the surge height after the meteorological effects had been filtered out (see previous sections).

In the correlation analysis a number of thresholds on the river discharge is considered. Figure 9.4.2 gives the result for threshold level 60 m3/s (discharge of both rivers together). This threshold appeared to give the best results.

Two conclusions can be drawn from Figure 9.4.2:

- There is a positive relation between the river discharge and the full surge height (before correction for meteorological effects): The surge increases with increasing river discharge. A (combined) river discharge of 120 m3/s for instance is likely to occur simultaneously with a surge of 0.20 m.
- There is no relation between the river discharge and the reduced surge (after correction for meteorological effects).

In other words: there is a dependency between river discharge and water level at Dublin Port Lighthouse, but this dependency is not caused by hydraulic factors. It only says that high river discharge and bad weather tend to coincide. **The dependency goes through the air, not through the water.**

9.5 Analysis of Extreme Water Levels at Dublin Port Tide Gauge

9.5.1 Introduction

A good understanding of the extreme water levels to be experienced over the study area is necessary in assessing the level and standard of protection of the flood protection assets. It is also important for planning purposes in:

a) identifying which areas should be prioritised for emergency works and capital expenditure; and
b) assessing future development plans or regeneration potential of land adjacent to coastal or river defences

In this chapter the extreme value distribution of the water level at Dublin Port is determined using different methods.

In Section 9.5.2 the results of analyses based on the extrapolation of the high water level observations at the Port are given. Several methods and the differences in terms of extreme values are discussed.

Section 9.5.3 details the analysis of the extreme water levels using a joint probability approach [Ref 9.2]. Analyses using joint probability approaches are complex and require large data sets however, they are generally accepted as being more robust and accurate.

The last section summarises the results of the different methods and comes with conclusions and recommendations on the extreme water levels to be used for the Dublin Port (Lighthouse) tidal station.

9.5.2 Extreme Value Analysis of Recorded High Water Levels

For the Dublin Port tide gauge a summary sheet is available giving the maximum annual water levels recorded since the start of records in 1923. Up to and including 2002 a total of 80 high water levels are available. All were checked and reduced to a common datum; chosen to be Lowest Astronomical Tide (LAT). There is a risk that some of the years that were not digitised also have a datum error. It is only possible to detect this by digitising each specific month and doing a harmonic analysis on these months.

The list of annual maximums is given in Table 9.4 below.

Table 9.4 - Maximum Annual Water Levels Recorded at Dublin Port

Year	Level m (LAT)	Year	Level m (LAT)	Year	Level m (LAT)
1923	4.74	1951	4.83	1979	4.75
1924	5.10	1952	4.74	1980	4.69
1925	4.71	1953	4.65	1981	5.05
1926	4.74	1954	4.98	1982	4.90
1927	4.62	1955	4.80	1983	4.84
1928	4.65	1956	4.62	1984	4.76
1929	4.53	1957	4.74	1985	4.70
1930	4.74	1958	4.86	1986	4.70
1931	4.92	1959	4.92	1987	4.78
1932	4.77	1960	4.83	1988	4.86
1933	5.18	1961	4.89	1989	4.94
1934	4.71	1962	4.95	1990	4.96
1935	4.92	1963	4.74	1991	4.82
1936	4.95	1964	4.71	1992	4.88
1937	4.77	1965	4.65	1993	4.90
1938	4.80	1966	4.83	1994	4.92
1939	4.68	1967	4.83	1995	5.00
1940	4.65	1968	4.83	1996	4.76
1941	4.86	1969	4.71	1997	4.85
1942	4.74	1970	4.74	1998	4.72
1943	4.80	1971	4.65	1999	5.04
1944	4.86	1972	4.77	2000	4.80
1945	5.04	1973	4.71	2001	4.81
1946	4.71	1974	5.04	2002	5.46
1947	4.80	1975	4.86		
1948	4.83	1976	4.74		
1949	4.77	1977	4.83		
1950	4.80	1978	4.61		

To this list a first order fit was applied on the basis of log-linear extrapolation. This fit was done using all annual maxima since the start of records

The results of each of the above analyses are shown both diagrammatically and in tabular form in the Figures 9.5.2 to 9.5.4 and Table 9.5, below summarises the findings.

As can be seen from Figure 9.5.2, the analyses shows the event of 1st February 2002 to be off the line of best fit i.e. on the high side of the line. By using the log-linear fit as basis, in fact we bring down the return period as observed (once in the period of records, 80 years) to a higher number (even 200 years can be achieved). To justify this way of interpretation, there should be additional reasons that the extreme event is indeed more extreme than “once per observation period”. If not, the “artificial softening” of the event has no significant background, and it should be considered with care.

It should also be noted when applying the annual maxima method, that in any one year two or even more independent high water levels may occur, each of which may be higher than the highest water levels recorded in previous or subsequent years. For example, in 1924 a maximum water level of 5.10m LAT was recorded. This year could also have recorded water levels of 5.05m LAT or 5.00m LAT. These events would belong to the “tail” of the curve, but they are ignored by this method as there is a higher water level recorded for that particular year.

This limitation of the annual maxima methodology is eliminated by Kirk McClure Morton for an earlier study undertaken by then but not related to the DCFPP. However their draft feasibility report of July 2003 for the investigation of flood risk along the Royal Canal [ref. 9.3] presents the findings of their work which involved taking all extremes above a threshold of +4.0m LAT over a 22 years period between 1980 and 2002. The method used by KMM is known as a peak over threshold analysis. As with the three analyses listed above, the KMM analysis uses a log-linear fit to their data set. The results are displayed in the table below for comparison.

The recorded water level on 1st February 2002 was 2.95m ODM (5.46m LAT). By inspection both methods give similar results with the 1st February tide having a return period of between 1:100 and 1:200 years.

Table 9.5 - Extreme high water level

Return period (years)	Method 1. All annual maxima	KMM Peak over threshold
2	4.78	4.84
5	4.91	4.92
10	5.02	5.12
20	5.12	
25		5.23
50	5.26	5.32
100	5.37	5.40
200	5.47	
500	5.61	
1000	5.71	
Extreme high water level Dublin Port Lighthouse (m + LAT)		

9.5.3 Extreme Value Analysis of Tides and Surges

The basis of the joint probability technique is the fact that the water levels at coasts are determined by two major factors:

- the astronomical tide; and
- other effects which are summarised as 'surge' for the purpose of the analysis

Each of these factors are proven to be independent of the other and therefore behaves differently in terms of extreme values. See Section 9.4. This allows them to be analysed separately. Once the behaviour of each factor is known, the extreme values can be combined to give a joint probability of occurrence for differing return periods. This whole exercise is described in the following steps:

Step 1 - Analysis of the extreme astronomical tide

In the analysis of the astronomical tides in Section 9.3, it was concluded that the year-on-year variations in the tidal components and in MSL are not significant, thereby enabling a single year of data (in this case 2002) to be used as the basis of the extreme analysis of the astronomical tides. The results of the 2002 harmonic analysis (astronomical amplitudes and phases) has been combined with the average MSL, which was taken to be 2.43m LAT. Note that the 2002 MSL was 2.52.

With these data a time series of 19 years of astronomical tide was generated. In this way the full astronomical tidal cycle of 18.6 years is covered. The tidal cycle is predominantly determined by the periodic fluctuations in the position of the earth about its axis.

The full 19 year tidal sequence was used in the analysis of the extreme tides. The resulting highest high water levels are shown in Figure 9.5.5

The astronomical tide of 1st February 2002 was 4.42m LAT which, from Figure 9.5.5, has a relatively high probability of exceedance; 0.13m below the Highest High Water of the 19 year period. There are about 120 astronomical high waters in the 19 years period that are higher.

Step 2 - Extreme Value analysis of Surge Levels

As previously stated in Section 9.2, twelve years of historic water level observations are available in digital format, being 1943, 1954, 1968, 1973, 1983, 1988, 1993 and 1998 to 2002. For these years the surge level at each high water was determined by deducting the calculated astronomical high water level from the observed high water level. The surge as determined in this way may include contributions from seiches, low barometric pressure and wind set up. There may also be localised effects not covered by the above, but these are likely to be small in magnitude and therefore are not considered significant. The individual contributions from these factors are not treated as significant with respect to each other. In other words the effect of sieches is not given any greater weighting than say barometric pressure. As with the previous analyses the surge data are plotted using a log-linear scale in Figure 9.5.6.

From Figure 9.5.6, it can be seen that a surge of about 0.45m occurs approximately ten times per year, whilst a surge occurring on average once per year has a magnitude of approximately 0.65m. Moreover from the data analysed the highest surge observed was 1.28m, which occurred on 30th November 1954. The event of 1st February 2002 therefore belongs within the top-5 surge events, but it is not the most extreme.

It has to be noted here that above mentioned surges are related to the difference between observed high water and predicted high water. Both High water do not necessarily occur on the same moment. If we subtract the time series of observed water level and astronomical water level than the obtained residue includes timing errors as well. Higher residue values can be observed than in the period between High and Low water

Figure 9.5.6 also exhibits a discontinuity in the extreme trend of the surge line at a level of 0.60m–0.65m. Lower surges follow a slightly concave line, while the more extreme surges are more-or-less linear and will a much shallower gradient.

The reasons behind this change in behaviour are not clear, although similar behaviour in other parts of the world may provide the basis of an explanation.

- In areas where tropical storms occur, a similar change in surge behaviour is well known. In India for instance, the majority of time the statistics are dominated by monsoon weather. From time to time a hurricane passes through which completely disrupts the ‘normal’ pattern. Hurricane conditions are usually very severe and thus ignoring them (which could happen if only one or two hurricanes were present in the data set) would result in serious underestimation of the design conditions.
- In Dublin there are no tropical storms or hurricanes. However, changes in weather patterns cannot be excluded. Normal daily conditions might give rise to surges that reach a maximum of about 0.70m. This is the basis of the curved line in Figure 9.5.6. Severe storm depressions crossing the Atlantic from West to East might result in different surge behaviour. The complete change in the surge curve at around the 1:1 year return period suggests that severe storms may contribute to this change. There is insufficient data at this stage with which to make an informed judgement, and further research is recommended. This should look more widely at the frequency of storms passing across the Atlantic and crossing in the vicinity of Ireland.

The high end of the surge curve in figure 9.5.6 is used for extrapolation to arrive at the values for extreme surges. This extrapolation is done by applying a first order log-linear extrapolation. The extrapolation line is shown in Figure 9.5.6.

The curve of occurrence of surges, inclusive of the extrapolated extreme end, is used in the joint probability analysis.

9.5.4 Joint Probability Analysis of Extreme Tides and Surges

In the joint probability technique both the astronomical tide and the surge components are combined, assuming that these parameters are statistically independent. This assumption is valid since the mechanisms which give rise to each are independent. For example there is no correlation between the phase of the moon and the occurrence of storms.[Pugh and Vassie, applications of the joined probability method for extreme sea level computations, Proc Inst. Civ. Engr. Part 2, 69, Dec, 959-975)

For both parameters the full range of levels is used in the analysis and is not confined to the extremes. In this way the joint occurrence of a storm and neap tide is properly taken into account. To combine the probabilities, a matrix was set up in Excel with fine intervals on both tide and surge. The probability of occurrence of each cell in the matrix is determined by multiplying the probabilities of occurrence of both parameters for that specific cell. The matrix is than sorted on combined water level (astronomical tide and surge) and then the occurrence frequencies of all events giving rise to the same combined water level are added together.

The result is shown in Figure 9.5.7. The combined probability curve is no longer a straight line on log-linear scale. The curve varies in accordance with the joint probabilities as calculated.

The average annual MSL as observed in the analysed data varies between 2.32m-2.52m LAT. The high water analyses have been executed assuming a MSL of 2.42 m. For this reason two lines are included in the Figure 9.5.7 to reflect the uncertainty in MSL.

The table below summarises the results of the joint probability analysis.

Table 9.6 - Joint Probability Extreme High Water Levels

Return period (years)	Joint probability water level
2	4.86
5	4.99
10	5.12
20	5.26
50	5.42
100	5.54
200	5.64
500	5.75
1000	5.82
Extreme high water level Dublin Port Lighthouse (m + LAT)	

From the table it is seen that event of 1st February 2002 has a joint probability return period of approximately 1:60 years.

9.5.5 Discussion

In the sections above several different methods were applied to arrive at an extreme water level distribution for the tide levels at Dublin Port.

The traditional straightforward methods of fitting a log-linear curve to a series of annual maxima were compared to a peak over threshold approach employed by consultants Kirk McClure and Morton (KMM). The KMM analysis was not part of this project.

The conclusion of these analyses, is that the February 2002 event was an extreme event having a return period in excess of 100 years but less than 200 years. It is implicit in all of the straightforward methods that all processes contributing to the occurrence of an extreme event are independent. As a result, an event like 1st February 2002, "jumps out of the line" or in other words this event lies above the log linear line, fitted to all yearly maxima's, or the log linear line fitted to all peak over threshold values.

The joint probability analysis of tide and surge demonstrates that the astronomical tide on the 1st February 2002 was a severe spring tide but not the worst recorded in recent years. The same approach applies to the surge analysis: The 1st February 2002 surge was greater than the 'once per year' surge, but was not the highest recorded.

The February 2002 event in terms of likelihood of occurrence, fits very well within the joint probability analysis.

The extreme water levels as given in the table in Section 9.5.5 are the best estimate on the basis of the presently available information.

New data may define the limits more precisely, but this is beyond the scope of this study.

10 NUMERICAL MODELLING – OVERVIEW

10.1 Introduction

The Dublin Coastal Flooding Protection Project requires consideration of the interaction between fluvial and tidal components to assess the relative contribution from these to the level of flooding. To examine these, the study has utilised a range of numerical models. More detailed descriptions of the methods used with each of the models are given in the sections that follow.

Within this chapter, a brief description of each of the models is given along with an outline of the data collected and over view of how the models proposed and the methods used will combine in the development of the flood forecast system.

Numerical models are required:

- for the generation of transfer matrices that enable offshore wave conditions to be converted to inshore conditions;
- to reproduce the tide conditions along the coastal frontage of the project area including transfer of surge predictions from offshore to inshore;
- to investigate combinations of river discharge and water level;
- to provide predictive information for the operation of a coastal flood warning system; and
- to aid in identifying and assessment flood risk areas through the use of the modelling results.

10.2 Numerical Models

10.2.1 SWAN Wave Transformation Model

The modelling of wave conditions was carried out using the 3rd generation wave model SWAN (**S**imulating **WA**ves **N**earshore), which was developed by the Technical University of Delft and supported until recently by research funding from the Office of Naval Research in America as part of a research and development programme to improve the understanding and representation of the nearshore processes in numerical models. The SWAN model is able to represent the following physical processes:

- Wave propagation in time and space, shoaling, refraction due to tidal currents and depth;
- Frequency shifting due to currents and non-stationary depth;
- Wave generation by wind;
- Interaction between groups of waves i.e. tri-wave and quad-wave interaction;
- Whitecapping, bottom friction and depth-induced breaking;
- Wave-induced set-up; and
- Transmission through and reflection from obstacles.

SWAN cannot represent wave diffraction and scattered reflections.

Operating on a digital representation of the seabed bathymetry, SWAN computations can be made on a regular (square or rectangular) grid or on a curvi-linear (curved grid). In the case of the DCFPP, a regular rectangular grid has been superimposed over the model bathymetry in order to reduce the computational effort needed to generate the large number of model runs required to produce the transfer matrices.

The full extent of the SWAN model as set up for the DCFPP is shown in Figure 12.1, Appendix L.

10.2.2 FINEL2D – Tidal Model

To reproduce tidal conditions across the project area, the 2-dimensional FINEL model was used (**FIN**ite **EL**ement) and operated on the same bathymetry as the SWAN model described above. The use of the finite element technique allows for a very flexible representation of the complex sea bed geometries in Dublin Bay, particularly those of the Burford Bank and Kish Bank. The full extent of the FINEL model is shown in Figure 11.1, Appendix K.

Although the finite element technique requires more computational effort, the advantage of the use of the finite element method for representing the seabed bathymetry lies in the ability to increase the density of the elements in areas of particular interest.

The seaward boundaries of the model have been chosen to coincide with the UK Met Office Shelf Seas model output points (nodes), see Figure 11.3, Appendix K. This makes it possible to export data directly from the Shelf Seas model to feed into the boundary of the FINEL model.

10.2.3 ZWENDL - River Model

ZWENDL (a Dutch acronym for salt and water movement) is a 1-dimensional hydraulic model developed by the Rijkswaterstaat (the Dutch equivalent of the Office of Public Works). The area of interest is represented by a network of open water flows with currents distributed equally over the cross-section.

ZWENDL calculates:

- Water levels;
- Discharges;
- Salt water concentrations; and
- Currents.

ZWENDL can also incorporate hydraulic structures such as bridges, weirs, sluice gates, many of which are present on the Rivers Dodder and Liffey.

Figure 13.1 in Appendix M, shows the schematisation of the river model for each of the rivers. From the figure it can be seen that all of the main river within the study area, the Rivers Liffey, Dodder and Tolka, as well as the Royal Canal and Grand Canal, are integrated in one model with its downstream boundary at the end of the Dublin Port training walls. This is a considerable advantage to have three separate models of each of the main rivers and provides a much more versatile and useful model.

10.2.4 AMAZON – Overtopping Model

The AMAZON model used in this study is a 1-D wave overtopping model developed by Haskoning and Manchester Metropolitan University. The model solves the shallow water equations of wave propagation and run-up providing time series changes in water levels and depth averaged velocities using random waves as a boundary condition.

AMAZON can model wave propagation over complex and rapidly changing bathymetry and is applicable to any beach or revetment profile, including vertical walls. A coarse grid can be generated at the start (offshore end) of the model profile and can be made finer across the profile and at the structure. The model forecasts the instantaneous, mean and peak overtopping rates and has been used for evaluating wave overtopping in many locations around the UK and Ireland.

For the DCFPP it has been used to model the overtopping at various locations around the project area for a wide range of nearshore wave conditions. The nature and location of these model runs is described in more detail in Chapter 12. A screen dump showing the model in operation for one of these locations is presented in Figure 10.1 below.

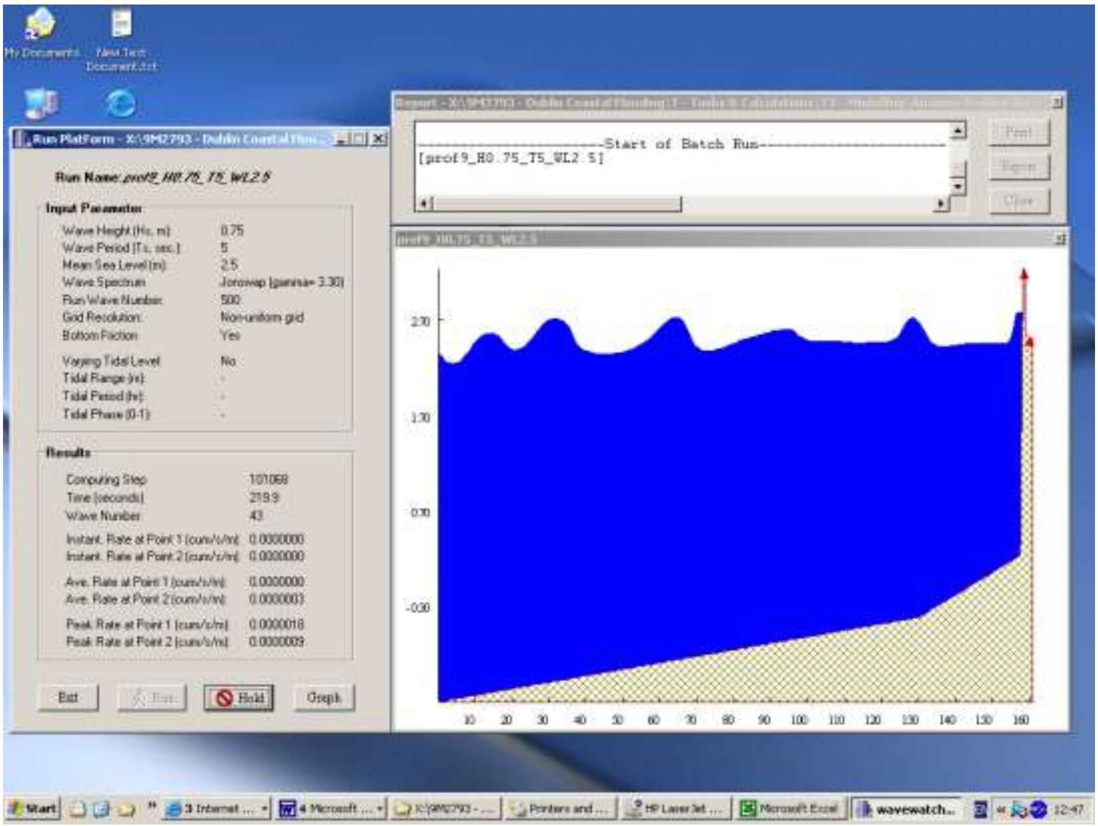


Figure 10.1 - View of Front End of AMAZON Overtopping Model

10.3 Data Collected

An extensive amount of data was been collected throughout the study which has been invaluable in the setting-up, validating and running of the models described above. A detailed listing of the data collected for this project is given in Table 8.2, Appendix D.

In establishing the models for the study, the most useful information came from existing and specifically commissioned topographic and hydrographic surveys, hydrological data (including remote gauging), and from several study reports covering the coastline and rivers within the project area.

The important data elements can be summarised as follows:

Coastal and river bathymetry:

- Liffey Bathymetry Plans and Bridge cross-sections from East Link Bridge to Heuston Station.
- Aerial survey of the mudflats between the Port and Clontarf, 2001.
- Bathymetric Survey of Port Medway approach channel, 2001.
- Plan of Dublin Port and table showing berth locations and working depths.
- Dublin Bay Project – Bathymetric details over the pipeline orientation.
- Dublin Bay Project – Burford Bank survey.
- Dublin Bay Project – North Bull Island and Sutton Strand survey.
- Baldoyle Estuary and Velvet Strand bathymetric details.
- River Tolka Study model bathymetry and information.
- Topographic survey of the coastline and foreshore
- Topographic and bathymetric survey of the River Dodder from the River Liffey upstream to Ballsbridge Wier

Hydrological data.

- River Gauge Data at:
  - Waldrons Bridge – River Dodder
  - Botanic Gardens and Dromcondra – River Tolka
  - Lexlip – River Ryewater
  - Dublin Rainfall Time Series data
  - Various reports on the river catchments

10.4 UK Met Office Forecast Models

10.4.1 Storm Surge Model

The two-dimensional Met Office Storm Surge Model is run operationally, and provides a 36 hour forecast twice daily for British and Irish waters. The aim of the model is to warn against storm surges around the British and Irish coastline, combining tidal heights with water elevation produced by wind effects. The Surge model is suited only to areas of shallow water (<120m depth) where the water column is well mixed.

10.4.2 Shelf Seas Model

The operational Shelf Seas model is a three-dimensional model capable of representing the effects of temperature and salinity and able to resolve vertical current structure both on the shelf and at the shelf break and beyond. The model was written by the Proudman Oceanographic Laboratory (POL), Birkenhead and has been tested by them in many different configurations. It has been running operationally at the Met Office, in one form or another, since June 2000.

The Atlantic Margin Shelf Seas Model (the main operational model which produces output for the European continental shelf) does not replace the Surge model rather it complements it, as the former model is valid in regions where the latter is not, e.g. areas of deeper water or near the shelf break. In addition, the Shelf Seas Model also provides salinity and temperature information as well as currents and sea surface elevations and is fully three-dimensional.

The model is run on the same grid as the operational surge model across the NW European Continental Shelf; i.e. the geographical area of 12W-13E, 48N-65N at a resolution of approximately 12km (which corresponds to 1/6th of a degree by 1/9th of a degree). This region has now been extended to the south and west, reaching to 20W and 40N.

The model is forced both at the surface and at the open ocean boundaries. The atmospheric forcing is by wind stress, sea surface pressure, heat and precipitation minus evaporation. These fluxes are taken from numerical weather prediction (NWP) models and further details and examples of the atmospheric fluxes can be found here.

At the open boundaries to the Atlantic, forcing is provided by a larger scale circulation taken from FOAM (Forecast Ocean Assimilation Model), another operational model run at the Met Office which deals with the deep ocean. A radiative condition is also applied at the open boundaries, allowing outflow and inhibiting reflection back into the model domain.

In addition to this, the Shelf Seas Atlantic Margin Model also includes 36 rivers and other sources of freshwater inflow including the Baltic Sea.

The Irish Sea model is nested into the Atlantic Margin model. It runs at a resolution of approximately 1 nautical mile (1/60th of a degree by 1/40th of a degree) and has been operational at the Met Office since March 2003.

10.5 Forecast System

A major deliverable of the Dublin Coastal Flooding Protection Project is the tidal flooding forecasting system which will allow advance warning of potential flooding to be received.

The forecast system comprises several elements, each of which contributes to the final forecast at the coastline. These are:

- Input from the UK Met Office Shelf Seas and Storm Surge models;
- Wave Transformation matrices;
- Overtopping matrices;
- Water level prediction matrices; and
- Tidal water levels from the Alexander Pier Lighthouse tide gauge.

The surge predictions are received twice daily from the UK Met Office (UKMO) is in the form of hourly predictions for a period of 36 hours ahead. This information is currently generated at 0200 hrs and 1400 hrs each day. The information received which comprises:

- Wave height
- Wave direction
- Wave period
- Total tide level i.e. astronomical tide level plus surge component
- Residual surge height; and
- Wind speed & direction.

goes directly to the Dublin City Council server where it is stored until a call is made by the forecasting system for the most recent data set. Thereafter the data is passed through the matrices mentioned above, to arrive at a forecast of potential flooding for different locations along the coastline.

Exceptions to this process are found in the Rivers Liffey and Dodder and the area behind Bull Island. The information received and processed is presented on a user friendly system operated by the client's staff. It is important to note that the forecast system does not replace independent action by operational staff. Until fully validated the forecast system and any warnings issued as a result of the input received and interrogated, must be reviewed and assessed by a member of the operation staff.

Figure 10.2 presented in Appendix J shows a schematic representation of the integration between the UK Met Office forecast models and the DCFPP models for the development of the overall flood forecasting system.

11 TIDAL HYDRODYNAMIC MODELLING

11.1 Introduction and Methodology

11.1.1 Introduction

Chapter 10 presented a brief overview of the models used within the DCFPP and their relevance and use in developing a coastal flood forecasting system. This section describes in some detail the development of one of those models, the tidal hydrodynamic model. It describes the investigation work undertaken, presents the results of the modelling work and demonstrates how those results are used to form one of the main components of the proposed coastal flood forecasting system. The component of the forecasting system in question is the reproduction of the tidal conditions around the study area and the transfer of surge from an offshore point to the coastline. The model used to investigate this was the 2 dimensional finite element model FINEL.

11.1.2 Methodology

An overview of the methodology that underpins the mathematical tidal modelling is outlined in the following steps:

- **Step 1.** Setup the hydrodynamic model using the supplied bathymetry and boundary conditions from the UK Met Office Shelf Seas Model
- **Step 2.** Generate tidal curve and calibrate against 1st February 2002 event.
- **Step 3.** Validate model against a separate high tide event. This was done using information from the UKMO for the period 10th – 11th March 2001.
- **Step 4.** Use FINEL 2D to generate extreme tide conditions and typical spring and neap tides. The latter is developed without any influence of wind.
- **Step 5.** Develop relationships between the tides generated at Dublin Port tide gauge and the selected prediction/warning points around the coastline of the study area.
- **Step 6.** Develop astronomic tides and surge components for each prediction/warning point for inclusion in a water level matrix.

The above steps are discussed in more detail in the sections that follow.

11.1.3 Tidal Hydrodynamics

For the 2-dimensional tidal modelling of the tidal movement in Dublin Bay, the FINEL2D model package has been used. This model is based on the finite element technique which enables very complex sea bed geometries to be represented. The inherent flexibility in the technique is due to the unstructured nature of the grid used to represent the sea bed i.e. there is no need to follow square or curvilinear grid patterns, which in turn allows more detail to be included in areas of specific interest. The finite element mesh used for this study is shown in Figure 11.1.

The FINEL model has recently been applied to study:

- the impacts on the Dutch coast resulting from the seaward extension of the Port of Rotterdam;

- the design of a number of terminals in India and Taiwan; and
- the design of flood reduction measures in the River Lek in Holland.

At the seaward extent the model boundaries are chosen along the grid lines of the UKMO Shelf Seas Model (the resolution of this model is 1/9° latitude and 1/6° longitude, which is approximately 12km by 12km). This makes it possible to export data from the Shelf Seas Model directly to the boundary of the FINEL model. Thus the FINEL model covers roughly an area that is bounded by 53.6° N latitude in the north, 53.1° N latitude in the south and 5.7° W longitude along the seaward boundary. The model includes the Burford and Kish sandbank systems in Dublin Bay and Lambay Island in the north.

11.1.4 Calibration & Validation

The calibration of the FINEL2D model was achieved by reproducing the 1st February 2002 event (see also the POL Internal Report No 146, “Note on the Storm & Floods on the 1st February 2002”.). Thereafter, validation was carried out by reproducing a second event of known magnitude. In the case of the Dublin tidal model, this was taken to be 10th -11th March 2001. This checking (or validation) calculation was done with exactly the same parameters as the calibration run. In this way, the predictive capabilities of the model can be proven.

The hydrodynamic behaviour of the bay is simulated in the FINEL model and also includes the effects of wind shear and barometric pressure.

11.1.5 Calculation Runs

The validated model is used to determine the combined effects of wind shear forces and astronomical tides. For this combination of factors a number of tidal events are simulated, each of them giving a unique combination of wind (direction and force) and tidal conditions. Note: at this stage no river discharges have been considered, as these are modelled separately.

The selection of the tidal events is based on the results of the probabilistic analysis of the water level recordings at Dublin Port Tide Gauge. In this way for each relevant water level the wind induced surge for each warning point is determined within the FINEL model.

In addition to these surge cases a complete spring/neap tide cycle, without wind, is modelled in FINEL to determine the astronomical tide for each warning point.

11.2 Model Construction

11.2.1 Finite Element Grid

An unstructured triangulated mesh was constructed over the bathymetry as shown in Figure 11.1. The nature of the unstructured grid is such that it allows areas of specific interest to be given greater definition through the use of a finer mesh. The elements become smaller closer to Dublin with special attention being given to the North and South Bull walls, which are lying in front of Dublin Port. It is understood that these walls play an important role in the movement of water within Dublin Bay, and hence their representation must be as accurate as possible.

11.2.2 Bathymetry

Each element of the computational grid requires a bottom depth. The bathymetric information used in the schematisation of the sea bed originates from:

- Admiralty chart, no. 1468, Irish sea;
- Admiralty chart, no. 1415, Dublin Bay area ;
- Dublin Bay Project, Pipe line crossing survey;
- Clontarf Mudflats Survey;
- Dublin Bay Project, Bull Island Survey;
- Dublin Port, Channel & Berth Survey, December 2001;
- DEEP survey (2003), intertidal area of Sandymount and Merrion strand.

The bathymetry created using the above information is shown in Figure 11.2.

Special attention is given to the height of the North Bull Wall. The eastern part of this wall is submerged during each high tide. The level of the most eastern tip of the wall has been estimated at around –1.11m Malin Head Datum (ODM) based on observations of when the wall became submerged and comparison of that time with the levels at the DPC tide gauge. The height of the wall gradually increases going to the west. Approximately 1km from the eastern tip of the wall the height is 0.89m ODM.

11.2.3 Boundary Conditions

Water level data from the UK Met Office Shelf Seas Model was purchased as part of the study, to provide boundary conditions for the FINEL2D model. Time series of water levels were obtained for each grid point of the Shelf Seas model in the vicinity of Dublin Bay. At the seaward extent the model boundaries were chosen to coincide with grid lines from the Shelf Seas model itself. In this way the general tidal movement of the area can be passed directly into the FINEL2D model.

Figure 11.3 shows Dublin Bay with the location of the UKMO prediction point, which will be used for the early warning system.

The rivers Liffey, Dodder and Tolka run into Dublin Bay. In Chapter 13 the significance of these discharges is discussed further, however, in the context of the tidal hydrodynamics the river discharges are not considered significant.

By this stage the model is ready to run. The model was then calibrated and validated.

11.3 Calibration and Validation of the FINEL Model

11.3.1 Calibration against 1st February 2002 Event

i) Astronomical tide, February 2002

A distinction is made between the astronomical tide and the measured tide, so a better insight is given into possible sources of errors. In this section the astronomical tide of February 1st 2002 is discussed.

Driven only by astronomical boundary conditions the model is run for the 48 hour period between 31st January 2002 and 1st February 2002. The calculated water levels and the astronomic tide are shown in Figure 11.4. The astronomic tide is derived from the statistical analysis of the historic tide records obtained from Dublin Port. When looking at the high water levels the difference is approximately 10cm - 20cm. The high water levels calculated by FINEL 2D are higher than the real astronomical tide. The low water levels are comparable.

ii) Observed tide, February 2002

The observed water level at Dublin Port Tide Gauge is reproduced by FINEL2D, using the boundary conditions supplied by the UK Met Office Shelf Seas model. These boundary conditions include an astronomic component and a surge component. FINEL2D also uses the observed wind data (speed and direction) from Dublin Airport for this period to calculate a local water level response. The wind direction was Southwest with a force 7 – 8 Beaufort (14m/s - 21m/s). This resulted in a local water level set-down in Dublin Bay (offshore wind). The set-down was only a few centimetres.

In Figure 11.5 the predicted and measured water level at Dublin Port tide gauge is shown. In the figure the astronomic tide calculated by FINEL2D is also shown. The high waters levels can be reproduced to an accuracy of less than 20cm and in most cases to within ±10cm. The high water of the February 1st event is predicted to within 10cm accuracy.

To assess the sensitivity of the predicted tide to changes in bed roughness (which is the major calibration coefficient), further runs have been made with a different bottom friction. The differences in water level between these runs were negligible. In total 4 runs have been made with different bottom frictions. The bottom frictions which were used for the calibration were chosen in such a way that they cover the possible range of the friction coefficient parameter. The water level differences between the runs were in the order of centimetres. It was therefore decided that the original choice of a bottom friction of 0.2m is good enough to reproduce the water levels for the calibration period.

iii) Seiches

As can be seen from Figure 11.5 the predicted water level at Dublin Port tide gauge shows the occurrence of seiches of +/-10cm with a period of approximately one hour. A similar pattern is also seen in the tide records obtained from Dublin Port, however, the mechanism causing these seiches is not fully understood. Additional runs of the FINEL model were conducted to investigate the behaviour of the sieche and from these runs it was concluded that:

- the seiche does not a occur as a result of the interaction of the tide with the bathymetry of bottom friction;
- the height and shape of the North Bull wall does not affect the presence of the seiche. It was found that by removing the wall from the model, the seiche still occurred;
- the timing of the tide did not affect the formation of the seiche. This was investigated by starting the model run 30 minutes after the start of the seiche i.e. half way through the seiche. Even with this artificial start, the calculated seiche still remained the same; and

- the seiche is not a function of the astronomic tide.

It is believed that the shape of Dublin Bay may be a contributing factor, which when combined with the presence of wind blowing across the Bay leads to seiches being recorded at Dublin Port Tide Gauge. Whilst the importance of these seiches must not be underestimated, it is recognised that they are a phenomenon of real-time conditions occurring in Dublin Bay, and as such cannot be fully predicted by the FINEL2D tidal model.

iv) Tidal Patterns in Dublin Bay during 1st February 2002 Event

Figures 11.6 to 11.17 show the hourly patterns of tidal movement predicted by FINEL2D between 08:00 hrs and 19:00 hrs on 1st February 2002.

The colours indicate the tide level, whilst the arrows are indicating the magnitude and direction of the tidal flow. When an element in the computational grid becomes dry the element is shown in white. Also shown in the left part of the figure is the tide curve at Dublin Port tide gauge, an indicator of where in the tidal cycle the plot is for, along with an indicator showing the present wind speed and direction.

The first plot (Figure 11.6) shows the predicted tidal conditions around low water at the Dublin Port. Much of the shallow areas in Dublin Bay are dry in the model, including the North Bull Wall. The arrows show a northward moving velocity, indicating a flood tide. Some of the shallow areas show a higher water level than the overall water level. This is caused by the fact that this water is trapped in the model, because the element that should lead the water to the open sea is dry.

The next maps (11.7 – 11.10) show a further rise of the water level. The maximum flood current occurs approximately two hours after low water, as can be seen in Figure 11.9. Dublin Bay is filled with water coming from the South. Approximately one - two hours before high water i.e. low water + 4hrs to low water +5 hrs (one o'clock on the 1st of February, Figure 11.11) the velocities begin to swing towards the south. By LW + 5hrs the direction of the tide has turned and this can be seen in Figure 11.11. The tide has reached its maximum by LW+ 6 hrs and thereafter the tide is on the ebb as shown in Figures 11.13 to 11.17. The strongest ebb currents occur approximately one hour after high tide (see Figure 11.13). In Figure 11.17, the direction of the tidal flow has again turned and the tidal vectors can be seen to be moving in a northerly direction once again.

A valuable source of information to validate the tidal plots is available in the Tidal Atlas of Dublin Port & Docks Board (1971). The Tidal Atlas gives hourly plots of velocity and direction for several locations in Dublin Bay. In the Tidal Atlas velocity measurements are shown from 1876 upto1971, however, it is not known under what circumstances these measurements are carried out i.e. combinations of tide, wind and surge. Hence a direct comparison between FINEL2D vectors and those in the tidal atlas is not possible. Nevertheless a global comparison between the two sources shows good agreement; especially at the times indicated for the turning of the tide. Therefore it may be concluded that the FINEL2D model satisfactorily reproduces the overall patterns of tidal movement in Dublin Bay.

11.3.2 Validation

In this section the validation of the FINEL2D model is described. The validation period is 10th -11th March 2001. Again a distinction is made between the astronomic tide and the observed tide.

i) Astronomic tide, March 2001

In Figure 11.18 both the astronomic tide and the astronomic water levels calculated by FINEL2D are shown for the validation period. The high water levels of FINEL2D are approximately 0.25m higher than the observed tide. The difference between the astronomical tide and the calculated astronomical water levels are higher in the validation period than in the calibration period when differences of 0.1-0.2m occurred.

ii) Observed tide, March 2001

Figure 11.19 shows the observed and predicted water levels for the validation period. The predicted high water level is approximately 0.2 – 0.3m higher in comparison to the observed tide. This discrepancy is equal to the discrepancy found in the astronomical tide levels, which suggests that primary source of the discrepancy lies in the astronomical boundary conditions supplied by the UKMO for this event. At low water the discrepancy between the predicted and observed tide levels is between 0.1m – 0.2m.

At low water a seiche is visible, in both the observed and calculated astronomical tide levels.

iii) Validation against tidal current measurements

In 1995 MCS International (1995) carried out a hydrodynamic and sedimentation study for Dublin Bay. As part of the study tidal current measurements were used to calibrate and validate a hydrodynamic model. However, there are no metadata available to confirm when these measurements were carried out. As a result the prevailing conditions at the time of the measurements i.e. wind speed, wind direction, tide level, and location are not known. It is only known that the measurements are for an average spring and neap tide. As a consequence any comparison between these measurements and the FINEL2D model can only be indicative.

Current speeds are measured at four locations. These are:

- Site 1: 792 metres east of Bailey Point;
- Site 2: Rosbeg Buoy;
- Site 3: Approximately 2.5 km east of Dun Laoghaire Harbour;
- Site 4: Approximately 2.5 km north of Dun Laoghaire Harbour.

FINEL2D was run for a complete astronomical neap-spring tide cycle. The measured and calculated velocities are compared in Table 11.1 below.

Table 11.1 - Observed and Calculated Tidal velocities

		Site 1	Site 2	Site 3	Site 4
Spring tide	Meas.	1 m/s	0.7 m/s	0.7 m/s	0.25 m/s
	FINEL	1 – 1.2 m/s	0.7-0.8 m/s	0.5-0.6 m/s	0.2-0.3 m/s
Average tide	Meas.	0.9 m/s	0.5 m/s	n.a.	n.a
	FINEL	0.8-0.9 m/s	0.6-0.7 m/s	0.4-0.5 m/s	0.2-0.25 m/s
Neap tide	Meas.	n.a.	n.a.	0.4 m/s	0.2 m/s
	FINEL	0.6 m/s	0.4-0.5 m/s	0.3-0.4 m/s	0.15-0.2 m/s

From Table 11.1 it can be seen that the calculated tidal velocities of the model lie within the range of measurements obtained from the MCS International study. In the light of the lack of precise data relating to the measured velocities, the FINEL2D model can be seen to calculate velocities to an acceptable level of accuracy.

iv) Validation of Other Tidal Stations

At Rogerstown tidal observations were carried out for a five-month period, from 15th February 1984 to April 1985. These tidal observations were compared with the Dublin Port Tide Gauge. When averaged over the period of the readings, the high water level at Rogerstown is approximately 21cm higher than at Dublin Port tide gauge. The results of the FINEL2D modelling show similar high water differences. For an astronomical neap-spring tide the average difference at the high water level is 18cm.

Taken together the comparison between existing and predicted tide level data indicates that the FINEL2D model is reproducing the pattern of tidal movement correctly.

At Dun Laoghaire tidal observations are also available, although they were obtained too late in the modelling process to be incorporated in the validation. However, since receiving the data it appears that there are doubts over the validity of the tide gauge as the tide trace exhibits considerable fluctuations across the top of the tide, a phenomenon also referred to as noise. The information has therefore not been used.

11.3.3 Conclusion

The calibration and validation of the model have been carried out successfully. The overall patterns of water movement in the area can be reproduced satisfactorily. During the calibration process the tides that occurred on 1st February 2002 were reproduced to an accuracy of 10cm. The validation event however was only possible to within 0.3m. Further investigation of this is believed to be due to the quality of the boundary condition data received from the UK Met Office for 10th – 11th h March 2001.

FINEL2D has been used to calculate an accurate spatial distribution from the astronomical water level at Dublin Port tide gauge to the warning points and for the spatial distribution of the local surge at the UKMO point to the warning points. The absolute water levels are of less importance in this process than the overall tidal movement.

The surge levels as predicted by the Shelf Seas model seems to be sufficiently accurate for both periods.

Another source of discrepancy in the predicted tide levels is due to the occurrence of seiches. The phenomenon of seiches in Dublin Bay is not well understood as no research has been carried out. Within the remit of this study, some investigation has been undertaken to determine whether they are affected by the shape of the port entrance or the harbour basins, but the results of the tests suggest that the seiche is a function of the shape of Dublin Bay itself. The tidal trace shows the seiche occurring relatively frequently and indeed the FINEL2D model also shows the seiche occurring. It has been noticed that a slight difference in the phase of the seiche when compared to the tidal trace can result in a level discrepancy of 10cm – 20cm.

The bottom roughness, which is the major calibration coefficient, is set at a Nikkuradse roughness of 0.2m.

11.4 Water Level Scenarios

11.4.1 Warning Points and Tide Scenarios

The prediction of water levels, and hence the occurrence or otherwise of overtopping, at points along the coastline other than at the Port of Dublin is a key requirement of this element of the modelling. These ‘warning points’ are shown in Figure 11.20.

The validated FINEL2D model was used to determine the combined effects of wind shear forces and astronomic tides for each warning point. For this a number of scenarios have been simulated, each of them giving a unique combination of wind (direction and force) and tidal conditions. Also a complete spring/neap tidal cycle will be carried out to determine the astronomic tide of the warning point.

The prediction of the water level at each warning point of the early warning system consists of:

1. The astronomic high water level at Dublin Port tide gauge. This is described in Chapter 9.
2. The difference of the astronomic high water level between Dublin Port tide gauge and the warning point. This is described in Section 11.4.2
3. The surge prediction of the UKMO Shelf Seas Model. For this a grid point of the Shelf Seas Model has been chosen in the vicinity of Dublin Bay which will give a forecast of the surge.
4. The local surge from the UKMO point to the Dublin Port Tide Gauge. For this 64 wind scenarios have been defined which will be calculated with the calibrated FINEL model. The scenarios include:
   - 8 wind directions (N-NE-E-SE-S-SW-W-NW);
   - 2 constant wind speeds i.e. 15m/s and 30m/s;
   - 4 water level scenarios (neap tide, spring tide, spring tide + 1.0m surge, spring tide + 2.0m surge).

   This is described in Section 11.4.3
5. The local surge from Dublin Port Tide Gauge to each warning point. Using the same scenarios as described above the local surge difference between the Dublin Port Tide Gauge and the warning points will be calculated. This is also described in Section 11.4.3

This approach is based on differences at the high water levels. All calculations will be based on the high water level of 1st February 2002 at 14:00 hr.

11.4.2 Astronomic Tide

The first step of the prediction of the water level in the warning points is the prediction of the astronomical tide. For this the FINEL2D model is run for 14 days, which is a complete spring/neap tidal cycle. For each warning point the difference at each high water between Dublin Port Tide Gauge and the warning point is calculated. A relationship is established between the high water level at the Dublin Port Tide Gauge and the difference of the high water level between the Dublin Port Tide Gauge and the warning point. The general idea is that when the high water level is higher the water level difference at high water between the Dublin Port Tide Gauge and the warning point is also higher.

In the Figures 11.21 till 11.25 the results of this exercise are shown for each warning point. Each high water is shown in these figures with a “x”. When fitting a straight line through these points a relationship is established for the prediction of the astronomical high water level of the warning point. The correlation coefficient for warning points close to the Dublin Port Tide Gauge don’t look good. This is misleading because the difference in water level between this warning point and the Dublin Port Tide Gauge is very small. The difference which can still be found (less than 1mm) are caused by rounding errors and numerical errors. Due to these very small differences the correlation coefficient is not good, while the prediction of the astronomical tide of that point is actually very good.

From these figures it can be noted that in general the astronomical high water level increases for warning points located to the North of the Dublin Port Tide Gauge. Warning points located to the south of the Tide Gauge show in general a decrease in astronomical high water level. Compare for example the relationship for warning point 26 (located South of Dublin Port Tide gauge) and warning point 27 (located North of Dublin Port Tide Gauge) in Figure 11.25. This is in line with the general tidal movement in the Irish Sea, which shows an increase in high water level going to the North.

The relationship, which is shown in the upper right corner of each figure, will be used to calculate the difference between the astronomical tide of the Dublin Port Tide Gauge and each warning point.

Please note that the relationship of warning point 12 in Figure 11.22 is based on only three points, because this warning point is located high in the inter tidal zone, so it is only flooded three times during the complete astronomical neap-spring cycle.

11.4.3 Local Surge

Local surge of the UKMO point to Dublin Port Tide Gauge

For each scenario the high water level difference between the UKMO point and the Dublin Port Tide Gauge has been calculated and presented in Figure 11.26. In this figure all the wind directions are shown together with the difference in high water level between the UKMO point and the Dublin Port Tide Gauge. Four different lines have been drawn, representing four different tidal conditions; neap tide, spring tide, spring tide +1.0m surge and spring tide +2.0m surge. These different tidal conditions have been chosen because the wind set-up reacts differently for different water levels. The general rule is that the less water in an area the more sensitive the area is to wind set-up/down. So at low water the most wind effect can be expected. Also in shallow areas the wind effect is expected to be higher than in deep areas.

In the Figures 11.32 till 11.34 the difference in water level at high water between the run with and without wind is shown for different wind directions for spring tide conditions. Clearly visible is the water level set-up when the wind is blowing towards the coast. Shallower areas have a large set-up in comparison with deeper areas. Figure 11.34 shows a situation with a wind coming from the west. The result is a water level set-down of several decimetres. The areas with the largest set-down are the shallow areas, since the wind has the most effect in these areas.

Figure 11.26 shows, for example, a water level set-up on a spring tide of approximately 0.35m at the Dublin Port Tide Gauge when the wind is coming from the east with an average wind speed of 30m/s. When the wind speed reduces to 15m/s the set-up decreases to approximately 0.09m indicating the sensitivity of the set-up to wind speed. When the wind is coming from the west with an average speed of 30m/s from 270 degrees, a set-down in water level of approximately 0.48m is noticeable under neap tide conditions. When taking the scenario with the same wind conditions, but for a spring tide plus a surge level of 2m the set-down becomes approximately 0.42m. So the set-down is dependant of the water level.

To determine the prediction of the local surge of the UKMO point to the Dublin Port Tide Gauge an interpolation has to be made between the results of the scenarios, since the realistic wind speed, wind direction and water level will not be exact the ones used in the scenarios.

<u>Local surge of Dublin Port Tide Gauge to the warning points</u>
Using the scenarios as described in 11.4.1 the difference between the Dublin Port Tide Gauge and each warning point is established. The outcome can be seen in the Figures 11.27 till 11.31. Please be aware that the water level differences shown in the figures are not the actual surge levels but the difference in surge between the Dublin Port Tide Gauge and the warning points.

From these figures it follows that the surge level in shallower areas is sensitive to water level differences. Compare for example warning point 1, which lie in relative deep water, versus warning point 16, which is located in the shallow Clontarf area that becomes dry every tide. In warning point 1 almost no difference can be seen for different water levels. Warning point 16 on the other hand shows a dependency of the water level. The largest difference between the high water level at warning point 16 and the Dublin Port Tide Gauge occurs at neap tide and a 30m/s wind, which might be expected.

When the wind is coming from the east with 30m/s a difference in surge of approximately –0.3m can be seen in Figure 11.27. This looks strange at first sight because the wind is generating a wind set-up at the Irish east coast, however this can be explained by the fact that the location of the Dublin Port Tide Gauge is more sensitive to wind set-up from the east than warning point 1, so warning point 1 shows 30 cm less surge. When looking at a wind from the West with 30m/s a surge difference of 0.2m can be seen. The wind is generating a water level set-down, however at the Dublin Port Tide gauge this set-down is 0.2m more than Warning Point 1.

When the wind is coming from the direction East - South an extra water level set-up can be seen for warning point 16. When the wind is coming from the direction West - North a water level set-down can be seen. From this figure it can be concluded that Warning Point 16 is more sensitive to on- and off shore wind than the Dublin Port Tide Gauge.

The warning points in the Liffey which lie close to the Dublin Port Tide Gauge show little difference (points 17, 18 and 19), as might be expected.

To determine the prediction of the local surge of the Dublin Port Tide Gauge to the warning point an interpolation has to be made between the outcomes of the scenarios, since the realistic wind speed, wind direction and water level will not be exact the ones used in the scenarios.

11.5 Predicting High Water Level

All information required to contribute to the coastal flood forecasting system are described above. It is therefore possible to comment on the comparison between the observed tide and the predicted tide. Since 17th December 2003 surge data has been made available from the UKMO. From that date till 17th April 2004 a comparison has been made between the predicted tides, as would be generated by the flood forecast system, and the observed tide. This comparison gives an indication about the accuracy of the system. This section describes the comparison.

The prediction consists of three parts:

1. The astronomical water level. The astronomical level is generated using the parameters found for the year 2002. See Chapter 9 for further details.

2. The UKMO surge prediction. The UKMO surge prediction is given for the UKMO point, located 20 km offshore, see Figure 11.3. The Shelf Seas Model of the UK Met Office calculates a surge level for this point. In this surge level the difference is given between the astronomical level and meteorological effects, such as storms.

3. The local surge from the UKMO point to the Dublin Port Tide Gauge. The local surge from the UKMO point to the Dublin Port Tide Gauge is calculated using the scenarios as calculated by FINEL2D in this chapter. Figure 11.26 shows the additional surge for different wind speeds, wind directions and water levels at the Dublin Port Tide Gauge. The observed wind at the M2 buoy, located in the Irish Sea was purchased for the considered period. This wind is used to generate the local surge between the UKMO point and the Dublin Port Tide Gauge. To generate a local surge prediction it is necessary to use the observed wind to interpolate between the wind speed, wind direction and the water level of all the scenarios, since the observed wind speed and direction will not be the exact ones used in the scenarios. As the water level at the Dublin Port Tide Gauge in this stage is not fully known, the astronomical level plus the UKMO surge prediction is taken as a first guess to take into account in the interpolation. As can be seen from Figure 11.26 the water level is not very sensitive for the local surge, so this approach is justified.

The predicted water level is the sum of the three parts set out above. In Figures 11.35 - 11.39 the predicted and the observed water level is shown for the 4 month period between December 2003 and April 2004. Also shown in the figures are the discrepancies at the high water level between Dublin Port and the surge prediction data obtained from the offshore UKMO node point. As can be seen from these figures the accuracy of the predicted high water levels is generally within 0.2m. The percentage of the high water levels which reaches this level of accuracy is 96%. All the high water predictions are within 0.35m accurate. All the predictions which are less accurate than 0.2m show an overestimation of the high water level. Only at March 19th the observed high water level is 0.35m higher than the prediction.

Several of the high waters are considered in more detail below:

- 26 December 2003: The second high water of 26 December 2003 has the highest observed water level in the period considered. The high water level reaches 4.75m LAT. The prediction of the high water level is very accurate with a discrepancy of only 0.05m.

- 8 February 2004: The prediction of the first high water of the day overestimates the high water level by +/- 0.35m. The next high water levels also show an overestimation of the high water level of between 0.1m- 0.3m. The second high water of 10 February shows an overestimation of 0.3m, while the surge level is 0m. If the surge level is calculated correctly this would indicate an error in the astronomical tide. This assumption is fed by the fact that all the high waters from 8 till 10 February 2004 show an overestimation of the high water level, which can't be explained by the differences of the surge prediction.

- 19 March 2004: The first high water of this day shows an underestimation of the high water level by 0.35m. The surge predicted at the UKMO point shows a surge level of 0.5m. In Figure 11.40 this tide is shown in more detail and it can be seen that seiches are occurring during this tide. The high water level coincides with the antinode of the seiche, therefore the high water level is raised, which in turn disturbs the prediction. This is the primary reason why the prediction deviates from the observation in this case.

- 21 March 2004: During this day a surge level of 0.5m occurred. The prediction of high water on this day overestimates the high water level with 0.1m, which is quite accurate.

Possible sources of differences are:

- Astronomical tide: The astronomical tide has deviations of +/-0.1m – 0.2m as is concluded from Chapter 9.
- UKMO surge: The accuracy of the UKMO calculations of mean sea level requires confirmation. Information obtained from them to date suggests that their estimation is lower than that used in the study. Further evidence on this is required from the UKMO.
- Local Surge: The wind speed during the four month period of the comparison was not very high and therefore probably not representative of local conditions for the period. The local surge is very small as is the calculated local surge - in the order of centimetres. This therefore is unlikely to be a major source of discrepancy.
- Another important phenomenon for the high water prediction, which must be accounted for, is seiches in Dublin Port. The observed water levels exhibit seiches of up to +/- 0.3m. However, as seiches are not predicted in the flood forecasting

system this can be a significant source of difference in the prediction, as is shown for the tide of 19th March 2004. It is therefore recommended that further study is undertaken to understand when and how they occur, and possibly to include them in the flood forecasting system in the future.

11.6 Conclusions and Recommendations

11.6.1 Conclusions

For the translation of the water level prediction at the nearest offshore warning point of the UKMO model to the location of the tide gauge in Dublin Port, a two-dimensional hydrodynamic model has been applied. At the same time, this model is used to investigate the relation between the water level at the tide gauge and several locations along the shores of Dublin Bay. For this purpose the FINEL2D model has been applied. The model has been successfully calibrated against the event of 1st February 2002. After the calibration process, the model was applied for a second period with well known observed tidal conditions. This validation of the model was done for the period 10th -11th March 2001 and showed also results which are considered acceptable for use in the prediction of overtopping, which is itself a key objective of this study.

After calibration and validation of the model, in total 64 scenarios have been calculated, which form the basis of the prediction of the water level of the early warning system in each of the 27 warning points. Each scenario represents different conditions of tide, wind and surge. The total number of 64 scenarios covers the full spectrum of extreme and daily conditions that can be expected in Dublin Bay.

During some calculations, resonance effects occurred in the model. The dimensions of Dublin Bay and the wave length of the resonance wave correspond well. In reality, these resonance waves are also observed from time to time. For instance on March 3rd, 2002, 2 days after the flood event. At this day the oscillations with a period of about one hour, reached maximum amplitude of several decimeters - see Chapter 9. In other words: the model results confirm that Dublin Bay is sensitive to the occurrence of water level oscillations that are related to the local geometry of the bay (so called seiches).

By comparing the predicted water levels as would be by the flood forecasting system with the observed water level at the Dublin Port Tide Gauge for a four month period a first estimation of the accuracy of the prediction system is found. The comparison showed that 96% of all the high water levels are accurate to within 0.2m. All high water levels are within 0.35m. A major reason for this deviation is caused by seiches, which disturb the prediction of high tide. They are not accounted for in the flood forecasting system. Also it is believed that the astronomical water level causes some deviations.

11.6.2 Recommendations

From the model results it appears in general that the deviations in water level along the coast of Dublin Bay, compared to the water level at Dublin Port Lighthouse, are limited. This makes the tidal modelling a useful tool to transfer predicted water levels for the offshore location to any place along the shores of Dublin Bay. In relation to other aspects that contribute to the final range of inaccuracy of water level predictions, further improvement of the model results is not recommended.

The accuracy of the surge prediction of the UKMO Shelf Seas model with respect to mean sea level should be investigated further. With this in mind Met Eireann will soon have available their own version of the Shelf Seas model. However, before results from this model are substituted for the UKMO results, it is recommended that a period of cross correlation is undertaken. This period should cover a period of at least one calendar year in order to ensure that the seasonal variations in predictions are captured. Further enhancement of the forecast system should be made on a phased basis against published results.

The occurrence of seiches in Dublin Bay can be observed frequently in the tidal records of Dublin Port. The use of the FINEL2D model indicates that the generation of seiches is sensitive to the geometry of Dublin Bay, however, from studies undertaken thus far, it is not clear what events (meteorological or others) are causing the seiche to develop on one particular day and not on others. The unpredictability of the seiche effect may therefore under predict the total tide level. Further investigations into the mechanics of the seiche are therefore recommended.

12 WAVE AND OVERTOPPING MODELLING

12.1 Introduction

The forecasting of coastal flooding resulting from the overtopping of the sea defences, relies on knowledge of the wave conditions in the nearshore region. This is achieved first through the use of the third generation SWAN (**S**imulating **WA**ves **N**earshore) wave transformation model; a two-dimensional fully spectral wave model used to obtain realistic estimates of wave parameters in coastal areas.

As waves propagate from offshore into the shallower coastal waters, their height and direction are modified by the influence of the seabed or bathymetry.

Then overtopping may be estimated either by reference to published literature, through the use of physical scale models or through the use of numerical models. Reference to published literature is often site specific, or by reference to generic defence types. In such cases, if the defence under consideration matches or falls within the range of validity of the literature, a satisfactory estimate may be made.

Physical models are expensive and therefore limited in the range of defence types considered. It is therefore necessary to parameterise the defence type in order to maximise the scope and applicability of the testing regime.

Numerical models are able to consider a wide variety of defence types and are often calibrated against the results of physical model tests. Their advantage lies in their speed of application, which in turn enables many different defence types to be considered. For the estimation of overtopping quantities in relation to this project, the numerical model AMAZON has been used.

AMAZON has been described briefly in Chapter 10 and figure 10.1 shows the model in operation. AMAZON is applicable to any beach or revetment profile, including vertical walls, however it is currently unable to account for re-curved walls. AMAZON can model wave propagation over complex and rapidly changing bathymetry in shallow water.

AMAZON requires significant computational power and time to run a representative sample size of random waves to achieve appropriate peak and average overtopping rates. Whilst it is capable of dealing with complex and vertical elements in a profile, this does have the effect of requiring a very fine grid which can result in a considerable increase in running time and as such it is important to use the model and set up transfer matrices to relate the potential nearshore wave climates to overtopping rates for use in the forecast system.

12.2 Methodology

An overview of the methodology that underpins the mathematical wave and overtopping modelling is outlined in the following steps:

- **Step 1.** Setup the model bathymetry using the supplied bathymetry.
- **Step 2.** Overlay grid mesh and boundary conditions from the UK Met Office Shelf Seas Model
- **Step 3.** Import known wave height information and re-create wave spectrum.

- **Step 4.** Validate model against a separate high tide event. This was done using information from the UKMO for the period 10th – 11th March 2001.
- Step 5. Use validated model to generate wave height, wave period and water level combinations.
- **Step 6.** Develop bed and structure profiles for the warning points.
- **Step 7.** Run overtopping model for wave and water level combinations. Develop overtopping matrices from the results.

The above steps are discussed in more detail in the sections that follow.

12.3 Wave Modelling

12.3.1 Model Geometry and bathymetry

The geographic extent of the model was chosen to be sufficient to include the area of interest, i.e. the study frontage between Portmarnock and ensure the full aspects of any changes to the wave climate occur within the model. This entailed establishing the northern boundary above the Rogerstown Inlet and the southern boundary below Dun Laoghaire harbour. The eastern, offshore boundary was chosen so as to include the location of the M2 wave rider buoy, although having validated the model, the eastern boundary was moved inshore (see Figure 12.1).

The bathymetry was compiled from a number of sources including:

- Admiralty chart, no. 1468, Irish sea;
- Admiralty chart, no. 1415, Dublin Bay area;
- Dublin Bay Project, Pipe line crossing survey;
- Clontarf Mudflats Survey;
- Dublin Bay Project, Bull Island Survey;
- Dublin Port, Channel & Berth Survey, December 2001;
- DEEP survey (2003), intertidal area of Sandymount and Merrion strand.

A uniform 100m x 100m grid was superimposed over the bathymetry. At each node the wave spectrum is calculated as the waves are propagated across the bathymetry. The model grid is shown in Figure 12.1.

12.3.2 Validation

The formal validation of the SWAN model as a wave transformation tool was undertaken by the Technical University of Delft. Details of the validations are available in Booij et. al., (1999), and Ris (1997).

However, before using SWAN for the present study, time series wave observations from the M2 buoy located in the Irish Sea at 53° 28.8’ N, 05° 25.5’ W were obtained and re-compiled as a wave spectrum. SWAN was used to transfer this M2 buoy data into two points where the UK Met Office has supplied data from their Shelf Seas model. Figure 12.1 shows the position of the M2 buoy in relation to Dublin Bay and the UK Met Office Shelf Seas Model nodes.

This spectrum was then used to generate wave conditions at two points (Pt 1 – 53.5N 5.75W & Pt 3 – 53.28N 5.75W) for which the UK Met Office has supplied data from their Shelf Seas model. These are shown as nodes 1 and 3 in Figure 12.1.

Validation against the UKMO Shelf Seas Model forecasts for points 1 and 3 are shown in Figures 12.2 and 12.3 respectively. At each location the M2 wave data is well reproduced. The shape of the UKMO data is of the same form as that of the M2 buoy, however, there is a phase shift of approximately 2 hours. This is only partially explained by the difference in longitude (approximately 20 minutes) between the M2 buoy and the UKMO output points. Further consideration by UKMO suggested that the phase shift is most likely due to an early arrival in the Met Office model of swell wave energy from the southwest approaches and into St. Georges Channel.

The UK Water Wave Model outputs (both sea and swell) have been compared with the M2 buoy data in Figure 12.4a (plot provided by UK Met Office) in order to assess whether the phase lag between the model and the observations is due to the influence of either:

i) the wind sea component – an error in the model forcing i.e. wind, could cause high wind sea values to be generated at the wrong time; or
ii) the swell sea component – the early arrival of high energy swell energy due to advection errors in the wave model.

The comparison was performed for points 1 and 3 as before. Both sets of results are consistent in that they show the main energy occurring in the wind sea part of the spectrum, indicating that the majority of the wave energy is locally generated. This therefore suggests that the phase lag is as a result of a slightly early arrival of strong winds forcing the model. The wind fields in the UK Water Wave Model are abstracted straight from the UKMO mesoscale atmospheric model, which in turn is nested in a global atmospheric model. The resulting phase lag is therefore probably occurring as a result of the early forecasting of low pressure systems with which the winds would be associated.

The M2 buoy data provided in Figure 12.4a is presented in only 6 hour blocks. A comparison figure with the M2 buoy data presented at hour intervals was also prepared and is presented in Figure 12.4b. From that figure the phase shift can still be seen and it is still noted to be between 1.5 and 2 hours. However, it should be noted that the wave forecast peaks whilst occurring a few hours in advance, are represented well in terms of elevation

As a result of the above analysis, it is concluded that the SWAN model can accurately represent the wave spectrum data and therefore the transformation of offshore waves to nearshore points.

In respect of the UK Met Office wave forecasts, it is recognised that this may peak up to a few hours in advance of the actual. At present the models proposed are the best available and are constantly being monitored and up dated. The UK Met Office has been informed of this observation and they are aware of it.

In terms of implications for the system, it is recommended that the predictions are monitored further and compared with those recorded by the M2 buoy, since the observation plotted in the figures relate to one isolated event for waves from a southerly direction (February 2002 event). It is worth investigating if this is also valid for other wave directions and whether less or more severe storms are better or less well represented. One of the main recommendations at the end of chapter 11 was the need to monitor the surge prediction for at least one year and this would seem appropriate in the case of wave forecasts also.

When a clearer picture of when and why the phase difference occurs, it may be possible to factor this into the forecast system to improve it as required, for example by using the wave forecast up to two hours head of the peak of the tide for the analysis. Finally it should also be note that waves conditions around the coastline within the project area are only likely to produce extreme overtopping sufficient to cause severe flooding when normal tide levels are sufficiently heightened by a surge event. Under such conditions it is more than likely that the region will be subject to a "Flood Watch" condition and during such times wave action should be considered in more detail within the system and also on the ground to improve the overall performance of the forecasts.

12.3.3 Wave Height, Wave Period and Water Level Combinations

The combination of wave height, wave direction, wave period and water level were selected to give a wide spread of conditions covering potential forecast values. The aim is to compile matrices from combinations of the above parameters such that the interpolation of forecast values takes place within a practical regime of resources. A total of 3,600 model runs were required to cover the combination of parameters shown in Table 12.1 below.

SWAN is used to simulate the key physical processes of waves transformed from the offshore boundary to selected inshore locations. Each SWAN simulation transforms a single user specified offshore wave condition to inshore locations situated every 500m along the study frontage (excluding the rivers), and at distances of 100m, 250m, 500m, 1000m and 1500m from the shoreline. This ensures that the wave model is able to simulate nearshore wave conditions at or near the warning points which were selected later in the project when a fuller assessment of the more vulnerable areas and defences with the lowest standard of protection had been assessed. Figure 12.5 shows the range of location points mentioned above from which output from the SWAN model was obtained. This data was used not just for the development of transfer matrices for selected warning point locations, but could also be used to provide nearshore design wave climate for the assessment of risk and the development of risk reduction options.

Table 12.1 - SWAN Modelling Scenarios

Significant Wave Height Hs (m)	Wave Period Ts (s)	Wave Direction	Wind Speed (m/s)	Water Level m ODM
0.1	4	30	10	1.0
1.0	6	60	20	1.5
2.0	8	90		2.0
3.0	10	120		2.5
3.5		150		3.0
4.0		180		3.5
4.5				
5.0				
5.5				
6.0				
6.5				
7.0				
7.5				
8.0				
8.5				
9.0				

Note: Waves with Hs ≥6.0m are only modelled for wave periods of 8s and 10s

The mean astronomical tidal water levels at Dublin are given in Table 12.2. The tidal regime is semi-diurnal i.e. the tide rises and falls twice daily. The mean spring tidal range is 3.4m. Hence the water levels listed above are chosen so as to cover the expected range of tidal conditions and extreme water levels expected along the study frontage as identified through the probabilistic analysis.

Table 12.2 - Tide Levels at Dublin

Tide	Water Level (m LAT)	Water Level (mODM)
Highest Astronomical Tide (HAT)	4.5	1.99
Mean High Water Springs (MHWS)	4.1	1.59
Mean High Water Neaps (MHWN)	3.4	0.89
Mean Sea Level (MSL)	2.4	-0.11
Mean Low Water Neaps (MLWN)	1.5	-1.01
Mean Low Water Springs MLWS)	0.7	-1.81
Lowest Astronomical Tide (LAT)	-0.1	-2.61

Note: Chart Datum at Dublin is 2.51m below Malin Head Datum

Figures 12.6 to 12.11 shows a range of wave vector plots, which illustrate the manner in which the wave direction and height changes as the waves move from deep water into the relatively shallow water of Dublin Bay. Each of the plots show the same wave conditions from a different offshore direction to illustrate how different directions are more important for given locations around the project area. For example figure 12.8 shows a 3m wave from 90 degrees propagating into the bay. Along the Sandymount frontage the waves are seen to be around 1 metre a short distance out from the coastline, while at the eastern tip of Bull Island the waves are noted to be around 0.5m.

In Figure 12.11, when the same wave propagates from a 180 degree direction, the waves along Sandymount are smaller around 0.5m and those at the eastern tip of Bull Island are around 1m. This is primarily as a result of the location and how much the wave has to refract (or bend) to reach the given location from offshore, i.e. waves at Sandymount are reduced more by refraction when the offshore direction is more southerly.

12.3.4 Boundary Conditions

Forecasts of offshore wave conditions are received from the UKMO twice daily, for a period of 36 hours in advance. A typical example of the data received is included in Table 12.3 below.

Table 12.3 - UK Met Office Forecasts

Surge Data for	:	Dublin Bay
Position	:	53.39N 5.75W
Time and Date of Model Run	:	0:00 28/09/2004
Units:		
Water Level	:	Metres wrt Mean Sea Level
Wave Height	:	Metres
Wave Period	:	Seconds
Direction	:	Degrees
Pressure	:	Pa

T+	Year	Month	Day	Hour	Total Water Level	Surge induced water level	Significant wave height	Wave zero crossing period	Wave principal direction	Mean sea level pressure
0	2004	9	28	0	1.08	-0.02	0.63	3.75	248.26	1019.53
1	2004	9	28	1	0.55	0.0	0.69	4.0	238.75	1018.81
2	2004	9	28	2	-0.22	0.02	0.72	4.0	224.48	1018.59
3	2004	9	28	3	-1.05	0.0	0.81	4.25	194.23	1018.59
4	2004	9	28	4	-1.56	-0.02	0.81	4.0	202.25	1018.59
5	2004	9	28	5	-1.64	-0.03	0.78	4.0	194.52	1017.89
6	2004	9	28	6	-1.25	-0.03	0.75	3.75	192.74	1017.53
7	2004	9	28	7	-0.61	-0.03	0.72	3.75	189.76	1017.20
8	2004	9	28	8	0.16	-0.02	0.72	3.75	221.51	1016.95
9	2004	9	28	9	0.83	0.0	0.75	3.75	236.98	1016.67
10	2004	9	28	10	1.22	0.0	0.72	3.75	247.75	1016.51
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.	.
33	2004	9	28	33	0.48	-0.05	0.41	3.5	28.48	1016.60
34	2004	9	28	34	1.06	-0.03	0.56	3.5	133.27	1016.46
35	2004	9	28	35	1.31	-0.03	0.63	3.75	141.52	1016.26
36	2004	9	28	36	1.23	-0.03	0.75	3.75	142.72	1016.18

12.4 Overtopping Modelling

Where waves or extreme water levels can impact on a flood defence the nearshore wave forecasts are used to predict wave overtopping at specific sites. Wave overtopping is determined by the use of matrices that take nearshore wave height, period and water level and return values of peak and mean overtopping (l/s/m). These rates are used to determine the appropriate flood warning response, and by the use of overtopping length and time to develop flooding volumes.

12.4.1 Warning Points

Based on knowledge of the defence around the project area, areas flooded during the February 2002 event and the initial assessment of flood risk through comparison of defence levels with extreme water levels, a number of locations around the project area were chosen which would be vulnerable to wave overtopping and which could be representative of adjacent stretches of coastline. In addition the locations were chosen to represent different defence types within a given area. For each of these locations a profile was developed through the defence and extended a short distance offshore to the nearest SWAN output point. Each of these profiles were then setup within the AMAZON model and overtopping calculations undertaken. These locations could then be used as warning points in respect of wave overtopping risk. These wave overtopping warning point locations are shown in Figure 12.12.

The overtopping model requires the input of bathymetry (or cross-section profile for 1-D calculation) and incident wave conditions. AMAZON is an unsteady state model, and as such, random waves can be imposed at the boundary. For each profile a wide combination of nearshore wave height, wave period and water level combinations were run using and the results used to create overtopping matrices.

In many cases, particularly along Sandymount Strand, the beach dries out to a significant distance seaward of the defences. In these cases each warning point is considered only for water levels where there is a risk that waves can lead to overtopping of the defences, thereby mitigating abortive run-time.

Where locations were chosen at a gap in the defences, for example along Sandymount Strand, two profiles were set up. The first one was set through the gap and the second one over the adjacent defences. Overtopping calculations were undertaken for both, thereby ensuring that a comparison could be made both now and for the forecasting system.

12.4.2 Boundary Conditions

Boundary conditions used to drive the overtopping models were supplied by reviewing the nearshore output results from the SWAN modelling work. In the operational system the boundary conditions will be supplied via the wave transformation matrices created from the SWAN wave modelling work. The UKMO supply forecasts of offshore wave conditions as described in Table 12.3 above. Each of these wave conditions is then applied to the transformation matrices in order to obtain a wave condition at the nearshore location relevant to the warning point profile. This is in turn interpreted against the overtopping matrices, to arrive at a mean and peak overtopping quantity for the particular input condition.

The only exception to the use of SWAN output to drive the boundary condition for the wave overtopping forecast will be for those warning point locations which are located well within the harbour, behind Bull Island or within the Baldoyle Estuary. These include warning points 8, 9, 10a, b, c, & d, 11, 19 and 20 as shown in Figure 12.12. At those locations, input wave heights would be mainly governed by forecasted wind speed and direction. For these locations wave heights have been estimated for varying wind speeds and directions over the allowable fetch distance and a matrix of wave height relating wind speed produced. An example of one of these matrices for warning point 9 is presented in Appendix L.

Profiles 9 and 10a & b, which are located along the Clontarf frontage and east of Clontarf baths, could also be subjected to wave energy penetrating the harbour from out in the bay. This will be particularly the case when water levels are high and waves are propagating from an easterly direction. Therefore when certain conditions such as easterly winds are forecast, the wave heights used will be taken from the SWAN output at locations just inside the harbour entrance.

12.4.3 Overtopping Quantities

At each warning point, the mean and peak overtopping quantities have been determined. The overtopping results have been recorded and matrices showing overtopping for different wave heights, periods and for different water levels prepared to allow prediction of overtopping quantities within the range of combinations considered.

Although the AMAZON overtopping model has been validated against measured overtopping in flume tests, further validation of the results has also been undertaken by comparison with the range of options tested in the Environment Agency sponsored research technical report W178 (1999).

An example of one of the these overtopping matrices for warning point 9 along the Clontarf frontage and a sketch showing the shape of the profile are presented in Appendix L.

13 HYDROLOGICAL ANALYSIS AND RIVER MODELLING

13.1 Introduction

13.1.1 General

Section 10 presented a brief overview of the models used within the DCFPP and their relevance and use in developing a coastal flood forecasting system. This section describes in some detail the development of the river models. It describes the investigation work undertaken, presents the results of the modelling work and demonstrates how those results were used to develop the relationships between tidal and fluvial components with the intertidal reaches of the rivers. It presents the results as a simple spreadsheet which forms one of the seminal elements of the tidal forecasting system. The model used to investigate the intertidal reaches of the river was the 1 dimensional river model ZWENDL.

13.1.2 Methodology

An overview of the methodology of the mathematical river modelling is presented in Scheme 1-1 below. Each of the different actions identified in this figure is discussed in more detail in the following sections.

Construction of the ZWENDL model schematisation:
- River and Harbour sections
- Special constructions like weirs, bridges, sluices
- Other model parameters

↓

Calibration of the ZWENDL model with the
1st February 2002 Storm Event

↓

ZWENDL modelling of:
- Complete astronomical spring - neap tidal cycle (without wind)
- Complete spring - neap tidal cycles for moderate and extreme conditions (discharge, wind and surge)

↓

Develop Relationship between Dublin Lighthouse
reference location
and warning locations for all scenarios

↓

Develop Astronomical tide relationship for each
warning point and Surge level and discharge
matrix

Flowchart 13.1 - Schematic overview of mathematical river modelling methodology

13.1.3 ZWENDL

The 1-dimensional river model ZWENDL, has been used to model the tidal and fluvial conditions in the Rivers Liffey, Dodder and Tolka and also in the Royal Canal and Grand Canal Basin. This model has been used extensively for one-dimensional tidal calculations in rivers and estuaries and was developed for this purpose by the Department of Public Works in the Netherlands.

With respect to model capabilities and functions the ZWENDL model,

- is capable of calculating water levels, discharges, currents, temperatures, salt concentrations (coupled with hydraulics by the density term) and concentrations of dissolved chemical substances in intertidal area's and rivers.
- can set various boundary conditions including water levels, discharges, wind (velocity and direction), temperature and salt (and/or other substances) concentrations as time series or harmonic components.
- has a special routine for the schematisation of hydraulic structures with different compartments (sluices, gates, bridges, etc.). Therefore, the geometry of the different arches of the bridges in the river Liffey, Dodder and Tolka can be easily included in the model schematisation.
- can trigger sluices by a variety of parameters like concentrations, pressures (/ differences), water levels (/ differences), discharges etc. The sill level, lift height of doors or sluice width can be varied depending on the trigger boundary conditions.

The ZWENDL model has previously been applied to,

- the design and provide support during the construction phase and to develop closure strategies for the two biggest Dutch storm surge barriers: Eastern Scheldt Barrier [lit. 1.1] and Maeslant Barrier [lit. 1.2],
- an environmental impact assessment regarding the changing role of the Haringvliet sluices from discharge sluices to a storm surge barrier [lit. 1.3, 1.4],
- investigate potential flood risk reduction options and improvements to water quality in the Delta area of Holland by modelling existing and proposed new sluices in this area [lit.1.5],
- investigate potential flood risk reduction options and improvements to water quality by constructing new connections between the Eastern and the Western Scheldt [lit. 1.6. 1.7, 1.8],
- provide daily forecasts of the water levels in the Eastern and the Western Scheldt for ship navigation [lit.1.9],
- calculate the design flood levels in all Dutch main rivers (Rhine, Meuse) [lit.1.10].

13.1.4 Calibration

The ZWENDL 1D model was calibrated by reproducing the February 2002 flood event. The hydrodynamic behaviour of the rivers and harbour was simulated in the ZWENDL model, including effects of wind shear and barometric pressure effects.

Within the reaches over which the models were constructed, i.e. the tidal reaches, there are no flow or tidal gauges with which to undertake further calibration or validation runs and as such only observed data available for the 1st February 2002 could be used.

13.1.5 Simulations

The calibrated model was used to determine the combined effects of wind shear forces, astronomical tides and river discharges. For that purpose a number of events were simulated, each of them giving a unique combination of wind (direction and force), tidal conditions and river discharges.

The effect of the wind was divided between local surge caused by local wind and surge created from outside the model area:

- The local surge caused by local wind was modelled by applying wind (direction and force) in the ZWENDL model. Cases were selected in accordance with the FINEL modelling work. Chosen wind speeds are 0 (for reference), 15 and 30 m/s.
- The surge created from outside the model area has been covered by the FINEL modelling work. For the ZWENDL calculations a set of surge cases has been selected, based on the probabilistic analysis of the water level recordings at Dublin Port (**Chapter 9**).

The river discharges were chosen based on available hydrographic studies.
For each combination of surge and discharge a complete (astronomical) spring – neap tidal cycle was modelled in ZWENDL. In this way for each high water level, the combined effects of wind surge and discharge have been determined by the ZWENDL model for all the warning points.

13.2 Model construction

13.2.1 General

For the inland sections of the rivers and canals, the results of the Dublin Bay 2-dimensional FINEL model were combined with the results of the 1-dimensional river/canal ZWENDL model to determine the complete range of transfer matrices, exceeding trigger levels and design water levels for flood risk assessment and development of options.

To build the river model schematisation the package MatLab has been used. MatLab-routines are available:

- to convert geometrical data into the geometrical part (sections) of the model schematisation,
- to check the schematisation for consistency,
- to generate hydraulic boundary conditions and
- to plot the model schematisation and the results of the calculations.

An important aspect of building the river models was setting the down stream boundary of the models. If no relationship between the rainfall and the water levels at Dublin Port is found to exist then the seaward boundary of the river model could be set at the Dublin Port tidal gauge, at the entrance to Alexandra Basin West.

However if a relationship was found to exist either at the Port entrance, or as far out to be free of river effects, then this would be a better seaward boundary in the model.

Such a relationship was investigated as part of the probabilistic analysis of the tidal records described in **Chapter 9**. From that work no relationship was found between the water levels at Dublin Lighthouse and the river Liffey discharge, when considering data for the year 2002. However, experience tells us that especially at high discharges (with return periods greater than 1/10 year) there will be some element of fresh water influence on the water levels because of density effects. Therefore, the decision was made to place the seaward boundary of the river model at the entrance of Dublin Port, at the end of the training walls: the North Bull Wall and Great South Wall (see figure 13.1, Appendix M).

It was considered that the model could be split in two parts:

- the Liffey and Dodder rivers with part of the harbour area and
- the Tolka river.

However, this split would have caused a problem in respect of the location of the seaward boundary of the Tolka river model and its relationship with the water levels at Dublin Lighthouse. This relationship is important because it is the only location within the study area where there is recorded water level information. The basis of all the modelling and hence warning points predictions requires every water level (river warning point) to be related to the water level at Dublin lighthouse. If the Tolka had been schematised as one model with a seaward boundary just downstream of the business park road bridge, this relation would have been more difficult to establish and it would have been difficult to generate level boundary conditions. Therefore it was decided to schematise all the rivers and the harbour area in one model with one seaward boundary.

In the 1D ZWENDL model, the river geometry has to be schematised in sections. The choice of the section boundaries in the rivers is connected with the choice of the location of the warning points: the model calculates the water levels on the section boundaries. Hydraulically it is preferable to choose the section boundaries at discontinuities in the river course, like sudden changes of cross sections, bends, weirs and bridges.

In this model the warning points should be chosen in the close proximity of the bridges. Therefore, warning points do not form constraints on the layout of the sections: they are "naturally" present in the schematisation. In this model, sections start and end at bridges. When a warning point is required between the bridges, sections have to be split in two parts: from a bridge to the warning point, and from the warning point to the next bridge. So warning points between bridges would have had influence on the schematisation of the river in sections.

13.2.2 Geometrical data

Dublin Port

The bathymetry of the port area used in the depth schematisation has been derived from the FINEL model depth schematisation and originates from the following sources:

- Admiralty chart, no. 1468, Irish sea;
- Admiralty chart, no. 1415, Dublin Bay area;
- Dublin Bay Project, Pipe line crossing survey;
- Clontarf Mudflats Survey;

- Dublin Bay Project, Bull Island Survey;
- Dublin Port, Channel and Berth depths, Medway Survey December 2001;
- DEEP survey (2003), intertidal area of Sandymount and Merrion strand.

See figure 13.2, Appendix M for the bathymetry used in the ZWENDL1D model.

River Tolka

The bathymetry was taken from the River Tolka survey [lit 2.1] undertaken as part of the River Tolka Flood Study. This survey data was converted into x, y, z depth data, suitable for use in MatLab conversion programs to generate a section model. All model section boundaries were positioned at the location of the cross sections as given in that survey.

The long profiles, cross sections and bridges details from the survey were used to schematise details of the weirs and bridges and all other general information.
Details of the proposed new Dublin Port Tunnel bridge and the surrounding area were made available in the drawings from the tender documents [lit 2.2].

River Liffey

The bathymetry of the river Liffey and the geometry of the bridges, as far upstream as Heuston Station, were obtained from a recent survey [lit 2.3] of the river undertaken by the Dublin Docklands Development Authority (DDDA). Details of that survey were provided in ACAD drawings format. The bathymetry details were converted to x, y, z depth data for use in the MatLab conversion programs and the geometry of the bridges was measured from the relevant cross section drawings.

Other bridge details were also made available (digital and hard copy) from DCC, Road Design Division.

Quay levels along the river Liffey from Matt Talbot Bridge to East Link Bridge were provided by the DDDA from a recent survey of the campshires in that area [lit 2.4].

As part of this project, a topographic and bathymetric survey (the DEEP survey, [lit 2.5]) was undertaken to obtain additional information across the project area.. Additional street and quay levels and details of the bathymetry upstream of the Frank Sherwin Bridge to the Islandbridge weir were obtained from that survey.

River Dodder

The bathymetry of the river Dodder and the geometry of the bridges were mostly obtained from the DEEP survey [lit 2.5]. However, some additional bridge details were also made available through other sources, mainly hard copy drawings from DCC, Road Design Division.

Royal Canal and Grand Canal

The bathymetry for the Royal Canal and the geometry of the bridges along the canal was obtained from the DCFPP survey. Bathymetric details of the Grand Canal were obtained through discussions with Waterways Ireland, and quay details obtained from the DCFPP survey.

13.2.3 Model schematisation

Seaward Boundary

As explained in paragraph 2.1, the seaward boundary of the ZWENDL model schematisation has been chosen between the port training walls (North Bull Wall and Great South Wall). At this location hydraulic boundary conditions (i.e. water levels) are fed into the ZWENDL model from the results of the FINEL tidal model The northern boundary is located between the Bull Island at Dollymount and the mainland (see figure 13.1 & 13.3). Table 13.0 in Appendix M shows the important section nodes within the model as presented in Figure 13.3.

Careful consideration was given to the height of the North Bull Wall. The outer part of this training wall is submerged on each high tide. The level at the outermost end of the North Bull Training Wall was determined to be circa –1.11 m Malin Head Datum (ODM). The height of the training wall gradually rises moving towards the shoreward end and approximately 1km from the outer end the height is +0.89 m ODM. Details of levels along the Bull Wall were obtained from a number of sources including:- a port aerial survey of the Clontarf mudflats; literature obtained from Dublin Port on the history of the port and Admiralty charts. This information was augmented by observations of the time the tide level crossing the outer end of the training wall compared to Dublin Port tide gauge recordings. To improve accuracy the varying crest levels along the North Bull Wall were schematised in the ZWENDL model as a series of weirs with different crest levels instead of simply just one weir having an averaged weir level.

The schematisation of the Port area consists of a Northern and Southern branch, with the main entrance channels from the sea to the Tolka and the Liffey, interconnected by some short cuts. An additional blind arm to the north represented the channel running from inside the port under Bull Bridge behind the Bull Island. In the main mudflat area’s the sections are very wide and shallow and act more like storage sections rather than (conveying) stream sections. The ZWENDL model algorithm is capable of dealing with both circumstances in the same section, by defining a storage and a flow area for that section.

River Tolka

The upstream boundary has been situated near the Botanic Garden Bifurcation, this is a considerable distance upstream of the project limit set for the Tolka within the DCFPP, which was at Annesley Bridge. However this upstream boundary has been chosen for ease of applying upstream boundary conditions: a hydrographic station exists at Botanic Gardens and as such boundary conditions can be easily applied to the model at this location. In addition it was not clear to what extent the tide might be able to penetrate upstream during combinations of high tide and high river discharges. Since sufficient river details were available through the recently completed River Tolka Study, it was considered prudent to extend the model to a point that we could be sure was beyond any tidal influence. Note that according to the recently completed flood study [lit. 2.14], the Tolka is tidal until Drumcondra; which is about 1 km downstream of Botanic Garden.

Whilst the river model has been extended upstream of Annesley Bridge for ease of applying upstream boundary conditions, the model has only been used to assess flood risk downstream of Annesley Bridge, since flood risk upstream of this point has been assessed in some detail as part of the River Tolka Flood Study.

The sections of the River Tolka, from Botanic Gardens to the port have steep slopes and eleven weirs are located in this part of the river. The ZWENDL model package is sensitive to drying of shallow parts of the model sections, and careful attention has been taken to prevent instabilities and to provide a smooth flow, even at very low river discharges.

All section boundaries were positioned at the cross sections taken for the River Tolka survey [lit 2.1].

Rivers Liffey and Dodder

Both rivers over the study extent have very limited slopes. The length of the rivers which have been schematised, consist of the lower reaches, which are subject to considerable tidal influence.

The upstream boundary of the river Liffey is situated at the Islandbridge weir. From site visits and other project literature, this weir is known to be the definitive tidal limit, even at combinations of high tide and high river discharges.

The upstream boundary of the River Dodder was set at the Balls Bridge weir in the DCFPP brief. However this is not the optimal location since tidal effects on a very extreme tide can extent beyond this weir. However due to lack of additional useful survey data upstream of Ballsbridge, the upstream limit was set at the Ballsbridge weir.

For this reason the DCFPP survey was limited to the Balls Bridge weir. Some survey information in the form of cross section data was made available from the ESBI Inundation Study [lit. 2.12] until the Bohernabreena reservoir. However, these cross sections were not considered sufficiently accurate in comparison to the DCFPP survey in that:

- Cross sections from the ESBI study are approximate and base data for quality assurance checks no longer exists.

- Cross sections were located at very large intervals (500 m). E.g. the distance between London bridge and Balls bridge includes 15 cross sections in the DEEP study and 3 cross sections in the ESBI study.

Having a discharge boundary section within the region of tidal influence introduces an inaccuracy in that the discharges themselves are influenced by the tide. However it is considered that extending this hydraulically complicated part of the Dodder with the rough ESBI data could cause a larger inaccuracy then accepting the inaccuracy because of the tidal discontinuity at Ballsbridge weir.

It is recommended for this reason and also because of the nature of the extreme fluvial flood risk known to exist and as presented in section 15, that a more extensive model study of the River Dodder, similar to that completed for the Tolka, be undertaken to consider the issue of fluvial flood risk on a more detailed and catchment wide basis.

Royal Canal and Grand Canal

Both the Royal and Grand Canals are included in the model schematisation and act mainly as storage sections. The upstream boundaries are chosen at the lock gates just upstream North Strand Road on the Royal Canal and at the lock gates adjacent to Grand Canal Street on the Grand Canal.

The sluices that used to exist on the downstream end of the Royal Canal no longer exist having been removed in 1993. Currently proposals are being developed to restore the navigational lock at the Liffey entrance to the canal land and to construct a pair of flood gates.

The sluices at the entrance to the Grand Canal basin are still functioning. Their main function is to keep water levels high in the Grand Canal basin and they are normally kept closed other than when canal traffic exits and enters the basin. The Grand Canal and the Grand Canal drainage scheme discharges into the basin and water regularly flows over the crest of the downstream gates (which is at a level of 3.39 m ODM) and into the sea. The gates may be forced open under hydrostatic pressure when the outside (sea) water level exceeds 3.39 m ODM. However this would require a very extreme event (greater than a 1 in 500 years event under present conditions) to occur.

This triggering behaviour can be simulated in the model.

13.2.4 Hydrographic study and boundary conditions

General

Due to the existence of the dams and reservoirs on the rivers Liffey and Dodder, and therefore the ability to regulate at least in part the flows, extreme floods are reduced to a certain extend. However for calculations of extreme water levels along the rivers within Dublin, a combined statistical approach is important in which the risk of high sea water levels and the risk of high river discharges are both considered.

To set upstream boundary conditions for the ZWENDL river model, it is important to know the river discharges. A number of reports and data sets were available for the three rivers and these have been reviewed and information extracted to determine the discharges boundary conditions for the rivers, with return periods of more then once a year.

Hydraulic conditions River Liffey

The River Liffey is the largest river to enter Dublin. The catchment area (1370 km2) is divided in three parts:

- The upper catchment area (308 km2) is very mountainous and responds quickly to heavy rainfall. Pollaphuca dam exists at the end of the upper catchment area with Golden Falls dam, situated a further 2 km downstream. The inflow to the golden Falls reservoir is equal to the outflow of the Pollaphuca reservoir. In particular the Pollaphuca reservoir acts as a flood relief reservoir subject to ESB operating guideline restrictions intended to avoid overtopping.

- The middle catchment area (534 km2) is characterised by a rather flat landscape with Leixlip dam at the downstream end.
- The lower catchment area (528 km2) is flat and discharges through Dublin into Dublin Bay and the Irish Sea. There are four important tributaries between the Leixlip Dam and the Irish Sea (over a distance of 20 km): Rye water (215 km2), Griffeen (50 km2), Cammock (84 km2) and Dodder (113 km2). The Dodder enters the Liffey just upstream of the East Link Toll Bridge and Dublin Port and as such has little influence on the flows through the city.

There are three dams on the river Liffey: Pollaphuca, Golden Falls and Leixlip. These three dams are used for water supply, power generation and protection against floods.

Since the last dam is situated at Leixlip, 528 km2 (ca. 40%) of the Liffey total catchment area can not be regulated by the dams.

Form the literature and reports reviewed, in respect of hydrological data for the river, the report: "River Liffey Weir - Concept study" [lit 2.6] is the most important source of information for determining extreme river discharges. In this report, data for a number of extreme floods events (flood of June 1993, flood of November 2000 and the flood of December 1954) were used to determine appropriate return periods flood flows for the river. These are presented below [Table 13.1]:

Table 13.1 - Return periods discharge river Liffey [lit. 2.6]

Return period Date of flood discharge	1/20 Nov 2000 [m^3/s]	1/50 Jun 1993 [m^3/s]	1/100 Dec 1954 [m^3/s]	1/1000 Design flood [m^3/s]
Lower	210	260	300	360 - 390
Mean	236	285	325	390 - 420
Upper	**240**	**310**	**350**	**420** - 455

Those figure highlighted in bold are the discharges that have been used in the ZWENDL river calculations.

Other relevant reports included:

1. "River Liffey Flood of June 1993" [lit 2.7].
   This report deals with the June 1993 flood for the upper and middle catchment area (until Leixlip). The results are used in the "River Liffey Weir" study.
2. "River Liffey Flood of November 2000" [lit 2.8].
   This report deals with the flood of October 2001 for the upper and middle catchment area. The results are used in the "River Liffey Weir" study.
3. “Inflows into the tidal reaches of the river Liffey” [lit 2.9].
   This report notes that the Liffey is tidal until Islandbridge weir. The report mainly provides a summary of the hydromatic stations in the river Liffey and some data on the tributaries of the Liffey (Griffeen and Cammock river, which are included in the “River Liffey Weir study" [lit 2.6]) and the Dodder.
   The report indicates that two flow gauges existed on the River Liffey in the vicinity of Islandbridge Weir, one upstream and one downstream, and that the DCMNR were responsible for them. However, when approached for information on these gauges, the DCMNR indicated that they were no longer in use. Given that the

River Liffey is the main river flowing through the heart of Dublin City and the strategic importance of the Islandbridge Weir, it is strongly recommended that either these stations are brought back into use or new gauge stations installed at this location.

Hydraulic conditions River Dodder

The Dodder is the smallest (catchment area) river of the three (Tolka, Liffey and Dodder) entering Dublin city, it is however the second one in terms of discharge. The Dodder has a long history of flooding, more than any other river in Dublin. The total catchment area is 113 km2 with a steep mountainous (1:20) and fast reacting upper and middle catchment area and a flat lower (Dublin) catchment area. In the upper area, there are two reservoirs (Upper and Lower Bohernabreena Reservoir) but they collect runoff water from only 28 km2 (25 %) of the total catchment area. Some important tributaries like the Owendoher and Little Dargle are contributing downstream of the dams. Therefore extreme flood discharges on the river can only be regulated to a limited extend.

A number of reports with information on hydraulic conditions for the river Dodder were available and have been reviewed as part of this project to determine hydraulic boundary conditions for the upstream end of the ZWENDL model. These reports include:

1. “Presentation of analysis carried out by drainage design division of Dublin County Council on the Dodder river” [lit 2.10]. This report deals with the flooding 25th/26th August 1986. Two approaches were used to calculate the return periods: the unit hydrograph and the statistical approach. The hydrograph method (with generalised FSR factors) does not give very reliable results. The statistical approach with Extreme Value distributions EV1 and EV2, the latter with 2 different shape factors (ref. report 2.), gives better results and looks more reliable.
2. “Dodder river, flood study” [lit 2.11] . This report (similar to that of 1 above) deals with the sever storm of the 25th/26th august 1986, known as “Hurricane Charlie” which caused considerable damage. This report forms an extension on the first report [1.]. Both reports present the same return periods (see following table). A number of different models were evaluated to determine the rainfall – discharge relationship: Rational approach, Generalised Flood Studies Report approach (FSR) and Regional Curve analysis. The conclusion of these models was that they were giving unrealistic (high) return periods for the rainstorm of 1986.
   A statistical analysis (Extreme Value distributions EV1 and EV2, the latter with 2 different shape factors) using annual maximum discharges (at Orwell weir) was undertaken to determine the associated return periods. Of these distributions, the Extreme Value Type 2 (with shape factor k=-0.05) distribution was chosen as the most appropriate and the results are summarised in the following table.
3. “River Dodder Inundation Study” [lit 2.12]. This study assessed the effects of a breach at the upper and lower Bohernabreena dams and estimates the resulting peak water levels and discharges downstream until its outflow in the River Liffey. Since the peak flow associated with a dam breach is excessive, excess of 880 m3/s, the flooding implied is severe. The breach of the lower dam (880 m3/s) is giving a discharge of 340 m3/s at Orwell bridge, which compares to the natural flood event flow in August 1986 of 250 m3/s at this location. The rise of the water level at Balls bridge will be about 3 – 5 m above non flooding conditions in this case.

4. "Hydrological data: Annual maximum peak outflows from Lower Bohernabreena Reservoir" [lit 2.13]. A statistical analysis is presented in this report based on annual peak flows from 1949 – 2001 for the Lower Bohernabreena Reservoir. Note: This reservoir is at a considerable distance upstream of Dublin. The extreme floods of 1905, 1891, 1986 and 1931 were fitted with Extreme Value distribution EV1 and a General Extreme Value distribution (GEV). From the resulting plot, the extreme floods fit well, with lowest standard errors for distribution EV1. The results of these calculations are presented in the next table for comparison. Note that these results could not be used for this study due to the distance of the reservoir from Dublin, as much inflow will occur over the lower catchment area.
5. Data (maximum annual discharges) are available from Waldrons bridge (Orwell weir) from 1987 – 2001 (with 5 missing years). Since reports 1, 2 and 4 above are dealing with data untill 1986, this data would be useful for future evaluation, especially since there are 3 important floods in these 11 annual discharges: 156 m3/s (2000), 110 m3/s (1994) and 86 m3/s (1998). According to the EV2 distribution of report 1 and 2 the return periods of these floods are 1/50, 1/10 and 1/5 year respectively.

The results are summarised in the following table [Table 13.2]:

Table 13.2 - Return periods discharge river Dodder [lit 2.13]

Return period	1/10	1/50	1/100	1/200	1/1000
discharge	[m^3/s]	[m^3/s]	[m^3/s]	[m^3/s]	[m^3/s]
Report1 and 2	125	189	218	248	322
Report 4: EV1	57.9	81.9	92.0	102.1	113
Report 4: GEV	57.6	88.3	103.2	119.3	148
Chosen	**125**	**190**	**220**	**250**	**320**

Hydraulic conditions River Tolka

For the River Tolka there are not as many reports available with regard to flooding as is the case for the other rivers. However, recently a major flood study for the Tolka has been completed [lit. 2.14]. Data from that study, such as the geographical and hydraulic description of the Tolka, have been used in this report.

The River Tolka is the second largest river to enter Dublin in terms of catchment area. It is however the smallest one in terms of discharge. The river Tolka has a catchment area of 141 km2. In the upper-catchment, the river is just a stream with small meanders and low banks with a relatively flat bed gradient of about 0.4%. The river is 2.5 m to 5 m wide. Occasional flooding causes a flood plain extending up to 400 meters wide.

Entering urban environments, the profile of the river changes noticeably. Through the formalised Tolka Valley Park, Botanic Gardens and Griffith Park, it becomes somewhat wider and straighter, with generally higher and more defined grass banks. In its latter reaches through Glasnevin, Drumcondra and Marino, the river becomes increasingly canalised. In this section, the riverbank varies from natural riverbank to an ad-hoc arrangement of walls of varying height and robustness. Downstream of Drumcondra the river is also subject to tidal influence and the channel is wider with more formal riverside walls in the lower section.

The River Tolka has a history of extreme floods with relatively low frequency going back over 100 years. As a result, it has been the subject of a number of investigations, the most significant of which (prior to the River Tolka Study) were carried out following significant floods in 1954 and 1986. In both of those studies, the propensity of the River Tolka to severe occasional flooding was identified and specific remedial measures were recommended.

The River Tolka experienced significant flooding in November 2002 (the flow at the outlet estimated to be about 100 m3/s) which resulted in extensive damage in the upper and middle catchments. This latest flood did emphasize the importance of that flood study for the Tolka [lit. 2.14].

It is interesting to note that the flood of 26th August 1986 (Hurricane Charlie) which caused major flooding of the Rivers Dodder, Dargle and other South Dublin rivers, did not produce an equivalent extreme flood in the Tolka. It seems that the catchment area's of the Tolka and the Dodder (and Liffey) are different in terms of meteorology and hydrology.

Presented below are the results of the statistical analysis of the flood study for the Tolka [lit. 2.14]. This gives the following design flows at Drumcondra just upstream of Drumcondra Bridge []:

Table 13.3 - Return periods discharge river Tolka [lit. 2.14]

Return period [year]	1/25	1/50	1/100
Discharge	65 [m^3/s]	75 [m^3/s]	90 [m^3/s]

Discharge boundary conditions for the statistical ZWENDL simulations

The preceding paragraphs (13.4.2 – 13.4.4) discuss a number of reports, which present statistical analysis of discharges for the three rivers considered within the DCFPP.

The most appropriate statistical return period discharges, as determined in these reports, have been inter/extrapolated where necessary to provide specific return periods for this study and these are summarised in the following table [**Table 13.4**]. These values will be used to set the discharge boundary conditions for the three rivers in respect of calculations undertaken within the ZWENDL model. The canals are dealt with as storage areas of the model.

In addition to the discharges with return periods of once every 10 years and longer, the flows for the once a year and the yearly average discharges (or base flow discharges) have been estimated [**Table 13.5**]:

- by extrapolation of the statistical results from the reports (paragraphs 13.4.2 – 13.4.4);

- by calculating the average and the standard deviations using the daily discharges from the rivers as far as available.

Note that the discharges for return periods of once every 10 years or longer are accurate and based on statistical analysis; however the yearly low, average and high discharge and the once a year discharge are based on estimation and are less accurate.

When comparing the three rivers, it is interesting to note the influence of the river Dodder in respect of total discharge from the three rivers into the bay. For example the Dodder discharge increases significantly for higher return period events in comparison to the Liffey and Tolka, in terms of percentage (25% versus 58% at once a year discharge to 36% versus 48% at 1/1000 year discharge). This is probably caused by the geographic difference of the catchment area; the Dodder area is more steep then the Liffey area. Another reason might be (the operation of) the storage reservoirs, which might be more effective for the Liffey than for the Dodder.

Table 13.4 - Discharge boundary conditions statistical simulations with return periods longer then once a year

Return period discharge	Tolka		Liffey		Dodder		Total
	[m^3/s]	[%]	[m^3/s]	[%]	[m^3/s]	[%]	[m^3/s]
1/1 year	20	16	70	58	30	25	120
1/10 year	55	15	195	52	125	33	375
1/50 year	75	14	310	52	190	34	550
1/100 year	90	14	350	51	220	35	635
1/500 year	120	15	400	49	290	36	810
1/1000 year	140	16	420	48	320	36	880

Table 13.5 - Discharge boundary conditions statistical simulations for yearly discharges

Yearly discharge	Tolka		Liffey		Dodder		Total
	[m^3/s]	[%]	[m^3/s]	[%]	[m^3/s]	[%]	[m^3/s]
Low	1	20	3	60	1	20	5
Average	2	17	7	58	3	25	12
High	10	17	35	58	15	25	60

Boundary conditions calibration period February 2002

In order to determine the discharges before and during the February 2002 flood, the daily mean discharges for the three rivers have been used.

Measurements were available at the following hydrographic stations:

1. Tolka: Botanics Garden
2. Liffey: Leixlip Dam, Griffeen and Ryewater
3. Dodder : Waldron Bridge

It should be noted that there are no river gauging station at a convenient downstream location on the River Liffey, in particular in the vicinity of Island Bridge Weir. Therefore no definitive daily flow value for the 1st February 2002 through the centre of Dublin was available. However, a reasonable estimate from those stations available has been made and details are presented below.

In the following table the measurements at the hydrographic stations are summarised:

Table 13.6 - River discharges February 2002 event

	Tolka	Liffey			Dodder
River / Station	Botanics Garden	Leixlip Dam	Griffeen	Ryewater	Waldron Bridge
Date	Discharge [m^3/s]	Discharge [m^3/s]	Discharge [m^3/s]	Discharge [m^3/s]	Discharge [m^3/s]
28-1-2002	3.23	11.45	0.61	2.16	2.46
29-1-2002	3.67	12.75	0.63	1.95	3.12
30-1-2002	3.29	11.75	0.64	2.34	3.15
31-1-2002	3.06	22.05	0.70	2.59	2.76
01-2-2002	4.54	24.15	0.71	7.50	5.32
Used flow	**5**	**30**			**6**

The discharges shown in bold have been used in the ZWENDL river calibration calculations. As can be seen in the table discharges are chosen with the emphasis on the last day. In order to avoid drying or instability problems in the model simulations the design discharges have been applied throughout the period from January 28th to February 1st.

13.3 Calibration of the ZWENDL model

13.3.1 Introduction

Following construction of the model and after determining the boundary conditions, the model is ready to run. However before the model can be used as an effective tool it has to be calibrated and validated.

Calibration is the process of tuning the parameters of the model in such a way that the model performs as reliably as possible. This is usually undertaken by comparing the model results to observations in the field. Since the purpose of this particular Dublin river model is giving warnings for flooding, it has to be calibrated with emphasis on high water levels. These high water levels can be caused by:

- high tide levels on the seaward boundary or
- high discharges on the river boundaries or
- a combination of both.

The February 1st 2002 event is an excellent example of the first category as during that event the tide was the highest on record while the river discharges were very low. Since it is recommended to calibrate river models with high discharges, this event is not the best one for calibration purposes. However since reliable field observations were available only for this event and because of the importance of this event, the ZWENDL model has been calibrated on the flood of February 1st 2002

Validation is the process of checking the model settings found through the calibration against a period other than the calibration period. At the time of the calibration of the model, no reliable water level data with matching discharges for situations with high discharges were available. Therefore it was not possible to validate the model successfully. However every effort has been made to ensure that the model does reproduce a sensible backwater curve for the higher fluvial discharges. Nevertheless it is highly recommended that new water level and flow measurement gauges be installed at suitable locations at and within the tidal reaches of both the rivers Liffey and Dodder for future use in model validation. It is also recommended that models of both the Liffey and Dodder are extended upstream and further investigation into the fluvial flood risk be undertaken.

13.3.2 Calibration February 1st 2002 period

General

Because of the nature of the flooding observations against which the model will be calibrated (i.e. water levels only), the ZWENDL model can only be calibrated with emphasis on high water levels.

The ZWENDL model has been calibrated using the following data:

- Seaward area:
    - ♦ The observed water level at Dublin Lighthouse.
    - ♦ The calculated (FINEL2D) water level at warning point 16: Entrance Tolka. Of course this is not as accurate as a real measurement but it gives a good indication.

- River area:
    - ♦ Observed water levels during the 1st February 2002 flood taken from several reports [lit 3.1 – 3.3]. According to these reports and a map showing flooding locations [lit 3.2], flooding occurred at:
      1) Dodder: Greyhound Race Track, Newbridge Ave and Stella Gardens
      2) Liffey: Halfpenny bridge (to within 100mm),Victoria and Wolfe Tone Quays
      3) No flooding occurred along the river Tolka
      4) Royal Canal – quay walls
      The calculated water levels from the model in these areas have been compared with the quay levels.
    - ♦ The observed water level at the Rory O'More bridge (3.12m ODM) on the River Liffey which was marked and later measured by DCC staff.
    - ♦ The observed water level at the Landsdown Road bridge (3.25m ODM) on the River Dodder which was marked and later measured by DCC staff.

Note: The accuracy of the observed water levels on the quays is less then the recorded water level at Dublin Lighthouse.

The main parameter, which has to be calibrated, is the bottom roughness, which in ZWENDL is represented by the Chezy roughness factor. An initial Chezy value for the bottom roughness of 55 m/s2 was taken.

Calibration

Calibration runs were undertaken for the following boundary conditions:

- at the downstream side of the model (seaward boundary between the piers) water levels generated by the calibrated FINEL2D model for the February 2002 event,
- at the upstream end of the model at the river boundary locations, discharges as determined in §2.4.2,
- the observed wind speed and direction from Dublin Airport. The wind direction was Southwest with a force 7 – 8 Beaufort.
- chlorosity of 19 kg/m3 at the sea boundary and 0.2 kg/m3 (fresh water) at the river boundaries

According to the FINEL results the wind direction (Southwest) resulted in a local water level set-down within Dublin Bay (off-shore wind). This set-down was small and only in the order of a few centimetres.

Within the ZWENDL model the wind is schematised in 12 hours blocks, over which time the wind is constant in speed and direction. Shorter blocks don't contribute to significantly more accurate outputs from the ZWENDL model.

The calculations were carried out using salt concentrations coupled with hydraulics by the density term: density differences are directly used in the calculation of water level differences and (their derivative) velocities.

Several runs were made with different bottom roughness factors.

The main problems experienced during the calibration were instabilities caused by:

- The low water channels through the mudflats between the River Tolka and the outer River Liffey channel in the port part of the model. This was solved by increasing the roughness to 40 m/s2.
- The steep (bed) slopes of the river Tolka (and to a lesser extent the bed slopes of the river Dodder). This was solved by increasing the roughness to 25 – 35 m/s2 and by expanding the ZWENDL computer model with a safe-guard against drying of a section.
- Zero flows within the Canals. This was solved by applying a small discharge (1 m3/s) through each.

A second series of problems was caused by the calculation of the drop over the bridges. In modelling there are several ways from sophisticated to simple methods to calculate the afflux between the up- and downstream faces of a bridge. In the ZWENDL model a combination of parameters can be used for subcritical and supercritical flow; for bridge pier losses and for submerged bridge decks; and for flow in both directions. Under usual circumstances the value of the parameter(s) would have been adjusted during the calibration. In this case however hardly any water level measurements were available.

Based on theoretical models and values of coefficients found in literature, an estimation has been made of the discharge coefficients that had to be applied.

Through the calibration process, the resulting Chezy roughness factors to be applied to the final model are as follows:

Port:	40 – 65 m/s2
Tolka:	25 – 35 m/s2
Liffey:	35 m/s2
Dodder:	downstream flow (during ebb tide) 40 – 45 m/s2, upstream flow (during flood tide) 35 m/s2

The results of the calibration are given in figures 13.4 – 13.9, Appendix M.

In **Table 13.7** water levels in the Lower River Liffey port region are given. In the following table results for the tide gauge at the Dublin Port Lighthouse are given for the highest water level of the 1st February 2002 flood and the preceding and following low water levels. The accuracy of the high and low waters is within 0.10 m

Table 13.7 - Water levels Dublin Port Lighthouse

Levels in [m] ODM	Observed	ZWENDL	FINEL
Low water 1	-1.68	-1.58	-1.59
High water	2.95	3.04	3.01
Low water 2	-1.48	-1.46	-1.45

In figure 13.4b water levels along the River Liffey are given. The results are summarised in Table 13.8.

The flooding level for the Ha'penny bridge can be determined in two ways:

- The quay level from the DEEP survey is 3.2 m ODM,
- According to reports, the level came to within 100 mm of the opening at Halfpenny Bridge: 3.1 m ODM

The flooding level for the Sir John Rogerson Quay (and City Quay) is determined from reports, which indicate that these quay's were flooded. The street levels of Sir John Rogerson Quay vary from 2.80 – 3.30 m ODM and those for the City Quay from 2.75 to 3.00 m ODM.

The model compares very well with the estimates for the Rory O'More and Ha'penny bridges. The results for the Sir John Rogerson Quay and City Quay indicate that these quay's are at least partly flooded in the model.

Table 13.8 - Water levels Liffey

Levels in [m] ODM	observed	ZWENDL
Rory O'More bridge	3.12	3.10
Ha'penny bridge	3.10	3.10
Sir John Rogerson Quay	>2.80	3.06

In figure 13.5a water levels along the River Dodder are shown and the results are summarised in **Table 13.9**.

Table 13.9 - Water levels Dodder

Levels in [m] ODM	observed	ZWENDL
Race Track	> 2.60	3.07
Stella Gardens	3.00 – 3.05	3.08
Landsdowne bridge	3.25	3.11

The water levels at the Stella Gardens and race track appear consistent given the flooding that occurred. The Landsdowne Road bridge observed level is higher than that of the model. However in this case the observed water level at Landsdowne Road bridge is doubtful and an explanation to back up this consideration is given below.

In **Table 13.10** the observed and calculated water level (and bottom) slopes from the confluence with the Liffey to the Landsdown Road bridge are compared.

Table 13.10 - Waterlevel and bottom slopes Dodder

Location	Water level [m]		Distance [m]	Slope: water		Slope: bottom
	observed	ZWENDL		observed	ZWENDL	
Liffey	2.95	3.04				
			717	0.075	0.045	1.3 – 1.5
Stella Gardens	3 – 3,05	3.08				
			573	0.225	0.025	0.75
Lansdowne bridge	3.25	3.11				

When the calculated (ZWENDL) and the observed water level slopes are compared (and adjusted for the river bed bottom slope), the calculated water level slope seems more consistent with the bottom slope than the observed water level slope. Therefore the measured water level at Landsdowne Road Bridge might be slightly on the high side.

In figure 13.5b water levels in the Royal Canal are given. ZWENDL calculates the water levels at 3.10 m ODM. However there are no observed levels available in this area.
In figure 13.6 the water levels at the Sea and the River Tolka are shown.

The influence of the high water of the 1st February flood extended as far as weir 11 (between Drumcondra Road bridge and Industrial Estate Road bridge) on the Tolka.

In Table 13.11 the results for the tidal (FINEL) model for warning point 16 (mouth of the Tolka) are given for the highest water level of the 1st February 2002 flood. The accuracy of the high water is within 0.10 m.

Table 13.11 - Water levels warning point 16 (sea)

Levels in [m] ODM]	ZWENDL	FINEL
High water	3.07	3.01

For the Tolka River no observed water level information was available for the 1st February 2002 event to help with calibration of the model and so the model has not been calibrated on this event. However, given the extensive modelling of the river through the River Tolka Flood Study and the short reach of the river under consideration within this study, it was considered that it would be sufficient to compare the model results with those of the Tolka study for another event. Furthermore the reach of interest to this study, i.e. below Annesley Bridge , it situated at the down stream end of the tidal reach and so will be less influenced by high fluvial discharges. Nevertheless a comparison was undertaken for the November 2002 event, whereby the results of the ZWENDL prediction spreadsheet (see section 13.4.4) have been used to determine the water level profile which has been compared with the calibration run of the River Tolka Model. It should be noted that for that particular event, there was a blockage under the rail bridge, just upstream of the John McCormack Bridge, which would not be accounted for in the ZWENDL model. The results are presented below and are reasonably comparable to the Tolka Model results and observed levels.

The hydraulic conditions for the November 2002 event were a discharge of 97m3/s and a tide level of 1.513mODM. The River Tolka Study model calibrated, and observed, levels were taken from Table 8.2 of the River Tolka Flooding Study Report.

Table 13.12 - Comparison water levels River Tolka November 2002 Event

Location	**Observed Level mODM**	**Tolka Model Level (mODM)**	**DCFPP Model (mODM)**
East Point Business	1.52	1.74	1.51
John Mc Cormack Bridge	1.66	1.82	2.05
Footbridge u/s Dart Bridge	2.38	2.277	2.43

In figure 13.7 the water levels in the Liffey and Dodder are shown. The Liffey is tidal as far as the Islandbridge weir. The Dodder is tidal as far as the Balls bridge Weir.

In figure 13.8 the water levels in the Royal Canal and Grand Canal are given.

Note that because of drying out problems, i.e the model has difficulty with very small flows at low stages of the tide, the low water levels might be less accurate in the February 2002 situation. In this situation the river discharges were very small. In order to prevent model crashes because of drying, artificial (deep and very narrow) channels in the river bed have been inserted into the model. These don't contribute to the flow but prevent theoretical drying of the river. In figures 13.4 – 13.8, this can be observed by the constant (low) water level near the point of drying of the river section.

In figure 13.9 the chlorosity in the rivers Tolka and Liffey are shown for reference. Note the relatively low chlorosity in the Liffey, while the chlorosity in the Tolka amounts to 6 even at Drumcondra Road bridge. This reflects the much higher discharge in the Liffey compared to the Tolka.

13.3.3 Conclusion

The ZWENDL model has been calibrated with emphasis on high water levels. The high water levels in the model area compared to the observed levels and inundated areas are reproduced sufficiently accurately for the purpose of this study, given the lack of gauge data at the upstream end of the rivers and the lack of reliable observed water levels for the February 2002 event. The February 1st event can be reproduced by the ZWENDL model to within 10 cm accuracy.

The goal of this study is to carry out scenarios for the development of a flood forecasting system and also to provide details of extreme water levels for flood risk assessment and development of alleviation options. ZWENDL will be used to calculate an accurate spatial distribution of the (astronomically driven) water levels at Dublin Lighthouse to the warning points, with consideration of the influence of the discharges and the local surge. Absolute water levels are of less importance than the (relative) water level differences between Dublin Lighthouse and the warning points. Therefore the results of the ZWENDL model are satisfactory.

Note that the river Tolka has not been independently calibrated due to lack of observed water level data within the study reach for the 1st February 2002 event. However, given the short reach of the river within this project and the extensive verification work already undertaken through the River Tolka Flood Study for the November 2003 event, it was considered that a comparison of the results of the ZWENDL model with those of the Tolka calibrated model are valid.

The resulting calibrated Chezy bottom roughness factors are:

Harbour:	40 – 65 m/s2
Tolka:	25 – 35 m/s2
Liffey:	35 m/s2
Dodder:	downstream flow 40 – 45 m/s2, upstream flow 35 m/s2

13.4 Scenarios

13.4.1 Methodology

Along the rivers Tolka, Liffey and Dodder in Dublin , 40 output points have been defined to provide information on water level for use in development and operation of a water level forecast system. These so-called Warning Points (WP's) were chosen in the vicinity (downstream) of each bridge along the rivers in Dublin. This setup was chosen mainly due to model constraints (i.e. maximum allowable output points) and it is considered sufficient for the purpose of providing forecast water levels along the rivers. However, for the purpose of assessing the flood risk it was important to investigate the head loss past bridges and this was undertaken for specifically important runs in the model source. However, all the Matlab and excel spreadsheet post-processing modules were based on the former downstream warning points and so for the forecasting tool the number of output points has not been changed.

The calibrated ZWENDL river model has been used to determine the water levels for each warning point. At the seaward end the water levels are the combined effects of (local or distant) wind induced water level surge and the astronomical tide. At the rivers however, the discharge is the dominant factor that determines the water level, depending on the distance from the sea, the slope of the river bottom and the presence of weirs. In FINEL, the river discharge could be neglected because of its very limited influence on the sea water levels.

For the warning points near the sea, a number of scenarios have been simulated [lit 1.1], each of them giving a unique combination of wind (direction and force) and tidal conditions. Tidal conditions have been simulated by carrying out a complete neap-spring tidal cycle. For those warning points along the rivers within the city of Dublin, a number of scenario's with varying discharges have also been considered. The basic principle of this approach is directed to a comparison of the difference in water level from the warning points to the lighthouse for a high water level with those high water levels at the lighthouse.

In accordance with the methodology for the tidal warning points around the coastline, the prediction of the water level at each river warning point consists of the following components:

1. The astronomical high water level at the river warning points relative to the astronomical water level at Dublin Lighthouse. For the analysis a complete neap – spring tidal cycle has been calculated.
2. The surge prediction of the UKMO Shelf Seas Model, the local surge from the UKMO point to the Lighthouse and the surge from the Lighthouse to each Warning Point. The surge prediction for the UKMO model and the transfer of that surge from the offshore point to the Lighthouse, has been dealt with as part of the FINEL modelling. In this study, only the local surge from the Dublin Lighthouse to each Warning Point along the rivers due to wind speed has been modelled and has been calculated, in the following way:
   A. The surge created from outside the ZWENDL model is simulated by applying a water level increase on the astronomical tide of 0.5, 1.0, 1.5 and 2 m. on all discharge combinations.
   B. The local wind induced surge has been calculated for a situation with 1m water level increase with respect to the astronomical tide and a once a year discharge combination, for a number of wind scenarios:
      - 8 wind directions (N-NE-E-SE-S-SW-W-NW)
      - 2 constant wind forces: 15 and 30 m/s;
3. Seven discharge combinations for the rivers Tolka, Liffey and Dodder: estimated yearly low, average and high discharge situations, and the 1/1, 1/10,1/100 and 1/1000 yearly discharges have been applied and the effects of the discharge on the water levels from the Lighthouse to each Warning Point have been calculated.

The first two components (1 and 2) are in line (with some small variations) with the method applied to the prediction of the coastal warning points and these are discussed in detail in **Chapter 11**, which deals with the tidal FINEL modelling. The last component (3) has been included for the prediction of the river warning points in relation to fluvial discharges.

In addition to those two variables for the prediction of the coastal warning points, there is an additional independent variable (discharge) that should be considered in the statistical calculations. Because of interdependencies this is a more complicated variable.
In order to diminish the number of independent variables (from the two variables: astronomical tide and outside surge levels to one variable: water level Dublin Lighthouse), the dependency of the relationship between water levels at Dublin Lighthouse to the river warning points has been investigated for varying (outside) surge levels. The main problem to be solved here: is it reasonable to neglect the way the tidal wave travels upstream within the rivers and is it possible to use only the high water level in Dublin Lighthouse, the discharge in the rivers and the local wind, to predict the water level relations for the river warning points?

This method seems very reasonable because of the dominating influence of the river discharges on the form of the water level course in the rivers compared to influence of the outside water level boundary conditions.

In the next paragraph, this method shall be proved to be feasible.

13.4.2 Simulations with different (outside) surge levels and river discharge

An investigation into the dependency of the relationship in water level between the Dublin Lighthouse and the warning points for different levels of surge has been undertaken in the following way:

Simulations were made with 0.0 (which represents the actual astronomical tide, i.e. no surge component), 0.5, 1.0, 1.5 and 2.0 m (outside) surge values combined with varying river discharges. The water level differences from the Dublin Lighthouse to the warning point were compiled for all these scenario's.

The ZWENDL model was run for 14 days, which represents a complete neap-spring tidal cycle. For each warning point, the difference for each high water level between the Lighthouse and the Warning Point was calculated. A relationship was then established between the high water level at the Lighthouse and the difference of the high water level between the Lighthouse and the Warning Point for the complete tidal cycle (neap – spring).

The water level relationships between the warning points and Dublin Lighthouse for all discharges with outside surge values ranging from 0.0 (astronomical) to 2.0 m are shown in tables 13.13 to 13.20.

The differences between the water level at the warning points and the Dublin Lighthouse were calculated for spring tide, average tide and neap tide. For simplicity (in this case the investigation of influence and distortion of tidal wave in the rivers), the astronomical tide levels without surge, have been taken as 1.25 mODM for neap tide, 1.70 mODM for average tide, and 2.15 mODM for high spring tide. For conditions with surge, the amount of surge has been added to these values.

In the tables it can be seen that generally the water level difference is independent of the amount of surge applied to the astronomical tide. The variation is at most 10 cm but usually only about 1 – 2 cm. There are two exceptions to this general rule:

1) Greater differences are noted between (river) warning points within tidal influence and (river) warning points outside the tidal influence. The water level for those points above the tidal influence are independent of the tide and only dependent on the river discharge. Warning points above the tidal influence show a difference that only reflects the difference in bed height between the warning point and the Dublin lighthouse. Naturally the tidal influence is not set at a finite location and there can be a transition zone, particularly where the influence is not bounded by a high weir such as Islandbridge. Therefore in some cases a number of the points lie in the transition zone between total tidal influence and total fluvial discharge influence. This transition zone can move depending on the conditions, i.e. a high tide level or discharge event and hence influence on particular warning points can change. Warning points within such transition zones are shown marked in bold (black or red) in the tables.

2) For very high surges (2 m) the difference in water level for warning points in the upstream part of the Liffey, close to Islandbridge weir, are diminished. This is probably caused by the increasing water depths and the relative decreasing resistance in the model.

Note that the area of tidal influence and discharge influence are not static but dependent on the discharge (and to a lesser extent to the tide). For very high discharges the area with tidal influence is much smaller then at low discharges (compare Table 13.19 with 13.13)

The main conclusions that can be drawn from the investigation into the dependency of a relationship between the water level at Dublin Lighthouse and that at each warning point for different levels of surge are as follows:

- **It is possible to use only the high water level in Dublin Lighthouse, the discharge in the rivers and the local wind, to predict the water level relationship for the river warning points.**

- **There is no need for astronomical tide and surge to be treated as different independent variables in the statistical calculations.**

13.4.3 Simulations with different local wind surge levels

To investigate local wind surge, the ZWENDL model was run again for 14 days, which covers a complete neap-spring tidal cycle, together with a number of wind scenarios, which included:
- 8 wind directions (N-NE-E-SE-S-SW-W-NW).
- 2 wind forces: 15 and 30 m/s.

Tables 13.21 to 13.22 present the relationship between the water levels at the warning points and Dublin Lighthouse for one simulation with a once a year discharge and an outside surge of 1.0 m, subject to varying wind speed and direction. The difference in water level from the warning points to Dublin Lighthouse have again been calculated for a spring tide, average tide and neap tide. For the astronomical tide plus 1 m surge, the levels have been set at 2.25 m for neap tide, 2.70 m for average tide, and 3.15 m for spring tide.

The difference in water level at the warning points (related to Dublin Lighthouse) seems to be just weakly dependent on the tide: with a wind speed of 15 m/s the difference is about 1 – 5 cm. Only for a wind speed of 30 m/s and only for easterly wind directions in the upper reaches of the Liffey near the weir,does the model indicate a stronger dependency on the tide; about 10 cm.

13.4.4 Analyses of the water levels and the prediction spreadsheet

The same simulations as mentioned in Paragraph 13.4.2 (Simulations with different outside surge levels and river discharge) and 13.4.3 (Simulations with different local wind surge levels) have been used, but have been analysed in a different way. Relationships between the water level at Dublin Lighthouse and the difference in water level to the warning points have been fitted and will be used to predict water levels at each warning point based on the water level at the Dublin Port Lighthouse, discharge in the river and wind speed and direction. To predict the water levels at the warning points an Excel spreadsheet has been composed based on these calculated relationships. The screen dump showing the front end of this spreadsheet is shown in Figure 13.109 below.

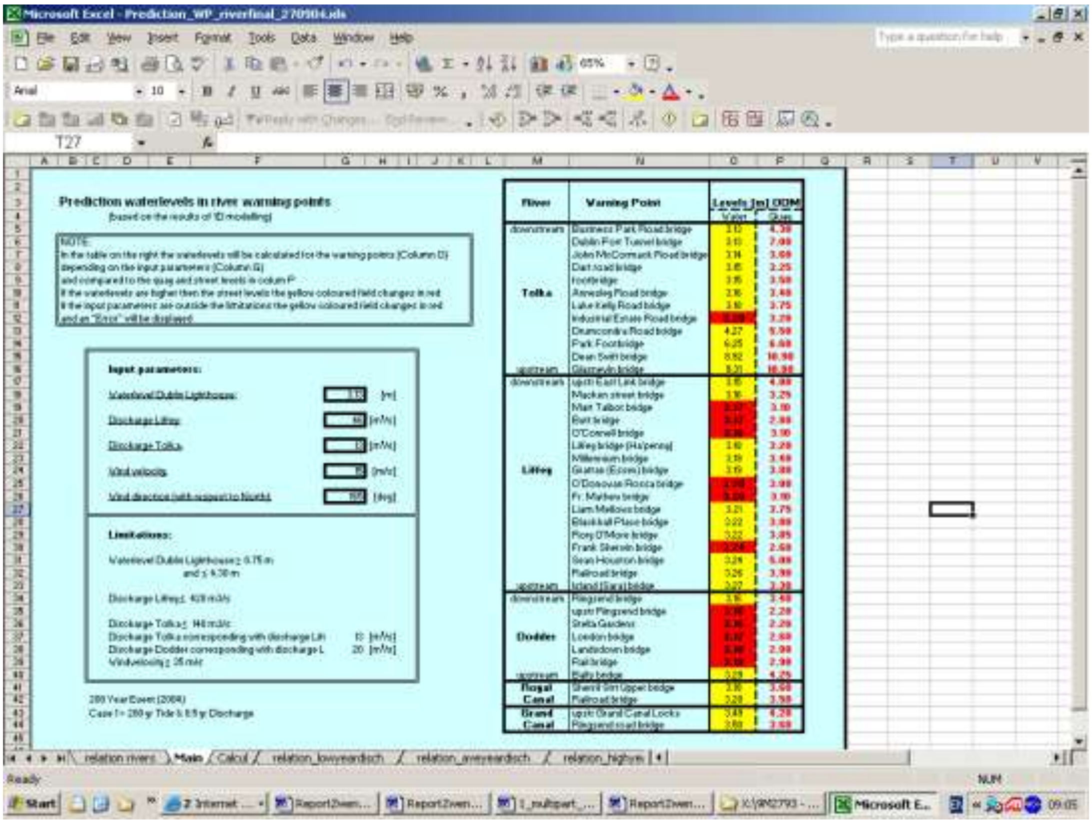


Figure 13.109 – Example of the Front End (Worksheet Entitled "Main") of the Prediction Spreadsheet

Since it has been proven that the high water levels on the rivers can be determined through information on river discharges, water level at Dublin Lighthouse and wind velocity with direction, relationships have been established for the following conditions:

- The high water level at the Lighthouse compared to the difference of the high water level between the Lighthouse and the Warning Points for all surge levels with varying discharge.
- The high water level at the Lighthouse compared to the difference of the high water level between the Lighthouse and the Warning Point for a surge level of 1 m and a once a year discharge with varying wind velocity and direction.

Initially only linear relationships were considered, however it became clear that the relationships themselves were not linear but dependent on the Dublin Lighthouse level. If only a linear relationship was chosen it would not be sufficiently accurate to predict water levels along the tidally influenced section of the rivers. The cause of this non-linear behaviour has been discussed in §4.20, but is summarised again below as:

- the location of the warning points, whether located within areas of tidal influence or areas outside tidal influence and
- the areas of tidal influence and river discharge influence are not static but dependent on the discharge and to a lesser extent, tide level.

The relationship between the water level at Dublin Lighthouse and the difference in water level to the warning points for low, average and high yearly discharge are shown for each warning point in figures 13.11 to 13.31 and for a 1/1, 1/10, 1/100 and 1/1000 year discharge in figures 13.32 to 13.59 respectively. Each high water is shown with a x and a third power line fitted through the points to establish a relationship for the prediction of the water level at the warning points with respect to conditions at the Dublin Lighthouse.

An examination of the plots leads to the following remarks:

1) **Type of relationship.** These plots are showing very clearly that the relationships are non-linear. By trial and error with different kinds of relationships it could be proved that a third order relationship had to be applied to provide a suitable fit and ensure that predictions of water levels are sufficiently accurate. Moreover, the range of Dublin High water level had to be divided in parts for some warning points, each with its own relationship (linear or third order), to improve accuracy. The figures shown in the report are for those relationships which were considered to best fit the data and no other figures produced during the trial and error process are presented.

2) **Tidal influence.** Some warning points show (some partly for low water levels at Dublin Lighthouse, others over the total range of water levels) a perfectly linear relation. This is a warning point lying outside the tidal influence (too far away from the sea). It is no surprise that these warning points are situated in the upstream reaches of the model. Each amount of decrease of water level at Dublin Lighthouse gives an equal amount of increase of water level difference (between Lighthouse and warning point), concluding to a constant water level at the warning point. As mentioned before (paragraph 4.2), this tidal influence area is not constant but depending on:

   - Outside water level at Dublin Lighthouse. The lower the water level, the more probable the water level at the warning point is not influenced by tide.

- River discharge. The higher the discharge, the more probable the water level at the warning point is not influenced by the tide.

3) The scatter. Note that warning points lying outside the tidal influence show no scatter at all. For points lying inside the area of tidal influence the amount of scatter in the plots is dependent on:

   - Discharge. Low discharges seem to increase the amount of scatter to a small degree (maximum 10 cm for lowest discharge to maximum 7 cm for highest discharge).
   - Location of the warning point.
     - Tolka upstream 0 cm to 7 cm downstream at East Wall Business park road bridge
     - Liffey upstream 10 cm (low discharge) and 2 cm (high discharge) to downstream 1 cm (low discharge) and 5 cm (high discharge)
     - Dodder upstream 5 cm (low discharge) and 2 cm (high discharge) to downstream 2 cm

   The dependency of the river must be seen in relation to the probability of lying outside the tidal influence. The Liffey (below Islandbridge) is always under the influence of the tide. The Tolka is almost never under influence of the tide. The Dodder is at times under the influence of the tide.

   The cause of this scatter is most probably twofold:

   - In paragraph 4.2 the influence of the outside surge as dependent variable and the way, the tidal wave travels upstream was discussed and considered to be negligible. However, there is probably some influence: at most 10 cm but usually only about 1 – 2 cm.
   - In the post processing programs the time between the tidal High Water levels (period of the tidal wave) is determined as accurately as possible. However, some inaccuracies might be present and small phase differences can cause small differences in High water level.

   In conclusion it is considered that the scatter is mostly artificial and caused by assumptions and inaccuracy in the determination of the phase of the tidal wave. The amount of scatter is on average 3 – 5 cm and at most 10 cm

4) The accuracy of the relation. The accuracy of the relations is dependent on the amount of scatter. In the preceding point (3) it is shown that the scatter is on average 3 – 5 cm and at most 10 cm. It should be noted however that the relationships are based on the averages of the points to be fitted. As a last remark with respect to accuracy: Some negative values in relationship between the lighthouse and the warning points (water level warning point less then outside water level) are noted on the downstream end of the River Tolka. However, these are at most 1 - 2 cm and hence can be neglected. They are probably caused by small phase differences in High water level.

In total 61 scenarios have been successfully calculated, which form the basis of the prediction spreadsheet, which in turn can be used as part of an water level prediction system. The results of these calculations are collated in a spreadsheet, and for each combination of:

- discharge of the rivers Tolka, Liffey and Dodder (input),
- water level at Dublin Lighthouse (input) and
- local wind speed and direction (input),

predicted water levels for all 40 river warning points (output) can be determined. The front end sheet entitled “Main”, of the prediction spreadsheet was shown earlier in Figure 13.4.49. It demonstrates how by typing in the relevant input parameters mentioned above, the resulting water levels at each of the warning points are calculated and compared with the quay levels. This comparison returns a safe result (yellow) when the river water levels are lower then the quay levels, or an unsafe result (red) when the river water level at a certain warning point exceeds the quay level.

Only input parameters within the range used in the ZWENDL calculations can be used within the spreadsheet, and as such restrictions on these input parameters are built in the spreadsheet and include:

- Allowable range of water level at Dublin Lighthouse (0.75 mODM – 4.3 mODM).
- Allowable range of river discharges (Liffey ≤ 420 m3/s, Tolka ≤ 140 m3/s and Dodder 320 m3/s).
- Allowable range of wind speeds (≤ 35 m/s).

In a separate sheet entitled “relation rivers” within the prediction spreadsheet, the discharge relationship between the three rivers is shown. The discharge of the Dodder is used in combination with the discharge of the Liffey, the discharge of the Tolka is used independent of the Liffey. Therefore in the spreadsheet discharge values for the Tolka can be entered independently, while for the Dodder they are related to the input value chosen for the Liffey. This relationship was considered to be practical given that both the Liffey and Dodder catchments are located in geographically similar regions and might be affected similarly by a given event. However, should a discharge in the Dodder be required, which does not comply with this assumed relationship, it is easy to manipulate the spreadsheet to give the correct backwater curve in the Dodder by considering the Liffey discharge necessary to give the required discharge in the Dodder. This is made possible by the fact that the influence of fluvial discharge on water levels at the East Link Toll Bridge, i.e. the down stream end of the rivers, is minimal and so a manipulated flow in the Liffey will have little influence on the Dodder water levels.

The results of the spreadsheet calculations are plotted in figures 13.60 to 13.108. These plots show the relationship for each warning point as used in the spreadsheet, with dependent of the water level at Dublin Lighthouse for all discharges and surge levels (but without wind). Notice that most of the time the 3rd power relation has been applied, only in special circumstances (extreme low or high water levels at Dublin Lighthouse), is a linear relationship noted to be more applicable. The formulas used in the spreadsheet calculations can be found on the worksheet entitled “Calcul” of the spreadsheet, and the relationships themselves on the remaining sheets of the worksheet.

13.5 Conclusions and recommendations

13.5.1 Conclusions

A river model has been constructed which combines all of the three main rivers within the study area and which has its seaward boundary located at the end of the Bull Wall and Great South Wall training walls. This provides a big advantage in that the model is a complete integrated model of all three rivers and not simply three stand alone and independent river models. The river model ZWENDL was used for the creation of this model.

The ZWENDL model has been calibrated with emphasis on high water levels. The high water levels in the model area compared to the observed levels and inundated areas are reproduced sufficiently accurate for the purpose of this study, given the lack of gauge data at the upstream end of the rivers and the lack of reliable observed water levels for the February 2002 event. The February 1st event can be reproduced by the ZWENDL model to within 10 cm accuracy.

The goal of this study is to carry out scenarios for the development of a flood forecasting system and also to provide details of extreme water levels for flood risk assessment and development of alleviation options. ZWENDL will be used to calculate an accurate spatial distribution of the (astronomically driven) water levels at Dublin Lighthouse to the warning points, with consideration of the influence of the discharges and the local surge. Absolute water levels are of less importance than the (relative) water level differences between Dublin Lighthouse and the warning points. Therefore the results of the ZWENDL model are satisfactory.

Several problems were experienced during the calibration.

1) Most important were instabilities caused by:

   - The low water channels through the mudflats between the River Tolka and the outer River Liffey channel in the port part of the model. This was solved by increasing the roughness to 40 m/s2.
   - The steep (bed) slopes of the river Tolka (and to a lesser extent the bed slopes of the river Dodder). This was solved by increasing the roughness to 25 – 35 m/s2 and by expanding the computer model ZWENDL with a safe-guard against drying of a section.
   - Zero flows within the Canals. This was solved by applying a small discharge (1 m3/s) through each.

2) A second set of problems was caused by the calculation of the drop through the bridge arches. In modelling there are several ways from sophisticated methods to simple calculations to determine the drop between the up- and downstream section of a bridge. In the ZWENDL model a combination of parameters can be used for subcritical and supercritical flow, for bridge pier losses and for submerged bridge decks, for flow in both directions. Under usual circumstances the value of the parameter(s) would have been adjusted during the calibration. In this case however hardly any water level measurements were available. Based on theoretical models, values of coefficients found in literature, and experience combined with common

sense checks, an estimation has been made of the discharge coefficients that had to be applied.

These problems have led to the conclusion that this model is very sensitive to drying of sections (instability) and the applied value of the bridge discharge coefficients (accuracy).

Note that the river Tolka has not been calibrated due to lack on water level data at the time the calibration was done. The calculated water levels in that river are only checked if they are within bounds of reasonability.

The resulting calibrated Chezy bottom roughness factors are:

Harbour:	40 – 65 m/s2
Tolka:	25 – 35 m/s2
Liffey:	35 m/s2
Dodder:	downstream flow 40 – 45 m/s2, upstream flow 35 m/s2

In total 61 scenarios have been successfully calculated as the basis of a water level prediction system comprising 40 Warning Points. The results of these calculations have been collated in a spreadsheet, which for each combination of discharge, water level at Dublin Lighthouse and local wind speed and direction, water levels for all 40 river warning points can be determined and compared with the quay levels adjacent to each warning point. This comparison giving a safe result (cell turns yellow) when the river water levels are lower then quay levels, or a unsafe result (cell turns red) when the river water level exceeds the quay level.

Examination of the relationships between the water level differences at the warning points and the Dublin Port Gauge reading indicated that most of these were non-linear. Most of the non-linear dependencies were best described by a third order function throughout their entire range while for some warning points the relationship was in part linear and in part third order.

Not surprisingly the warning points which displayed a totally linear relationship are those lying furthest upstream and above the tidal influence.

The amount of scatter (hence uncertainty) in these relationships ranged from virtually none at all for those warning points situated above the tidal zone to a maximum of 10cm elsewhere; the average scatter being of the order of 3 – 5 cm . This scatter is most probably due to ignoring (i) the influence of outside surge as a dependent variable, and (ii) the effects of the tidal wave as it travels upstream (and downstream). The magnitude of these discrepancies was estimated at a maximum of 10 cm but usually only about 1 – 2 cm.

Another cause is inaccuracy in the determination of the period of the tidal wave in the post processing programs. Small phase differences can cause small differences in High water level. It can be argued that the scatter is mostly artificial and caused by assumptions and inaccuracy in the determination of the phase of the tidal wave.

13.5.2 Recommendations

Sea:

Recommendations relevant to this aspect of the river modelling work are covered by the recommendations presented in chapter 11, tidal hydrodynamic modeling

Rivers:

- The river Dodder has been schematised to Ballsbridge weir as required by the brief. Tidal influence however stretches upstream of this location. No data of this hydraulically complicated part of the Dodder was available. It is recommended for this reason and also because of the nature of the extreme fluvial flood risk, as identified in Chapter 15, that a model study of the complete Dodder catchment, similar to that completed for the Tolka, be undertaken to consider the issue of fluvial flood risk on a more detailed and catchment wide basis.

- Rehabilitation of the river gauging station on the river Liffey in the vicinity of the Island Bridge weir. A working river gauging station at this location is essential in determining the discharge of the river Liffey under the full range of fluvial conditions.

The main purpose of the river model was for the prediction of high water levels (floods). For an accurate prediction a thorough calibration is necessary. This implies calibration for situations with high discharges and high outside (sea) water levels. The February 1st 2002 event is an excellent example of the last one. However during that event the river discharges were very low. Since it is recommendable to calibrate river models with high discharges, this event is not the best one. However since reliable field observations were available only for this event and because of the importance of this event, the ZWENDL model has been calibrated on the flood of February1st 2002. However, the absence of calibration data for high discharges is a cause for concern in relation to the accuracy of the model predictions for high fluvial combinations. It is strongly recommended that further validation checks of the model be made as soon as new data becomes available.

Note that the river Tolka has not been calibrated due to lack of observed water level data within the study reach for the 1st February 2002 event. However, given the fact that the Tolka model is only one part of a larger integrated river model and considering the short reach of the river within this project and the extensive modelling work already undertaken through the River Tolka Flood Study, it was consider that a comparison of the results of the ZWENDL model with those of the Tolka calibrated model would be sufficient in combination with the calibration work undertaken for the other rivers.

- Validation is the process of checking the model settings found through the calibration against a period other than the calibration period. At the time of the calibration of the model, no reliable water level data with matching discharges for situations with high discharges were available. Therefore it was not possible to validate the model successfully. It is highly recommended that new water level and flow measurement gauges be installed at suitable locations at and within the tidal reaches of both the rivers Liffey and Dodder for future use in model validation. It is also recommended that models of both the Liffey and Dodder are extended upstream and further investigation into the fluvial flood risk be undertaken.

- The bridge discharge coefficients as used in the model, are estimated and derived from theoretical situations. It is recommended to carry out water level measurements upstream and downstream of a drowned bridge during high river discharge for future use in bridge calibration.

14 FORECAST SYSTEM & EARLY WARNING RESPONSE

14.1 Forecast system

14.1.1 Introduction

The forecasting of extreme tide levels and potential coastal flooding events draws on the individual elements of the study. These elements, namely:

- The joint probability analysis of tides and surges to determine the return period of the February 2002 event also enables the prediction of extreme tide levels, against which the standard of protection of the current defences can be assessed.
- The defence asset survey database identifies the current state of the defences.
- Numerical modelling of tides, waves and overtopping provide significant detailed information about the distribution of the wave and tidal conditions across the study frontage under normal and extreme tides.
- Numerical modelling of overtopping provides information against which the standards of defence can be compared, and from which the trigger level criteria are derived.
- Flood compartment mapping enables the assets at risk to be identified in a consistent manner, as well as being able to assess the effectiveness of the options proposed to mitigate future flooding.

Each element of itself, whilst important as a decision making tool, does not provide sufficient information to enable a coherent forecast to be made of potentially destructive tides and wave conditions.

The forecast system employs the TRITON user interface to combine the results of the individual components in a manner that provides Dublin City Council with better flood forecasting of future events.

14.1.2 Development of transfer functions

Transfer functions are used to convert offshore conditions to nearshore conditions without the need to rely on the constant use of numerical models. The running of the various numerical models in real time immediately prior to a flood event would be too slow to provide practical advanced warning of the flood event. Pre-developed transfer functions on the other hand provide a practical and more effective means of setting up and running a forecast system using real time data. The transfer system does not require any model processing time, but simply requires input forecasts from the UKMO model, which can then be read through the transfer functions to give an instant estimate of the nearshore conditions from which a warning may be triggered.

A range of different transfer functions have been developed through the numerical modelling work described in the earlier chapters. This range includes transfer functions to,

- Convert offshore surge forecasts into nearshore surge estimates around the project area
- Convert offshore wave forecasts into nearshore wave climate forecasts

- Convert wind speed and direction forecasts into nearshore wave climate at specific locations as required
- Convert nearshore wave climate and water levels forecasts into overtopping estimates.

14.1.3 Flood warnings

In the UK, the Environment Agency currently employs a flood warning system that is built upon a four-tier warning approach, with each tier getting progressively more serious. These are:

- Flood Watch
- Flood Warning
- Severe Flood Warning
- All Clear

For the purposes of the Dublin system the Triton system flood warning trigger values have been based on the four tier system, although it should be noted that the Flood Warning Category is subdivided into two as shown in Table 14.1 below, to enable staged warnings (or more targeted warnings) to be issued to parties at risk. The probity of such an approach is to be agreed prior to the system going live.

Table 14.1 - Flood Warning Categories

All Clear	This indicates that there is no imminent risk of water levels reaching a point that requires further monitoring.
Flood Watch 	This indicates that overtopping is likely to occur at a level that could affect pedestrians and or motorists using walkways and roads adjacent to the seawall.
Flood Warning A 	This indicates that flooding is expected and that properties will be affected. It is considered that this would represent the first row of houses adjacent to a sea defence being flooded.
Flood Warning B 	This category would be used if a second area including small numbers of properties were to flood at a higher trigger level than for a normal flood warning, but for which a severe warning is not justified because the total risk is not sufficiently high.
Severe Flood Warning 	This indicates that there is an extreme risk associated with flooding. The value will be set such that if greater than 100 properties were at risk of flooding than this would initiate a severe flood warning.

Note: The symbols shown above are those employed by the UK Environment Agency and are included here as examples of how the various warning categories are differentiated visually.

14.1.4 Trigger criteria

In workshop No.1, the concept of Trigger Criteria was introduced to the stakeholders and a discussion held on what should constitute appropriate criteria. In August 2003 the Environment Agency published a new version of “The Flood Warning Code System”, which gave simple definitions for each of the four tiers of flood warning. In October 2003 a new work order was published by the Environment Agency, for which our best interpretation defines the triggers for flood watch, flood warning and severe flood warning as follows:

i) **Flood Watch:-** The trigger for Flood Watch is a forecast that flooding of low impact land is expected.
ii) **Flood Warning:-** The trigger for Flood Warnings is a forecast that flooding of property or high impact land use is expected.
iii) **Severe Flood Warning:-** The trigger for a Severe Flood Warning is a forecast that flooding of property or high impact land use is expected together with imminent danger to life.

These definitions can be broadly viewed as being in agreement with the definitions in Table 14.1. However, as the defences have been assessed in terms of their current standards of defence, there is no advantage to be gained in refining the definition. Moreover, categorising an area as ‘low impact land’ may inadvertently convey the impression that dealing with flooding in such an area would not be considered as high priority.

As the DCFPP study has progressed it has emerged that the most important criteria upon which to base the warnings in Table 14.1 are:

- Water level
- Overtopping by waves

Furthermore the river discharges and the accuracy of the forecasts will be taken into account. The river discharges will be discussed in the following section (see section 14.1.5), whilst in the remaining only the coastal aspects will be discussed. Regarding the accuracy of the forecasts a safety margin will need to be taken into account to ensure that actions can be taken timely.

The above mentioned criteria could be used to trigger the categories set out in Table 14.1, provided the threshold or trigger level is set to an appropriate level to reflect the category in question. Within the system a threshold level for each of the above criteria will be set which could trigger each of the four categories. The appropriate level of the threshold would of course vary from location to location and is dependent on the following aspects and sub aspects:

- Asset related aspects:
  - The height of the defence structure
  - The condition of the defence structure
  - The assets behind the defence structure

- Operations related aspects:

- The time needed to issue warnings
- The time needed to put demountable and temporary defences in place
- The time needed for any other operational actions (see section 14.2)

As each criterion and each aspect is interrelated, over time the trigger values could be fine tuned through monitoring and feed back to improve the overall system in terms of accurate response. Initially the system will operate with default values.

A set up similar to the example shown below, see Table 14.2, can be used to issue appropriate flood warnings.

Table 14.2 - Flood Warning Trigger Levels (default values)

Flood Warning Categories	Water level, ∆H (m)	Wave overtopping, q (l/s/m)
Flood Watch	0.50	0.10
Flood Warning A	0.35	1.00
Flood Warning B	0.25	2.00
Severe Flood Warning	0.10	5.00

Obviously the flood warning trigger levels are also time dependent. Since a 36 hour forecast is given with twelve three-hourly predictions, it may well be possible that the forecast results in more than one trigger. Each trigger will need to be taken into account and dealt with in the operational response bearing the previously mentioned aspects in mind.

As mentioned before the system will initially be delivered to the client with default values for the trigger levels. In subsequent development phases a fine tuning of the trigger levels will occur dealing with the aspects mentioned before. When the system is fully operational these trigger levels will be monitored and adjusted as required to improve accuracy within the system.

14.1.5 Fluvial Flood Warnings

The issuing of flood warnings along the tidal reaches of the Rivers Liffey, Dodder and Tolka are governed by the state of the tide and the flow at the upperstream end. It is anticipated that the prediction spreadsheet discussed in Chapter 13 and presented in Figure 13.4.49, will be used in conjunction with the Triton system to provide forecast levels within the tidal river. The Triton system will be capable of providing a forecast for water levels at the East Link Toll Bridge (WP 19, Figure 11.20) and also will provide forecasts of wind speed. These will be fed into the prediction spreadsheet to provide a forecast of water level at the downstream side of each of the bridges within the tidal reaches. In the future a link will be provided within the Triton system which will allow access to the prediction spreadsheet and then there will be a requirement for manual inputting of the data to obtain the back water curve along the tidal reaches of the rivers.

Of course there is one other parameter which will affect the water levels within the river and this is the river discharge. This parameter can also be fed into the spreadsheet, however at present no model or system is available to provide a forecast of the likely flood. In such circumstances there will be a requirement for more operational intervention to obtain an estimate of river discharge which can be included within the spreadsheet to provide the best estimates of likely back water curve within the river reaches. It is suggested that some form or protocol has to be written which can be triggered by the Triton system when a storm surge event is forecasted which is likely to cause problems within the rivers. This could take the form of a staged or stepped approach which the operational forecast would be required to follow. Such a stepped approach might include,

1) Obtain most recent flow gauge data for the rivers. A telemetric link to these most important sites could be set up so the information is instantly accessible.
2) Contact ESB and request a forecast of likely discharge at the Liffey Dams over the next 36 hours and request that they keep the forecaster informed of likely changes to this. Contact operational staff at Bohernabeena Reservoir and request similar for the River Dodder.
3) Contact Met Eireann and request latest forecasts for rainfall over the river catchments.

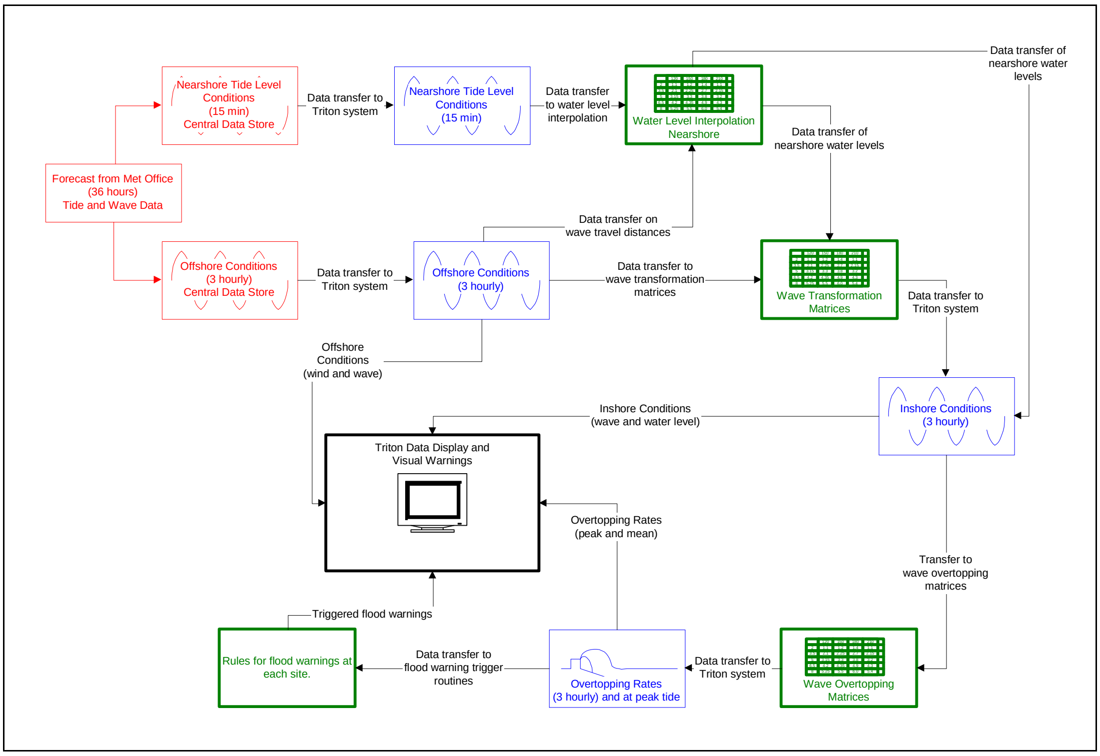
Using the above data a better informed judgement as to the likely flow in the river can be made and used within the forecast spreadsheet.

Consideration should be given to whether the development of a hydrological model which uses forecasted information from 2 and 3 above, could produce added benefit in respect of forecasting improved water levels within the tidal reaches of the rivers and indeed also upstream of these reaches.

14.1.6 Triton User Interface

The DCFPP flood forecasting system analyses forecasts of wind, wave and water level data to determine potential flooding of coastal regions. The system is based on existing available forecasts of winds, waves and water levels, by the Met Office for a point offshore in Dublin Bay. In the future it is likely that more data will be available for use, and the systems have been developed to take these into account.

The TRITON user interface is the operator's means of accessing the data created by the various stages of the project. The manner in which this information reaches the operator is illustrated in the flow diagram shown below.



Flowchart 14.1 - Data Flow through the TRITON User Interface

The elements shown in RED are external to the system. The elements shown in BLUE are stored data, whilst those shown GREEN are look-up tables/matrices.

14.1.7 User Manual and System Training

A user manual for the TRITON user interface is provided along with training in the use of the system itself.

14.1.8 Future monitoring requirements

Forecast System

The forecast system will require a period of formal trial testing before going 'live' and being used to actively issue flood warnings. The trial period should, were possible, include a winter period, where the risk from flooding could reasonably be expected to be at its highest.

During the trial period, it is deemed important that the following aspects be tested:

a) Delivery of data from the UK Met Office to the PC server.
b) The validity of the proposed trigger level values i.e. the levels above which warnings will be issued. These values are defined by the end user.
c) The validity of the tide predictions compared to the actual values. Currently this will only be possible for the water levels at the Alexandra Basin Tide Gauge. As more

tide level information becomes available, the levels predicted at intermediate warning points can be validated.

d) The validity of the UK surge and water level predictions.

Tide Gauges

Discussions are underway with the Marine Institute of Ireland for the installation of new tide gauges.

Flow Gauges

It is considered very important that a number of new/old flow gauges are installed/reinstated at or around the tidal limits of each of the three main rivers.

Future Proposals

The EWS proposals are currently being advanced to a joint partnership with Met Éireann, DCC and RH.

14.2 Early Warning System

14.2.1 The Components of an Early Warning System

For any flood forecasting and warning system to deliver benefit to the communities that it is intended to protect, it must be viewed as a coherent entity, if the overall objective is to reduce the risk to life and property of the general public. “The benefits of issuing flood warnings to the public are only realised if the dissemination is effective.” (Ref 1)

The established thinking on flood forecasting and warning systems looks at six components. These are:

- detection;
- forecasting;
- warning;
- response;
- evaluation; and
- improvement

The above components are linked and must not be considered in isolation. The first components are looked at in more detail below. The last two elements, i.e. evaluation and improvement, can only occur if adequate records of flood events are maintained.

Detection

At the coastal margin, detection takes the form of wind/wave/surge forecasts that are both forecast for specific sites and actual for locations distant from the target site. The distant locations help to verify the forecast. This requires forecast data (usually from the Met Office) plus tide gauge real time data.

Along rivers, detection will include access to flow and/or level data in real time together with forecast precipitation supplied by the Met Office. In Dublin weather radar is available although this is considered to be a real time system and generally only provides data up to two hours ahead of an event.

Detection of potential flood events in an estuarine environment requires a combination of the information required for detection of coastal and river flooding.

Forecasting

This compares what is happening (or forecast to happen) with the standards of existing defences. Forecasting looks at ‘where’ and ‘when’ the defences might be exceeded and for how long. Having forecast the exceedance, the next action must address the question of “what will the impact be?”. To do this requires access to an asset survey database and flood risk maps.

Warning

A flood warning terminology adopted (and understood) by all “interested” parties is an essential pre-requisite for any warning system. Without this it will be impossible to convey the seriousness of a message from which the necessary actions must flow.

However the terminology is developed and defined, if it is specific to an individual system, there will always be the possibility that the warning will fail, because of a lack of understanding on the part of the person issuing the warning or on the part of the recipient. It is therefore imperative that the terminology used is common to all those who may potentially need to use it. This will include the responsible local authorities of Dublin City Council and Fingal County Council, the Police, the emergency services, utility companies, hospitals, regional flood committees’ etc.

Hence, when developing the terminology, the needs of three target groups must be taken into consideration. These include:

1. general public who require information between three and six hours in advance of flooding;
2. the broadcast media who will use the warning messages within their broadcasting schedule; and
3. professional and emergency service organisations who have an operational role to perform and who may require a lead time to carry out that role effectively.

The main requirement of this is a message dissemination system similar to the Environment Agency’s (EA) Automatic Voice Message system (AVM), although the number of message recipients and the available lead time will influence system selection. The system needs to disseminate messages in a variety of formats (voice, fax, Text message, pagers, SMS, email). The system works from databases of recipients, messages and locations that all need periodic updates.

Response

The response required from any particular warning will vary according to the target group to which the warning is directed.

- **Members of the public:** The general public should know what to do because they are aware of the meaning of warning messages and the response they should make. This could be reinforced by encouraging the preparation of personal/family flood plans. To further embed this in the psyche of the members of the public, there will be a need for regular (annual) public information campaigns.

  A natural reaction to the issuance of a warning is the need to confirm its validity. In England and Wales the Environment Agency (EA) has established "Floodline"; which is a contracted out service that can both endorse the flood warning messages as well as provide background information. The Scottish Environment Protection Agency has also joined the "Floodline" service.

- **Broadcast Media:** If the broadcast media are to be used in disseminating warning messages, a protocol must be established, under which, the media will receive warnings within a specific timescale. The protocol must also include an agreement from the broadcast organisations, not to exercise editorial control over the wording of the message; all of which would follow a standard format. This will require a procedure (or series of procedures) to be agreed with the flood warning organisation, detailing how the message is handled, who is responsible for its delivery and manner in which it is delivered.

- **Emergency Response Organisations:** The emergency services and other organisations responding to a warning will need to amend their operating procedures to accommodate the flood warning messages that they may receive. They may want to establish a "feedback" mechanism whereby their operational controllers can liaise with the flood warning operators.

  To operate a flood forecasting and warning service, there must be staff (appropriately trained) and available, who can respond whenever the predetermined threshold values are forecast to be exceeded. There needs to be adequate facilities from which the procedures can be delivered.

  It is important that the entire process of receiving and responding to warnings is fully documented and cross checked against related procedures. The procedures will need to be checked (by exercise) routinely.

A log should be established to record all actions, from which evaluation and improvement can be initiated.

14.2.2 Requirements for Study Area

At an early stage in the development of an early warning system, the "target audience" for flood warnings should be identified. It is generally accepted that there are four distinct audience groups to which warnings may apply:

a) The members of the general public who are at risk, either because they live and work within a flood risk area or because they travel through such an area.
b) The professional services who provide the operational response to flooding and or the risk of flooding but who are not usually available 24 hours per day.
c) The emergency services who would deal with flooding as one of the range of hazards that they are trained and equipped to respond to.
d) The media organisations that would play an important role in the dissemination of warnings and in the updating of information throughout the event.

The type, number and distribution of the four groups need to be established in order that the size of any technological solution can be adequately scoped.

Experience has shown that the most effective way of determining the appropriate message format is through consultation with representatives from each of the above groups. The aim of these consultations is to ensure that the appropriate action is taken when flood warning messages are received.

14.2.3 Option for the Development of an Early Warning System

There is an extensive array of media whereby the delivery of the warning can be achieved to the target audiences. These include:

- **Telephone:** the mass use of the telephone within society, whether fixed landlines or mobile, means that it is a readily available medium for the dissemination of flood warning messages. A telephone based alerting/warning system would therefore be recommended as a major component in the arsenal available to the organisation issuing the warnings.

  To operate such a telephone based system a call centre approach or an automated dialling approach would be required, depending on the size of the calling database. Essential information required for the calling database includes:

  - A list of all the recipients of flood warnings;
  - A list of all flood locations that flood messages relate to;
  - Predetermined lists of recipients for each type of flood warning; and
  - The ability to handle text and voice messaging.

  It is feasible to record whether a recipient has received and/or acknowledged a message.

- **Personal Computer:** The wide availability of personal computers, both at home and in the workplace, means that the computer can be considered as a means of distributing flood warnings to a wide audience. As a messaging system it is similar to that described within the telephone system (above), and would enable messages to be sent to appropriate recipients via email.

  The advantages of the email system are that messages can be sent out using pre-arranged templates, and is a low cost option. The main disadvantage to using an email based system is that there is no comprehensive method to determine whether a message has actually been received by the recipient, although it is possible to set up an acknowledgement system for the recipients to respond with.

- **Television Text Messaging:** The advent of digital TV has enabled text messaging to be sent across the screen. There may however be broadcasting legislation that restricts or delays the implementation of this technology and therefore the option should be considered as a future enhancement rather than an initial requirement. Moreover, the success of this option is dependent on how many households have made the switch from analogue to digital signals.

- **Television and Radio Broadcasts:** The broadcasting of messages by radio and TV stations is an effective means of mass communication. The effectiveness varies according to the time of day the warning is broadcast. If it is to be used as a means of disseminating warnings, then both the flood forecasters and flood warning staff should liaise when compiling flood forecasts and warnings. Liaison and consultation with the broadcasting community is important from an early stage of planning the system. The broadcast media have a great desire for information and may fill any perceived gaps in information with their own interpretations which may lead to false impressions being created.

  Both Dublin City Council and Fingal County Council should consider whether any message handling technology already in use, or planned, by the professional partners and emergency services could meet the needs of a flood forecasting and warning system. The aim of a flood warning system should be that there is as little intervention by third parties as possible i.e. between the professional and emergency service partners receiving the messages and the flood warning staff issuing the messages.

- **Direct Warnings:** Direct methods of warning such as via a network of community volunteers should also be considered whereby flood warning messages are cascaded to the recipients via two or three community representatives. Such an approach works well where there is an established community spirit. It can engender a sense of ownership within the community who become keen to see improvements identified and implemented. The disadvantage is that when there is a long period between flood events local interest in the system declines. Moreover the effectiveness is diluted in communities where the residents are away for much of the time. The community volunteers (flood wardens) do need to have a link and feedback mechanism with the flood warning organisation.

  Other direct means of warning, such as the sounding of sirens, may be considered where appropriate. Installation, testing and maintenance of the system are required together with a public education campaign as to what the siren signal means.

  Vehicle mounted loud hailers can also be an effective method of disseminating flood warnings. The route of vehicles can be pre planned and the messages pre-recorded. The time taken to deploy the vehicles must figure in the consideration of the viability of this approach. At peak travel times, congestion in Dublin could potentially delay the delivery of the messages. However, if this type of vehicle is already available it can be a cost-effective solution.

Irrespective of the dissemination system used research has shown that there is a need for recipients to be able to verify the flood warning information that they have received. One method to provide this information is via a dedicated call handling centre (Floodline as used by the Environment Agency (England and Wales) and SEPA). Research has again indicated that the greatest reassurance is provided by the recipients contacting another “human” rather than a technological answering machine. Call centres can be totally remote from the flood site, the use of carefully scripted prompt sheets, to be used by call centre staff, has been shown to satisfactorily answer the majority of public enquiries received. Any call centre used in this way must also receive a copy of all flood warnings and preferably receive some advance notice that warnings may be issued.

14.2.4 Fluvial Considerations

The requirements for a fluvial flood warning system do not vary significantly from those required for a tidal/coastal system. The main difference between the two types of flooding is that the lead time required for the issuing of fluvial flooding warnings is generally shorter than that of tidal/coastal. Dependent upon the lead time required by the warning message recipients and on the reliability of the fluvial flood forecasting system there may be a greater number of inaccurate warnings issued.

In all other respects the components of a fluvial flood warning system are as described in the sections above.

14.2.5 Links to ERP and MEP

The Emergency Response Plans (ERP) and the Major Emergency Plan (MEP) focus primarily on the situation when a Major emergency has occurred. Flood warning actually provides preparation time and enables actions to be taken when a major emergency is likely. Some flood warnings, particularly if the system is also used for fluvial flooding, will only affect one or two areas, in which case it may not be appropriate to classify them as a major emergency.

The definition of flooding and flood warnings as a major emergency must be established in order that the definitions can be applied consistently by all parties involved.

The response to major flooding is somewhat different to other emergencies in that the Flood Risk Areas that will be affected are predefined, hence the access and egress routes can also be predetermined.

Review of the ERP and MEP is being undertaken as part of the DCC SAFER work using the results of the flood risk assessment work to identify damage functions and appropriate responses. A review of the location of command centres and equipment depots referred to in the ERP and MEP should be undertaken to ensure that none lie within an area at risk. Utility supplies to command centres should also be reviewed and amended if necessary to ensure continuity of supply in the event of flooding.

14.2.6 Strategic Recommendations

From the review of the documents listed in Chapter 7, and the consideration of the components of a flood forecasting and warnings system, the following strategic recommendations can be made:

1 Identify who, where and when requires a warning and determine a system performance specification.

2 Identify the current or justifiable flood forecasting system for Dublin City Council area and the linkages that need to be made to translate forecasts to warnings.
3 Determine the most appropriate warning methodologies to meet the performance specification.
4 Identify the investment programme to procure and implement the chosen methodologies.
5 Review the existing operational arrangements within professional organisations and the emergency services to determine the impact of flood risk upon their operations.

14.2.7 Work required

In order to establish an effective flood forecasting and warning system, the following elements of work are required:

1. Identify locations that require flood-warning messages. This is partially accomplished by the Dublin Coastal Flooding Protection Project. Establishing the system for the rivers does not form part of this commission.
2. Identify "target audience" from each of four groups for each location.
3. Identify through liaison and consultation the most effective warning messages together with the target lead time that provides sufficient time to take effective action.
4. Review flood forecasting system and "trigger" and "threshold" levels to link to the flood warning system.
5. Identify appropriate technology to deliver the required number of messages to the identified recipients within the stated lead time. Identify costs and timescale to procure commission and implement the technology.
6. Identify existing operational procedures and future requirements to operate the flood warning system and carry out trials. The system to have the ability to record warnings issued and cancelled so that "real time" summaries can be produced as well as post event reports.
7. Establish routine (annual) test programme.
8. Implement system, record and review performance.

15 FLOOD HAZARD IDENTIFICATION AND FLOOD RISK DETERMINATION

15.1 Introduction

In this section a detailed assessment of the flood risk around the project area is considered. The work on which this section has been built, draws from much of the information collected earlier in the project; from the reviews undertaken in relation to international best practice; from the results of the probabilistic analysis; from the projections of the effects of climate change and from the numerical modelling. All these earlier stages, which have been set out in detail in earlier chapters, were necessary before a full and detailed evaluation of the flood risk could be undertaken. Obviously as a result of the February 2002 event it was known that there was a coastal flooding risk. However, this risk needed to be confirmed, verified and quantified in order to set appropriate standards of protection for the region in the future and further to justify the need for risk reduction options, whether they be physical flood alleviation options or risk management options through the use of forecast and warning systems.

In the section that follows, a stepped approach through our assessment of the flood risk is presented. This starts with a consideration of the coastal zone topography and identification of possible flood compartments, and continues with a brief overview of drainage issues across the region. It concludes with an explanation of our methodology for assessing the flood risk, through setting appropriate design criteria and how that criteria were used to quantify the risk. It concludes with a brief overview of impact likely to arise through that risk and presents details of those areas at risk and the level of that risk.

15.2 Flood Compartments

15.2.1 Methodology and Information Available

An initial part of the flood risk assessment involved the identification of low lying coastal topography and hence possible flood risk areas or flood compartments. This was undertaken by identifying all low lying regions which could be at risk of coastal flooding through consideration of topography alone. The low lying topographic areas or compartments are identified as the maximum possible area at risk, using extreme water levels identified through the probabilistic analysis and general topographic information across the project area. They do not necessarily represent the areas that will be flooded for any given scenario, as many of them may be defended to some level of protection already and others may only become accessible through complex flood routes. As such not all of the identified low lying compartments might actually flood for a particular event, indeed different parts of a compartment might be at risk for different events depending on any number of conditions that might be in place at that time.

However to identify the possible low lying compartments it is reasonable to chose a given extreme water level and identify all regions which fall below this level. This is considered an acceptable initial approximation when considering the vast sink of water available through the sea for coastal flooding, and particularly when the compartments are not very large or the inundation of flood water is not restricted through a fixed small breach width.

For this project a 200 year extreme water level with an allowance for sea level rise to the end of the century was taken as the criteria to identify possible coastal zone low lying compartments.

This criteria was chosen based of the fact that it is an indicative level of protection against tidal flooding which is used in a number of other countries, including the UK, and further being of a magnitude greater than the recent February 2002 event, (1 in 67 years), it was considered an appropriate starting level for the assessment. The nature of the topography around the majority of the study area also reflected this choice in that, should protection against a more extreme event be considered more appropriate at a later stage, flooding whilst deeper and possibly longer, but would not be much more extensive in nature.

From the results of the probabilistic analysis a 200 year extreme water level now has been considered at a level of 3.13mODM for the Dublin Port tide gauge. Through our assessment of climate change, which has been set out in Chapter 8, an annual average mean sea level rise of 4.15mm/year has been considered as appropriate for use. Taking a base year for these estimates as 2002, this provides a 200 year extreme tide level in 2100, of **3.54**mODM. Whilst the 200 year tide level around the project area might vary slightly by taking the sea level rise allowance to the end of the century, it is considered that this estimate of 3.54mODM, is a sufficiently conservative value against which low lying and potential compartments can be assessed. To identify these potential compartments, the area which would be at risk if unlimited volumes of water were to inundate the hinterland at this level of 3.54mODM, has been considered. Therefore all ground topography, around the coastline or joined to the coastline through tidal reaches of rivers or canals, at or below this level has been identified and possible flood compartments formed. In the tidal reaches of the rivers, preliminary results from the numerical modelling of those reaches indicated that possible back water profiles, particularly for the more extreme combinations of high fluvial discharge, might give rise to extreme water levels in the river along the upper reaches in excess of this 3.54mODM level. This is primarily due to rising river bed profiles and the back water effect which is more pronounced when fluvial flows are extreme. Therefore topography and possible compartment boundaries at the upper reaches of the tidal river lengths were extended out to ground levels as much as 5.5mODM.

The compartments have been split up where regions of natural high ground or other man made structures such as railway embankments or roads, dictate boundaries between the continuous low lying areas. In such cases the sub compartments are linked by the fact that in the first case flood levels might exceed the levels along the ridge of high ground and therefore flood water could pass between sub-compartments. In the second case, often road underpasses through the railway embankment exist and these also provide a connection or flood route between the compartments. Indeed flooding by this type of mechanism was very evident during the February 2002 event, particularly in the East Wall region, where water escaped from the Royal Canal into the adjacent CIE property and then from there through a number of routes, which included both pedestrian access gates and road underpasses, see description of flood areas for that area presented in Chapter 3.

The topography across the project area was assessed through use of a digital terrain model (DTM) which was provided by the water division of DCC and which had been complied for use in future water supply projects across the greater Dublin Metropolitan area. The information used to create the initial DTM included,

- Manhole cover levels
- Road spot heights taken from OS plans
- LiDAR, aerial survey, information obtained for a number of other flood alleviation projects.
- Verification survey work undertaken as part of the DTM generation study

This DTM was then enhanced and developed through the inclusion of various other sources of level data collected as part of this project. This included,

- Topographic and bathymetric information collected as part of the project.
- Manhole cover data obtained from FCC.
- Topographic data for the Liffey quays between East Link Toll Bridge and Matt Talbot Bridge.
- PGM information supplied by Dublin Port Company (DPC) and the Tolka flood study.
- Mudflat survey information obtained from DPC.
- Bathymetric information around Bull Island supplied by Dublin Bay project.
- Bathymetric information in Baldoyle Estuary supplied by FCC.

15.2.2 Identification of Low Lying Topographic Compartments

Based on the above criteria and using the DTM, a list of low lying compartments with topography below specified levels were identified as the first step in identifying the flood hazards through out the study area. The location and extent of these compartments are presented in Appendix O, Coastal Zone Topography Maps 1 to 8.

These figures do not show flood hazard or risk, but simply indicate all low lying coastal topography which theoretically has the potential to be at risk of flooding based on a 200 year event, taking 100 years of sea level rise into account and assuming no limitation in flooding volumes.

Table 15.1 - Summary of Low Lying Coastal Zone Compartments

Compartment			Location	Possible Flooding Mechanism
Name	Sub-cell	Map_1		
Portmarnock	1	1	East of river under Portmarnock Bridge	Weiring over defences & through gaps.
	2	1	West of river under Portmarnock Bridge	Weiring over & through defences & through gaps
Baldoyle	1a	1 or 2	North of Mayne River	Enter sub – compartment by 1. Wave & weiring over coast road 2. Through Mayne Outfall (only if flap not operational)

Compartment			Location	Possible Flooding Mechanism
Name	Sub-cell	Map$_1$		
	1b	1 or 2	South of Mayne River to Baldoyle Town	Enter sub – compartment by 1) Through Mayne Outfall (only if flap not operational) 2) Wave & weiring over coast road.
	2	1 or 2	Baldoyle Town Fronatge to Dart Line	Weiring through gaps in defences but mainly wave overtopping defences.
	3	2	South of Dart Line	Isolated low lying area would require very extreme event to penetrate.
Sutton	1	2 or 3	East of Entrance to Baldoyle Estuary	Mainly wave overtopping
	2	3	South of Sutton Cross	Weiring over defences and wave overtopping.
Howth Harbour	1	3	West Pier of Howth Harbour	Wave overtopping onto reclaimed land area.
Clontarf	1	4	Alfie Byrne Road to Haddon Road	Weiring and wave overtopping
	2	4	Haddon Road to West of Vernon Avenue	Weiring but mainly wave overtopping
	3	4	South extents of Vernon Avenue	Weiring but mainly wave overtopping
	4	4	Clontarf Yacht Club	Weiring but mainly wave overtopping
	5	4	West of Bull Wall	Weiring but mainly wave overtopping
Fairview	1	4/5	Fairview Park and Road	Over spill from Clontarf Compartments via Dart Underpass.
Port	1	4	North Dublin Port	Wave overtopping
East Wall	1	5	Area bounded by CIE & East Wall Road	Weiring over boundary walls of CIE land. Original source Royal Canal
	2	5	Ossary Road Area	Over or through failed boundary CIE walls or down railway line. Original source Royal Canal
	3	5	East of Royal Canal	Overtopping banks of Royal Canal mainly &/or River Liffey
Tolka	1	5	Area between River Tolka and Royal Canal, north of Dart Line	Over topping banks of Tolka. Many flood routes within compartment via underpasses etc. Possible link to East Wall 2 via railway line (would require very extreme event).

Compartment			Location	Possible Flooding Mechanism
Name	Sub-cell	Map1		
Connolly	1	5	Area between Royal Canal, River Liffey and elevated DART line	Overtopping of Royal Canal banks mainly &/or River Liffey.
	2	5	Northern side of Elevated DART line & Butt Bridge to O'Connell Bridge	Overtopping of Liffey Quays and underpass links with Connolly 1.
Liffey Left Bank	1	5 or 6	O'Connell Bridge to Fr Mathews Bridge	Overtopping quay walls & surcharge gully systems.
	2	6	Liam Mellows Bridge to Father Mathew Bridge	Overtopping quay walls & surcharge gully systems.
	3	6	Rory O'More Bridge to Liam Mellows Bridge	Overtopping quay walls & surcharge gully systems.
	4	6	Frank Sherwin Bridge to Rory O'More Bridge	Overtopping quay walls & surcharge gully systems.
	5	6	Upstream of Sean Heuston Bridge	Overtopping quay walls
	6	6	Sarah Bridge to Islandbridge Weir	Overtopping river bank.
Liffey Right Bank	1	5 or 6	O'Connell Bridge to Grattan Bridge	Overtopping quay walls & surcharge gully systems.
	2	6	Grattan Bridge to O'Donovan Rossa	Overtopping quay walls & surcharge gully systems.
	3	6	O'Donovan Rossa to Liam Mellows Bridge	Overtopping quay walls & surcharge gully systems.
	4	6	Rory O'More Bridge to Liam Mellowes Bridge	Overtopping quay walls & surcharge gully systems.
	5	6	Frank Sherwin Bridge to Rory O'More Bridge	Overtopping quay walls & surcharge gully systems.
	6	6	Railway Bridge to Sarah Bridge	Overtopping River Banks
	7	6	Sarah Bridge to Islandbridge Weir	Overtopping River Banks
Pearse	1	5 or 7	South Side of Liffey from Butt Bridge to Grand Canal Basin	Overtopping quay walls & surcharge gully systems.
Dodder	1	7 or 8	Area bounded by River Liffey, Grand Canal Dock and DART line	Overtopping river banks
	2	7 or 8	Area south west of DART line	Underpass connection from Dodder 1 & Overtopping of Upper river banks.
	3	7 or 8	South of DART line, east of Ballsbridge	Underpass connection from Sandymount 1 & Overtopping of Upper river banks.
Ringsend	1	7 or 8	East Link Toll Bridge	Overtopping river bank

Compartment			Location	Possible Flooding Mechanism
Name	**Sub-cell**	**Map$_1$**		
Sandymount	1	8	Area East of River Dodder from East Link Toll Bridge to Merrion Gates	Overtopping banks of River Dodder & Wave Overtopping along Sandymount.
	2	8	South West of Dart Line	Connection to Sandymount 1 via rail crossing
	3	8	South of Merrion Gates	Connection to Sandymount 1 via Merrion Gates crossing 7 wave overtopping.

15.3 Drainage Considerations

15.3.1 General

Drainage issues across the Greater Dublin area are currently being extensively investigated as part of the Greater Dublin Strategic Drainage Study (GDSDS). Therefore the issues in respect of flooding through back up of drainage networks has not been considered in detail as part of this study. However, it is necessary to consider some aspects of drainage in so far as identification of issues that might cause concern and will require clarification by the GDSDS or investigation works. Such issues include outfalls into the tidal reaches of the river or around the coastline, in so far as whether extreme tidal conditions could cause surge charging through flapped or unflapped outfalls. This project has not tried to quantify flooding through such sources but has carried out a general overview of the drainage and inspection of outfalls to identify those that could result in possible problems and might require further investigation.

The section commences with an initial overview of the drainage system in Dublin and then presents the findings of a brief outfall inspection survey along the River Liffey and Dodder. The river Tolka was not considered in view of the extensive study already undertaken for that river and the small extent of the river with the project area.

The overview description of the Dublin drainage system should be read in conjunction with a drainage plan for the Dublin Area, which is presented in Appendix O.

15.3.2 Overview of Dublin Drainage System

The North City Centre catchment, which is roughly bounded between the North River Liffey Quays and the Royal Canal is drained into the North City Interceptor Sewer which runs along the North Liffey quays. Approximately halfway between the O'Connell Bridge and the Butt Bridge the pipe line crosses beneath the Liffey to Burgh Quay on the south side of the river. The North City Interceptor Sewer then runs eastwards to the Main Lift Pumping Station at Ringsend, and then onto the Ringsend Treatment Works.

The South City Centre catchment, which is roughly bounded between the Grand Canal Tunnel and the South River Liffey quays drains into the South City Interceptor Sewer. This sewer then runs down to the South River Liffey quays. The pipe then runs eastwards and connects with the North City Interceptor Sewer pipe mention above.

The extreme Southern catchments (Dodder Valley, Clondalkin and Lucan, etc) either connect to the Grand Canal Sewer, which runs alongside the Grand Canal or go directly to the Main Lift Pumping Station via the South City Interceptor Sewer. These combined sewers run in an easterly direction, alongside the Grand Canal and the southern side of the River Liffey, respectively. Both sewers then link into the Main Lift Pumping Station, before continuing onto the Ringsend Treatment Works.

The Blanchardstown catchment collects into a combined sewer, which connects to the western end of the Grand Canal sewerage system on Davitt Road near Sur Bridge. Both the Clondalkin combined collector sewer and the Blanchardstown collector sewer are not owned by Dublin City Council. As such gauges at the connection points to the Grand Canal monitor the amount of sewerage that enters the system from the Blanchardstown and Clondalkin catchments for the purpose of regulation by Dublin City Council.

The area around East Road is low lying, and therefore there is a need for the use of three pumping stations at Castleforbes, Mayor Street and East Road in order to efficiently drain and discharge of sewerage from this area. Castleforbes Pumping Station (CPS) and Mayor Street Pumping Station (MSPS) both collect combined sewerage from small catchment areas. Castleforbes pumps directly into the East Road Pumping Station (ERPS), which also collects the combined sewerage from the rest of the catchment. However Mayor Street pumps to the North City Interceptor Sewer. The ERPS pumps all the collective combined sewerage through a tunnel under the River Liffey, which runs directly beneath the East Link Toll Bridge, and connects directly to the Main Lift Pumping Station.

The outer North Dublin drainage catchment (North Dublin, Clontarf, Vernon Avenue, etc) collects into the combined North Dublin Drainage System Trunk Sewer, which runs eastwards towards Sutton. It connects with the combined sewer, running south from Dublin Airport, at the Kilbarrack Pumping Station, and is then pumped to the Baldoyle Pumping Station, where it connects with the combined sewerage that has been pumped south from Portmarnock. Until more recently all the combined sewerage that was collected at this point was then pumped eastwards through a series of pumping stations, until it was final discharges out of the Howth Outfall.

However, a new pumping station has recently been constructed at Sutton as part of the Dublin Bay Pipeline Project, and this was opened on the 8th September 2003. This new pumping station now takes all the combined sewerage from Howth and the North Dublin Drainage, etc away from the Howth Outfall and pumps it through a submerged pipeline under the Dublin Bay to the Ringsend Treatment Works.

In general the surface water and foul sewerage, within the canal areas, are combined, however outside the canals they are more separate sewer systems. In the locations where they are not combined, the surface water runs directly to the nearest watercourse, be that a river or coastal outfall. The foul runs to either the nearest combined pumping station or combined sewer run.

Due to navigation issues along the lower reaches of the River Liffey from East Link Toll Bridge to Butt Bridge, protrusions from the edge of the quay walls are not permitted. This means that many of the flap valves are either recessed back into the quay walls, or are flapped at the nearest manhole. In most cases there are flaps within the connecting pipes further back up the system. However, if the nearest manhole is too far back up the system, a secondary and tertiary flap valve system has been created within the pipes, prior to the surface/storm water leaving the system.

The East Wall Industrial Estate has its own sewerage system and pumping station, which works independently from the Dublin Drainage System. The foul sewage is collected from the catchment, and flows to the East Road Pumping Station, which connects into the Dublin combined sewerage system. The surface water for this catchment collects and drains directly to the Dublin Bay.

The sewage from Dun Laoghaire flows directly to the Ringsend Treatment Works, through a submerged pipeline that runs from beneath the Dun Laoghaire Nautical College.

The Ailesbury Pumping Station, off St Albans Road, Sandymount Strand, pumps surface and storm water directly back out into the Dublin bay. This area is low lying and it has been reported that it has the potential to tide lock and flood. Although during the 1st February 2002 event this did not happen.

The developers at the Royal Canal Spencer Dock are installing a new pumping station. This pumping station will be used to pump sewerage from the new development planned in this area and will be used as a strategic link between East Road Pumping Station and the new Spencer Dock Pumping station. The Mayor Street Pumping station will also connecting into it. It is proposed to construct a new service tunnel under the River Liffey and connect all the East Wall and Royal Canal sewerage through it using a new new rising main before connecting to the Main Lift Pumping Station, on the way to Ringsend Treatment Works.

The Ringsend Treatment works collects all the combined sewerage from all the above catchments. There is an effluent outfall just eastwards of the treatment works, along the Great South Wall.

At the treatment works they are able to detect and inform the Drainage Department within Dublin City Council whether there has been an increase in saltwater in the sewerage collected. Should this be the case, it may mean that a flap valve on one or more of the outfalls into the Liffey, or Dublin Bay etc, may be jammed open, or defective. Therefore the tide gate crew can investigate and fix the problem.

15.3.3 Inspection of Outfalls Along Tidal Rivers

As part of the flood risk assessment and to help identify possible flood paths through to particularly low lying flood compartments where drainage might be a significant issue, a brief inspection of the drainage plans, followed up by an inspection of the river outfalls within the study area, has been undertaken. This assessment has been aimed at inspecting the outfalls and establishing whether they are currently in use or not and whether they are flapped or not. It does not attempt to answer these questions in its entirety or quantify flood mechanisms as a result. Moreover, it hopefully provides a history of the existing outfalls, clarifies those that are readily comparable with the existing drainage plans and those that are not and as such may require further examination. In particular one important aspect to consider was the localised road gullies that discharge into the rivers and where the quay and ground levels are low with respect to extreme water levels. This combination of occurrences could provide a possible flood route onto the quay as occurred in February 2002 along Victoria and Wolfe Tone Quays.

Using the drainage plans a desktop study was undertaken, marking along the banks of each of the rivers where the active drainage outfalls were located and the drainage systems they were attached to.

Following the desktop study a site inspection was made to each of the rivers to catalogue the outfalls along each of the banks or quays. These were marked on the same plan as the desktop study to compare all outfalls noted with those considered to be still in use as identified by the drainage network plans.

The information gathered from the site inspection (Appendix O) was tabulated to include the location of the outfall, the condition of the outfall, whether the outfall was flapped at its point of discharge. Other information recorded included the approximate invert level (whether in or near the tidal range), its designation on the drainage plans if applicable, and if necessary a new designation, as part of the DCFPP.

It was decided prior to the site inspection to designate the existing outfalls along the Rivers Dodder and Liffey, i.e. those that were already on the DCC plans, by the following system:

1st digit	D = Dodder L=Liffey
2nd & 3rd digits	RB = right bank looking downstream (shown on drainage records) LB = left bank looking downstream (shown on drainage records) RN = right bank looking downstream (not shown on drainage records) LN = left bank looking downstream (not shown on drainage records)
4th & 5th digits	position in sequence from the upstream end (note that there are generally four sequences corresponding to RB;LB;RN & LN.

The results and photographs of all the outfalls, in Appendix O, are shown collectively in consecutive order, from upstream to downstream, to aid the location of specific outfalls.

Looking at the photographs taken of each of the outfalls it is clear that many of the outfalls do not appear to be flapped. However it may be the case that they are flapped further up the system, perhaps at the nearest manhole. It is considered that further investigations into this may be necessary, although it is likely that many of the uses and nature of the outfalls can be clarified by the drainage department. Where outfalls are not flapped this should be clarified.

Future work may include a more thorough investigation of the flap valves that may be required for the existing outfalls that are already in use, with those that have been decommissioned perhaps being fully blocked up. It is considered that the flooding that occurred on Wolfe Tone and Victoria Quays resulted from water escaping through a drainage system that may not have had satisfactory flap valves for flood protection. We understand, through discussions with the Drainage Division, that this problem has since been rectified at this location.

Other locations on the River Liffey where the quay levels are lower than possible extreme tide levels and hence possible surge charge through gully outfalls might occur include, Merchants Quay and Inns Quay, between Father Mathew and O'Donovan Rossa Bridges. The quay levels on these quays are approximately 3.1mODM. On Merchants Quay there are three outfalls, only one (LRB8) of which is noted on the DCC drainage plans, and appears to be a gully or over flow into the South City Interceptor Sewer. Similarly on Inns Quay there are six outfalls, only two of which are noted on the DCC drainage plans. They appear to be linked to localised drainage systems (LLB9). One (LLB8) is located directly downstream of Father Mathew Bridge, which over flows into a combined sewer network.

Wood Quay and Ormond Quay Upper, between O'Donovan Rossa and Grattan Bridges also have low quay levels. The quay levels on these quays are approximately 3mODM. On Wood Quay there are three outfalls, only two of which are noted on the DCC drainage plans. One(LRB9) which appears to be a road gully, overflows into a combined sewer network, located on the downstream side of O'Donovan Rossa Bridge. The second is the outfall for a separate surface water drainage network, which runs along Fishamble Street, but was not seen during the site inspection. Anecdotal evidence suggests that during the February 2002 flood water was seen backing up the drainage pipes onto the road behind. Whilst this might not have been as severe as the flooding on Wolfe Tone and Victoria Quays, there is a potential flood risk hazard along these quays that will need addressing.

Similarly on Ormond Quay Upper there are six outfalls, only two of which are noted on the DCC drainage plans. Both of these drainage outfalls are categorised as over flow outfalls from combined sewers and both link into the North City Interceptor Sewer.

Between the Millennium Bridge and Matt Talbot Memorial Bridge the quay levels are between 2.7mand 3.3mODM. At present the quay walls would prevent the tidal flood risk, as the majority of the quay wall crest levels are above 4mODM.

However there are a significant number of outfalls along both the quays. On the right quay there are some 13 outfalls and 17 on the left quay. Only 3 on the right quay are shown on the DCC drainage plans. Whereas only 7 on the left quay are shown on the DCC drainage plans. The majority of the outfalls shown link back into localised combined sewerage networks or appear to be road gullies.

Whilst the majority of these drainage outfalls do not appear to be flapped at the discharge end, further investigation of the outfalls along the River Liffey Quays is recommended to establish the validity of the outfalls not noted on the DCC drainage plans. Many of these outfalls may already be blocked or flapped at the nearest manhole. However, once this situation has been clarified through further investigation, the potential for tidal flood risk posed by the unflapped outfalls still remains.

The River Dodder has some 10 outfalls on its left bank (looking downstream), between Ballsbridge Weir and the River Liffey Confluence, and 8 outfalls on its right bank. The DCC drainage plans indicate that 4 of the outfalls on the left bank and 3 on the right bank exist and are still in operation.

The majority of the outfalls on the left bank link into the Rathmines and Pembroke No. 1 trunk sewer, through overflow pipes, or via a surface water network that ultimately connects to the trunk sewer. All but one of the outfalls appears, through visual inspection, to be unflapped. The flapped outfall (DLN3), has a large metal outfall, that looks to be in use on an infrequent basis.

For the outfalls on the right bank two of the three know outfalls (DRB1 and DRB2) are outfalls for surface water drainage networks. DRB1 is a short section of surface water drain, which looks to have been recently installed, with an outfall pipe, as part of the new quay wall construction. However DRB2 is a surface water pumping outfall, which is possibly part of the network from the Shrewsbury Pumping Station. The third outfall, which is noted on the DCC drainage plans is DRB3, this is a storm overflow pipe, which connects into the Rathmines and Pembroke No.2 Combined sewer, both of which are flapped.

The remaining outfalls, whether noted on the DCC drainage plans or newly inspected on both banks do not appear to be flapped. If these are not flapped further back up the drainage network, perhaps at the first manhole, they could pose a flood risk hazard, not necessarily directly behind the quays themselves, but further up the drainage network.

Therefore two recommendations from this outfall inspection can conclude that a further more detailed inspection of the drainage outfalls is required, along with a possible updating of the DCC drainage plans as necessary. In particular attention should initially be given to those outfalls located along low quay levels, i.e. less than3.25mODM in elevation. In addition it may be prudent to permanently block up the outfalls that are not in use.

15.3.4 Issues Relevant to DCFPP

Some drainage issues may affect the way that the coast and the rivers within the project area are protected from flooding. Many of these issues can be alleviated through good housekeeping (further investigation and maintenance and possible replacement of failing infrastructure). From the river outfall survey it was clear that many of the pipes were not flapped at the point of outfall.

However it is recognised that many of these unflapped outfalls may either be defunct, or be flapped further up the pipe at the manhole. These issues will need to be investigated in more detail, as they may be important during an extreme flood event.

In addition, it is recognised that significant work has been undertaken to assess and map the current working network of drainage within the Greater Dublin area. However, during the February 2002 flood event much of the system became dangerously overloaded, almost to the point of a complete system failure due to the surcharge of the system through inundation of tidal flood waters. However prevention of extensive flooding of low lying area such at the Stella Gardens and East Wall by improved tidal defences, will benefit the drainage system in the future. Such improvement would mean that the system would not have to cope with the removal of large quantities of saline water entering from the sea by an overland flood path, only to be removed by the drainage system.

15.4 Detailed Assessment of Flood Hazard Areas

15.4.1 Methodology

The assessment of flood risk, whether from coastal or inter tidal flooding, is made on the basis of comparisons between extreme tide levels, wave climate, fluvial discharges or combinations of these, with appropriate allowance for the effects of climate change, and the existing defences.

Any assessment of flood risk must consider and address a number of important questions:

a) Where has flooding previously occurred?
b) What is the standard of protection of the existing defences?
c) What is an appropriate standard of protection?
d) What are the dominant mechanisms?
e) What flood compartments are affected? and
f) What is the extent of flooding for any given flooding event?

Addressing these questions is an iterative process often requiring disparate data sets to be merged into a coherent whole, from which the overall picture emerges.

Therefore before any detailed assessment of the flood risk around the coastline could be undertaken, a significant amount of information had to be collected and other elements of the study completed to provide the information necessary to undertake a comprehensive analysis. This included the completion of a detailed topographic survey across the study area to obtain level information in relation to existing ground and defence levels. Details of this survey work and the information collected is presented in more detail in Chapter 4, as well as information in relation to levels of existing defences and the surrounding ground. Another important element to consider was the condition of the existing defences. This has been evaluated through a detailed coastal inspection survey and the information is presented in database format, see Chapter 5. Information from this database, which highlights where existing defences are in a poor condition or where remedial works have been recommended, has been extracted and is also presented to supplement the flood risk associated with the level of the defences. For example, a wall may be of sufficient height to be defined as having a standard of protection of 1 in 100 years, however its condition might be such that if a less extreme event were to occur it would fail and consequently it would actually offer a much reduced standard of protection. Therefore, when assessing the flood risk it is important to consider both of these aspects for any defence.

In addition to this, information on various extreme tide level scenarios, and how they vary around the coastline and within the tidal reaches of the rivers and canals, was necessary for comparison against the level information obtained through the topographic survey work. This information was obtained as output from a number of earlier elements of work, which included the sea level rise review and probabilistic analysis of tide records at Dublin Port, see Chapters 8 and 9, the numerical modelling of the tidal and wave conditions around the coastline, see Chapters 11 and 12 and also the hydraulic modelling of the rivers, see Chapter 13. Through the probabilistic analysis information in relation to the severity of extreme tidal events and their associated return period was provided. In addition information in relation to the correlation between fluvial discharges and surge conditions was investigated and also the effect of fluvial discharges on the water levels at the downstream end of the rivers near the port.

The numerical modelling of the tide and wave climate provided important information on and an understanding of how the tide varies around the coastline and how the waves are affected as they propagate toward the shore. This information is extremely important as the flood risk can vary form one location to the next around the study area, simply by virtue of the fact that tide levels are higher for example north of Howth Head, than those at Dublin Port. In addition surge and wave conditions can vary significantly from one side of Dublin Bay to the next depending on the direction and strength of the wind.

Therefore in assessing the flood risk across the study area a number of parameters need to be considered and each, in combination or individually, can result in flooding and therefore have an impact on flood risk. Each can also vary in magnitude around the study area depending on the given condition of any day/event. Whilst it is not possible to assess the flood risk for every possible combination and permutation of the parameters that could exist, those which are likely to provide the worst combination of events have been considered and the flood risk evaluated on that basis. For example a slight change in the wind direction off this critically considered direction, may result in little or no overtopping occurring for a particular event at a particular location, whilst a similar event with wind blowing from that critical direction might cause that location to flood. In each case the parameters likely to affect the area in the worst way have been considered. These parameters include,

- Tide levels
- Wave climate
- Wind speed and direction
- River discharges

Rainfall and drainage capacity of existing catchments was not considered in combination with any of the above parameters.

When considering any given scenario, the correlation of one parameter in relation to the next should be carefully considered and some level of joint probability concept considered, to ensure that appropriate combinations of each parameter are considered. This should equally ensure that while events which are clearly not independent of each other are combined in some manner, that they are not combined in such a way as to produce an unrealistically onerous event which could greatly over estimate the flood risk of any area. Each of these parameters and how they have been correlated for use in the DCFPP assessment of flood risk is described in more detail in the following section on risk criteria.

15.4.2 Definition of Hazard Criteria and Design Parameters

Risk Criteria

Introduction

As part of the study it was necessary to set some initial risk criteria which we could adopt as a basis for the assessment and which could be reviewed as the process evolved and the level of risk and impact of that risk became clearer. These initial criteria were set through consideration of a number of issues, which included,

- A review of international best practice
- A consideration of the extent of flooding on 1st February 2002
- Results of the probabilistic analysis in respect of extreme tide level values.

Review of International Best Practise

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

Table 15.2 – 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 contains areas with very high population density and high economic activity. However, more recently in the Netherlands there has been a step away from the more traditional assessment methodology of providing defences to a given return period event and more emphasis is being placed on the risk, being defined as probability times damage. However, it is believed that this method is proving to be too onerous and insufficiently transparent. There are wide spread calls, including from within government, to return to the historic and more traditional method of setting risk levels and overall standard of protection.

Recent guidance set out in the policy documents developed through the 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.

February 2002 Event

The February 2002 event caused extensive flooding around the Dublin City and Fingal County coastlines. Descriptions of the extent of this flooding are presented in Chapter 3, with maps given in Appendix C. Based on this event, it can be seen that there are significant numbers of properties which are at risk from coastal flooding and which would certainly fall into the UK land use band “A”. Some of these areas are also very low lying indeed and were flooded through complex flood paths. Close examination of the low lying compartment maps presented in Appendix O, also demonstrates that there are considerable areas of low lying land, whilst exempt from flooding during the February

2002 event, could be at risk from a more extreme event or from continued deterioration of existing defences.

Probabilistic Analysis

Results of the probabilistic analysis and in particular the joint probability analysis of extreme tides and surges, suggests that the 1st February 2002 event, based on still water level alone, was in the order of a 67 year event (annual probability of 0.015). This has dropped considerably from initial thoughts that it was an event in excess of 200 years.

More recently the event of the 28th October 2004, which also caused some but less flooding than the 2002 event, was noted to be the third highest tide (80 years) on record at the Dublin Port tide gauge. This event based on the results of the probabilistic analysis and still water level alone could be considered to be in the region of a 1 in 10 year event (annual probability of 0.1).

Therefore it is quite feasible that larger events could occur with more extreme consequences for the region, if the existing levels of protection, presented in Section 15.4.4, are maintained.

In light of this an initial view was taken that an appropriate indicative level of protection for Dublin and the surrounding area of 1 in 200 years (annual probability of 0.005) should be considered.

Design Parameters

In the following sections a brief summary of the design parameter which have been used in the overall assessment of flood risk with the DCFPP, and later in the development of alleviation options, is set out. This included a consideration of how particular parameters should be combined in a joint assessment of flood risk.

Tidal Levels

The issues of astronomical and extreme tidal events and their statistical probability has been considered in some detail in Chapter 9 and therefore is not discussed further here. However for clarity the extreme tide levels consider for use in the analysis are summarised below.

Table 15.3 - Summary of Extreme Water levels for Dublin Port Tide Gauge.

Return Period (years)	**Extreme High Water Level Dublin Port Lighthouse**	
	Extreme Water Level (mLAT)	Extreme Water Level (mODM)
2	4.86	2.35
5	4.99	2.48
10	5.12	2.61
20	5.26	2.75
50	5.42	2.91
100	5.54	3.03

200	5.64	3.13
500	5.75	3.24
1000	5.82	3.31

In order to determine how these tide levels might vary around the study area, the results of the FINEL tidal model, described in Chapter 11, were used. The results provided an astronomical relationship between the Dublin Port tide gauge, for which the extreme tide levels presented above relate, and a number of locations around the project area. These locations are shown in Figure 11.20 in Appendix K. This relationship together with an evaluation of how the surge might behave around the project area was used to estimate equivalent extreme water levels at other locations for assessment of risk.

Wave Climate

Offshore wave climate data, which was obtained from the UK Met Office for a previous study just south of Dublin Bay, was used to produce offshore extreme wave climate estimated for use in evaluation of nearshore wave climate and flood risk. The data obtained was broken down by wave height and direction and as such extreme wave heights were estimated for each direction. The extreme offshore wave climate used is summarised in the table below.

Table 15.4 - Extreme Offshore Wave Climate

Wave Direction (°N)	Wave Height & Period	Extreme Event (years)			
		1	10	50	100
30	Hs (m)	**3.58**	**4.93**	**5.87**	**6.28**
	Tz (s)	6.9	7.8	8.5	8.8
60	Hs (m)	**2.99**	**4.04**	**4.78**	**5.09**
	Tz (s)	6.1	7.0	7.7	7.9
90	Hs (m)	**2.74**	**3.87**	**4.65**	**4.99**
	Tz (s)	5.8	6.9	7.6	7.8
120	Hs (m)	**4.3**	**5.9**	**7.01**	**7.49**
	Tz (s)	7.3	8.5	9.3	9.6
150	Hs (m)	**5.15**	**6.74**	**7.86**	**8.34**
	Tz (s)	7.8	9.1	9.8	10.1
180	Hs (m)	**4.71**	**6.01**	**6.91**	**7.3**
	Tz (s)	7.6	8.6	9.2	9.5

Hs is significant wave height.

Tz is mean wave period, where $Tz = 11\sqrt{\frac{Hs}{g}}$

As described in Chapter 12, the offshore wave climate was transferred to the nearshore using the wave transformation model SWAN. This was undertaken for a complete range of scenarios, i.e. different wave heights, directions and periods on a range of water levels. The primary function of the numerous combinations being to set up a transfer function matrix which could be used to transfer offshore wave predictions to the nearshore for us in development and operation of a flood forecasting system. However, by similar means the results of this large number of runs have been used to transfer offshore wave climate for any given direction to the nearshore for use in assessing flood risk at any given location. The nearshore wave climate being obtained from one of the predetermined output points at or near the location at risk.

In addition to extreme offshore wave climate and its behaviour as it propagates to the coastline and into Dublin Bay, a number of locations exist at which the effect of these offshore waves is greatly diminished or will not exist. Such areas include the inner harbour along Clontarf, the tidal areas behind Bull Island and the inner Baldoyle Estuary. In such locations, wave climate will largely be governed by the locally available fetch, wind speed and wind direction. For these locations, wind generated waves have been determined using extreme wind speeds presented in the following section and where applicable have been used in the flood risk assessment evaluation (see also description in chapter 12, section 12.4.2). A summary of the wave climate generated for one of these locations along the Clontarf frontage (Amazon PROFILE 9, see Figure 12.12), is presented in Appendix L.

Wind Conditions

Wind speed data was obtained from Met Eireann for Dublin Port for the period January 1960 to December 2003. The data was supplied in scatter diagram format of observations by wind speed and direction. An extreme value analysis was undertaken on the data per direction to provide estimates of the extreme mean hourly wind speed which could be used in determining extreme wave climate or for use in determining appropriate wind speeds for using in wave propagation transfer. A summary of the extreme wind climate is presented in Table 15.5 below.

Table 15.5 - Summary of Extreme Wind Speeds.

Wind Direction (°N)	Mean Hourly Wave Speed	Extreme Event (years)			
		1	10	50	100
90	Knots	35.4	46.1	53.6	56.8
	m/s	18.2	23.7	27.6	29.2
120	Knots	35.1	45.5	52.7	55.9
	m/s	18.0	23.4	27.1	28.8
150	Knots	35.4	45.1	51.8	54.8
	m/s	18.2	23.2	26.7	28.2
180	Knots	34.5	45.5	53.1	56.4
	m/s	17.8	23.4	27.3	29.0
210	Knots	38.3	48.6	55.8	58.9
	m/s	19.7	25.0	28.7	30.3
240	Knots	40.1	49.8	56.6	59.5
	m/s	20.6	25.6	29.1	30.6
270	Knots	39.5	49.4	56.4	59.4
	m/s	20.3	25.4	29.0	30.6

River Discharges

Chapter 13 of the report covers in some detail the review of hydrological data undertaken for the study, the extreme fluvial discharges considered for use in the study and the methodology and results of the modelling of the tidal reaches of each of the rivers within the study area. In summary the design discharge parameters for each river are presented below.

Table 15.6 - Summary of Design Fluvial Discharges

Fluvial Event (years)	Discharge (m3/s)		
	Tolka	Dodder	Liffey
1	20	30	70
10	55	125	195
50	75	190	310
100	90	220	350
500	120	290	400
1000	140	320	420

Joint Probability Considerations

The above parameters have been chosen for use in determining appropriate design parameters for assessing the flood risk around the study area and later for development of appropriate flood risk reduction options and for the flood forecasting model. However in most cases and for much of the DCFPP study area, these parameters can not be considered in isolation but must be considered jointly and the likely correlation between them considered in some way.

For the purposes of this study it is sufficient to consider combinations of two parameters at any given location across the study area. The combinations to be considered at specific locations can broadly be classified as follows:

- Coastal Regions outside North Bull and Great South Training Walls – Extreme tide and wave combinations.
- Outer Harbour Regions and Estuaries – Extreme tide and wave combinations.
- Inner Harbour and Upper Reaches of Tidal Influenced Rivers – Extreme tide and fluvial discharge combinations.

The impact of waves in the coastal regions will be more severe than in the harbour regions although some wave energy will penetrate the harbour on high and more extreme tide levels. It is not considered necessary to consider fluvial discharges in the outer harbour. This is because the probabilistic analysis and river modelling work has concluded that the fluvial discharge in the rivers has little influence on the level of the water downstream of Matt Talbot Bridge and certainly downstream of East Link Toll Bridge.

Brief descriptions of the methodology proposed within the DCFPP for considering appropriate combinations of the above parameters is presented below.

Extreme Tide and Wave Combinations.

Common sense, and indeed the results of the probabilistic analysis, tells us that it is not uncommon to have high tides and strong wind speeds occurring at the same time. High wind speeds in turn give rise to waves. What is important is the wind or wave direction, the impact on the coastline can vary depending on a given direction.

The probabilistic analysis results indicate that strong winds blowing towards the coast produce set up in the still water level and create a surge. In addition low pressure systems which are often responsible for stormy conditions, produce surges by the very nature of their lower atmospheric pressure and also produce higher winds. Therefore when extreme tides occur as a result of a surge created by a low pressure system, it would not be unreasonable to expect it to be combined with strong gale force strength winds and hence higher than normal wave action. In the Beaufort scale the following wind speeds are associated with gale force strength,

- Near gale 28 – 33 knots (13.9 – 17.1m/s)
- Gale 34 – 40 knots (17.2 – 20.7m/s)

It has been considered that an appropriate level of protection for the study area should be that against flooding which could occur for any event up to and including that with a 200 year return period (an annual average probability of occurring of 0.5%). For the coastal region a 200 year tide and wave joint probability combined event is required. Given the considered good correlation between these two parameters, a common sense approach, which might be sensible to adopt, would be the consideration of a 200 year tide level with a 1 year wave climate (or 1 year wind).

Evidence to back up this correlation can be found through the first of February 2002 event and indeed more recently on the 28th October 2004, when the highest and third highest recorded tide at the Dublin Port tide gauge were noted. If we consider the event of the 1st February 2002 the following observations were noted:

- Wind speeds at the M2 buoy were recorded to be in excess of 30 knots (15.4m/s) from 06:00 to 11:00 and for at least two hours the wind speed was recorded at around 36 knots (18.5m/s). Wind was general from a southerly direction.
- Wind speeds recorded at Dublin Port were in excess of 35 knots (18m/s) between 07:00 and 09:00 before dropping and then picking up again to 29 knots (14.9m/s) 1 hour before high water and 25 knots (12.9m/s) 1 hour after high water. Wind speeds had a general southerly to south easterly direction.
- Wave heights recorded by the M2 buoy indicated that between 07:00 and 14:00, wave heights were generally in excess of 4m and with period of around 7 to 8 seconds. The largest wave was recorded at 4.4m at 14:00. Whilst the buoy does not record wave direction, it is not unreasonable to assume that this was similar to that of wind direction.

From the wind speed and wave height parameters presented earlier in this chapter it can be seen that

- 1 year wave from 180 degrees is 4.7m.
- 1 year wind speed from 180 degrees is 17.8 m/s.

The conditions recorded for the 1st February 2002 were close to but slightly under the 1 year condition. As indicated in Chapter 9, the extreme tide level associated with this event was of the order of a 67 year return period.

In respect of the event of the 28th October 2004, the extreme tide level recorded was 5.13mCD (2.62mODM) and the results of the probabilistic analysis would place this tidal event at around a 1 in 10 year event.

Based on these observations it could be reasoned that,

- Extreme tidal surges are likely to be very highly correlated with extreme winds, which are most likely blowing towards or parallel to the coastline.
- Extreme winds are likely to be correlated with high tides but to a lesser extent. i.e. extreme winds will not necessarily always be associated with a significant surge particularly if they are blowing offshore or slightly west of south/north.

For the purpose of determining a series of 200 year joint probability combinations for use within the project, a joint probability formulae developed in house will be used to produce pragmatic combinations guided by the above philosophy. This approach has been widely used for determining appropriate combinations in the assessment of flood risk and has been presented in a number of papers at DEFRA and a hydrology seminar in Ireland.

The basic equations is presented below,

$$Tc = \frac{242.Tt.Tw}{k(Dt + Dw)}$$

Where;

Tc = joint return period (years)
Tt = Tidal return period (years)
Tw = Wave return period (years)
K = correlation factor
Dt = duration of tidal event (days)
Dw = Duration of wave event (days)

K = 1	independence
K = 2	low correlation
K = 20	modest correlation
K = >100	good correlation

The basis for this JP equation is that there is a recognition that for two events to occur together they must happen at the same time and not just in the same year. Thus a 1:10 year event and a 1:20 year event have a JP of 1:200 years of occurring together in the same year, assuming the events have independent origin. However, if each event was only 1 hour long then the JP of them overlapping in time is much greater than 1:200 years. The equation attempts to represent this by considering the combined likely duration of each event within the time frame of which they could potentially occur in any given year. The 242 represents the number of days in any given year when it is likely that combinations of extreme events will occur, i.e. say 8 months out of the year. In addition the equation also includes an empirical factor reducing the joint probability by allowing for a physical linkage or correlation between events. For example high surges are associated with strong winds blowing onshore which also produce waves and so these two parameters could be well correlated for a given wind direction.

Using the above equation appropriate combinations were developed in two ways.

(A) The first considering a range of extreme tidal events from a 1 in 200 year return down to a 1 in 0.05 year return period and what wave events would be required to give a 200 year joint probability combination. With the 200 year tidal event a high correlation factor is used to represent the high correlation considered, while for the 0.05 year tide a lower but still relatively high correlation factor was used.

(B) The second considered a range of extreme wave events from a 1 in 100 year return down to a 1 in 0.05 year return period and what tidal events would be required to give a 200 year joint probability combination. With the 100 year wave condition a modest correlation factor was used to represent that fact that high waves will be associated with some surge activity but not necessarily always very extreme surges. For the 1 in 0.05 year wave condition a higher correlation factor was used.

Combining the two ranges of scenarios, a list of likely combinations of wave and tidal conditions was established to represent the 200 year joint probability condition and these are presented below.

Table 15.7 - Summary of 200 year Tide and Wave Combinations

Scenario	Joint Event (yrs)	Tidal Event (yrs)	Wave Event (yrs)
1	**200**	**200**	**0.62**
2	200	100	1.25
3	200	50	2.1
4	200	10	8.25
5	200	4	10
6	200	0.6	50
7	200	0.3	100

Details of the analysis described above in deriving the above combinations is presented in Figure 15.4.1, Appendix O.

In a similar way combinations, where required, were also developed for other joint probability events as required for the analysis.

The combinations given above are reflective of conditions, which will give the worst overall flood risk combinations for the study area. For example if winds were more west of north or south, then whilst large waves could exist, significant surge build up accompanying those waves is unlikely and so assessment of joint probability for such directions might require a much lower correlation factor.

Extreme Tide and Fluvial Combinations

In a similar way to the development tidal and wave combinations, the joint probability relationship presented above has been used to determine suitable combinations of tidal and fluvial events which jointly might represent a 200 year condition.

In a similar way, a credible philosophy in relation to the likely correlation of any two events was considered and based on the correlation analysis undertaken as part of the probabilistic analysis. That analysis suggested that,

- A low surge was noted to occur at the same time as higher than normal fluvial discharges. Values in the order of 0.2 to 0.35m were noted with fluvial discharges of between 60 and 120 m3/s in the River Liffey. The analysis suggests that high fluvial discharges have an effect on the water levels at the port, but this effect is through the air and not through the water. By through the air, we mean the increase in water levels is caused by methodological conditions such as low pressure and wind and not significantly by the discharge in the river.

What this means is that conditions that might give rise to a high flow within the rivers, are likely at least to result in some localised surge effect, i.e. low pressure system over river catchments in Dublin causing heavy rain, is likely to also produce some set up in still water levels.

Therefore when using the joint probability relationship, and starting with a high fluvial discharge, a modest to good correlation factor should be used to represent this modest correlation with surges.

This correlation analysis also observed, within the timeframe of the data set used, a number of high surge events. For the data set the following observations could be made,

- Surges of >0.4m coincided with discharges of less than 60 m3/s
- Surges of >0.6m coincided with discharges of less than 30 m3/s

In general a low correlation relationship was noted between high surge levels and discharge in the rivers.

Therefore it can be considered that whilst some of the surge required to produce an extreme surge event may be attributed to local meteorological conditions, a more significant portion must originate from conditions outside of the local area.

Therefore when using the joint probability relationship, and starting with a high tidal event, a relatively low correlation factor was used to represent this lower correlation.

In a similar way to the tide and wave analysis, combinations were developed in two ways. One working out the required tide events to combine with a range of fluvial events from 100 year condition down to a 0.05 event and the second considering the required fluvial events for a range of tidal events from a 200 year condition down to a 0.05 year event. These two ranges of scenarios were then combined to provide a realistic set of scenarios from which to assess the 200 year combined tidal and fluvial flood risk.

In a similar way combinations where required, were also developed for other joint probability events.

The scenarios considered for use within the tidal reaches of the rivers are presented below,

Table 15.8 - Summary of 200 year Tide and Fluvial Combinations

Scenario	Joint Event (yrs)	Tidal Event (yrs)	Fluvial Event (yrs)
1	200	200	0.5
2	200	100	0.75
3	200	50	1
4	200	10	1.5
5	200	3.5	10
6	200	0.75	50
7	200	0.35	100

Details of the analysis described above, in deriving the above combinations is presented in Figure 15.4.2, Appendix O. It should be noted that for Scenarios 1, 2 & 3, the analysis indicated fluvial events of lower magnitude than shown in Table 15.8, however extrapolation below fluvial events of 0.5 to 1 year was not practical and the above events have been chosen as representative of suitable fluvial events for Scenarios 1 to 3.

15.4.3 Discussion on Results of Initial Flood Risk Analysis

In this section it is proposed to provide a general discussion of some of the initial observations during the flood risk assessment process, present some of these results and discuss how that process and choice of design parameter evolved within the overall analysis.

Coastal Flood Risk Assessment

For the coastline the important parameters, as discussed earlier, include those of tide levels and wave climate and/or wind speed and direction. In Table 15.7 a range of scenarios which are considered to represent a joint 1 in 200 year wave and water level condition are presented. In assessing the flood risk it is important to consider some or all of these scenarios, as one may provide a higher level of risk than another and as such the one providing the greatest level of flood risk should be used as the defining criteria.

Values representative of the events in Table 15.7 were taken from Table 15.3 and Table 15.4 and transferred to the coastline using the results of the wave transformation matrices developed as part of the wave modelling work described in Chapter 12. This was undertaken for different wave directions to determine which direction provided the worst nearshore wave climate for various locations. Overtopping scenarios occurring as a result of these nearshore wave climates, were then assessed using the overtopping matrices produced through the overtopping modelling work described in Chapter 12.

From this assessment it soon became clear that the worst overtopping and hence flood risk resulted from the higher water level combinations, i.e. Scenario 1 in Table 15.6.

The physical explanations for this can be attributed to the fact that conditions around much of the coastline are depth limited, i.e. the bathymetry close to the coastline and defences is high which results in shallower water depth which in turn forces the waves to break. Therefore for the extreme tide levels, the water depth will be maximised, allowed larger waves to approach the coastline and for the lower water levels wave braking will be greater. While a 100 year wave offshore is extremely large, as it propagates towards the coastline, it will be affected more by the shallow water conditions. As such the wave heights at the coastline for Scenario 1 (offshore wave of 1 year) are larger than those of Scenario 7 (offshore wave of 100 years). Indeed even when a 100 year and 1 year wave were transferred to the coastline on the same water level, the nearshore wave heights where very similar.

This lead to the conclusion that it was not necessary to analysis all of the scenarios presented in Table 15.7. On this basis it was concluded that Scenario 1, with the extreme wave climate rounded up to a 1 year event, would be taken as the criteria against which an acceptable standard of protection and hence flood risk would be evaluated.

200 year joint probability condition for assessment of acceptable level of flood risk to be taken as a 1 in 200 year total (astro+surge) tide level combined with a 1 in 1 year wave.

Intertidal Flood Risk

For the intertidal reaches of the rivers the important parameters, as discussed earlier, include those of tide levels and fluvial discharge. In Table 15.8 a range of scenarios which are considered to represent a joint 200 year fluvial discharge and tide level condition are presented. As with the coastal situation, it is important to consider some or all of these scenarios, as one may provide a high level of risk than another.

To assess each of these scenarios, the “prediction spreadsheet” developed as part of the numerical modelling of the tidal reaches of the rivers, see Section 13.4.4, was used.

Values representative of each of the events in Table 15.8 were taken from, Table 15.3 and Table 15.6, for extreme tide and fluvial discharges respectively. The results for both the River Dodder and River Liffey are presented in Figures 15.4.3 and 15.4.4, Appendix O. These figures show the present day conditions, i.e. without allowance for climate change.

When we consider the back water curves on each figure, it can be seen that over the downstream end of the tidal reach the water levels and hence flood risk are governed by the scenarios which are tidally dominated, i.e. Scenario 1 – 200 year tide plus 0.5 year discharge. At some point along the tidal reach, these water levels and hence flood risk become more fluvial dominated, i.e. Scenario 7 – 100 year fluvial discharge plus 0.35 year tide.

It can clearly be seen on the River Dodder this transition point is very close to the downstream end of the river and occurs in the vicinity of London Bridge. On the River Liffey the point of transition is much further upstream and occurs around the location of the Fr Mathews Bridge.

This can be explained by the fact that the River Dodder is a much smaller river, with a fairly steep bed slope, yet the design discharges are considerable. In addition the bridges along the central reach of tidal River Dodder all have low and obstructing soffit levels. These cause considerable obstruction to the flow and hence result in extremely high backwater levels. On the River Liffey most of the bridge soffit levels are higher than the back water curve for each of the design scenarios. The exceptions being

- the soffit level of the Sean Heuston and Frank Sherwin Bridges which become submerged on the higher fluvial dominated events, i.e. Scenarios 6 & 7. Indeed the Sean Heuston Bridge is particularly restrictive for both of these events.
- The soffit level of the Blackhall Place Bridge which is submerged by all but Scenarios 4 & 5.
- The proposed soffit level of the Macken Street Bridge which is submerged by Scenarios 1, 2 & 3. It should be noted that this level (2.7mODM) was taken from a drawing marked TBC and should be clarified.

Please note the above are soffit levels and not bridge deck levels.

The results on the upstream reaches of the Rivers, for the highest fluvial dominated scenario, predict very high back water levels. This is particularly the case for the River Dodder. These very high levels on the River Dodder can be attributed to the steep bed gradient and the restrictions caused by the low bridges. On the Liffey the bed gradient is very much flatter and the step up in back water curve, upstream of Sean Hueston Bridge, is mainly due to the reducing river cross section and restriction caused by a couple of bridges In that vicinity.

When allowances are made for climate change, which include a sea level rise allowance of 4.15mm/year and an increase in the fluvial flows by 20%, the results for the fluvial dominated scenarios become even more pronounced.

The primary aim of the DCFPP is to consider the risk of flooding from coastal flooding and the development of options to reduce that. As part of this it was necessary to consider sensible joint probability combinations as outlined above. However, in light of the initial findings with respect to the upper tidal reaches of the rivers and the concerns noted in Chapter 13 with regard to lack of data to aid validation of the model under these high fluvial scenarios, it is not considered prudent at this stage to proposed options which might be capable of dealing with the flood risk presented by the fluvial dominated scenarios, without first considering the river catchment as a whole. For this reason options and flood risk maps for the rivers have been restricted at this stage to tidally dominated scenarios, which is also in line with the original project boundaries that were set. However, locations where fluvial flood risk is an issue have been identified and recommendations made for further considerations to ensure that this risk is fully evaluated and dealt with. The main issues are

- The lack of validation data along the reaches investigated. In this respect it is considered important that some new flow and water level recording stations are installed as a matter or urgency and the model checked with actual observed data.
- The restriction of the investigation to within the tidal reaches of the rivers. This precludes the examination of the catchment as a whole which could be extremely important for the development of flood alleviation options. I.e. options to contain the flood waters within the tidal reaches only would be technically difficult and very obtrusive.

- The use of 20% increase in fluvial flow to cover likely climate change. This has significant implications for the flood risk associated with the high fluvial flows. In many locations, while there is a low risk of fluvial flooding now, a considered increase of 20% in discharge significantly increases the level of this risk. The basis for this figure should be further evaluated and the implications of having to deal with it in financial terms fully justified.

It is therefore highly recommended that the above issues are addressed before evaluation of the flood risk associated with respect to high fluvial scenarios is finalised.

Recommended Actions:

- Installation of flow and tide recording gauges within the tidal river reaches. On the River Liffey it is recommended that the flow gauge at the Island Bridge Weir is reinstated or a new one within that vicinity provided.
- Extension of the river models to cover the upstream reaches and a study to investigate the flood risk associated with the rivers on a catchment wide basis.
- Review 20% increase in design fluvial flow recommendations in context of financial and technical implications of incorporating it.

An additional consideration for the rivers is the change in bed level over the years. Increased sediment build up could also enhance the flood risk, particularly for high fluvial conditions. No historic information on bed level for the River Liffey was uncovered as part of the data collection process, however an old DCC drainage plan was discovered for the River Dodder.

This old Dodder drawing did not have a date or reference to the datum used. However all levels were given in feet and so it is concluded that the survey was at least pre 1965. In addition a comparison of the bridge soffit levels with soffit level measured during the DCFPP survey lead to the conclusion that the datum used was Poolbeg and so river bed levels were converted on that basis and compared with the bathymetry survey undertaken as part of this project.

The results are presented in Figure 15.4.5, Appendix O and show that there is no significant change in bed level over the tidal reach, although the general trend appears to be one of accretion. This accretion is generally less than 0.5m. Nevertheless there is likely to be some merit to cleaning out the river bed by up to this amount over the tidal reach. Care should be taken not to dredge to close to the river walls or around the bridge abutments without further investigation works.

15.4.4 Locations at Risk and Standard of Protection

Methodology

In order to help in the flood risk evaluation process the project area was broken into a number of lengths. These lengths include:

Length 1 - Merrion to the Great South Wall
Length 2 - Great South Wall to East Link Toll Bridge
Length 3 - East Link Toll Bridge to Mouth of the River Tolka
Length 4 – Mouth of the River Tolka to the Central Causeway at Bull Island

Length 5 – Central Causeway at Bull Island to Martello Tower, South West Howth
Length 6 – Howth Harbour to Mouth of the Baldoyle Estuary
Length 7 – Baldoyle Estuary
Length 8 – Portmarnock
Length 9 – River Liffey
Length 10 – River Dodder
Length 11 – River Tolka
Length 12 – Royal Canal

In order to help with the assessment flood risk spreadsheets were developed. Where applicable primary defence, secondary defence and road levels were presented with chainages, so that extreme water levels could be compared against each. The level data was taken from the DCFPP survey described in Chapter 4. At some locations no level information was available due to access restrictions at the time of the survey. For such locations, estimates of the defence/ground levels have been made from follow up site visit and interpolation from other data sources. These have been shown in red in the spreadsheets and clarification of the flood risk at the more strategic of these locations should be undertaken with follow up survey work following access approval from those land owners involved.

In addition extreme water level data was incorporated in to the spreadsheets for this comparison. A full range of events from 1 in 2 to 1 in 500 year was considered and the appropriate water levels based on the results of the probabilistic analysis and transferred to the relevant location within the project area using the results of the FINEL modelling around the coast. For the tidal reaches of the rivers extreme water levels based on the scenarios presented in Table 15.7 for the 200 year event and other combinations for the 100, 50, 10 and 2 year event were developed by similar means and included for comparison with the river defences.

A number of comparisons were undertaken. The first included a straight forward comparison of still water level with defence levels and road levels. However in general it is practical to allow some order of freeboard to deal with uncertainties in the analysis, deficiencies in the defences and particularly on the coast wave action. On the coast where the wave action dominates, the required freeboard allowance and requirements can vary form location to location depending on exposure to wave action, bed levels etc.

The freeboard is taken into account, particularly in the determination of existing standard protection and can be defined as the difference between the actual defence level and the still water level. The standard of protection is the level of protection against a given event which the defences afford. This is generally presented as a return period. For example a defence with a 20 year standard of protection will provide adequate protection against events up to and including those with a statistical return period of 20 years when adequate allowance has been made for freeboard. Or in terms of probability there is an annual probability of 0.05 or 5% that the defence will be exceeded.

For the analysis a detail assessment of the appropriate freeboard allowance has been made for the complete coastline, considering the nature of the defences, the design conditions, the assets at risk and in regions of wave action acceptable overtopping limits.

For the purposes of determining the standard of protection an overtopping threshold of 0.0001m3/s/m was chosen. This was considered appropriate for locations where infrastructure such as a road was located within 5 metres of the defence. However, this value was reviewed across the study area and varied as considered appropriate. For example the threshold was increased slightly in regions where the important assets were set back from the coastline, such as the Clontarf frontage along the length of the Linear Park.

The asset database defence boundaries were also marked on the spreadsheets at the relevant chainage, so that standards of protection could be defined to these defence units, thus providing a holistic assessment of risk which considers both the level of the defence and the condition.

In considering the standard of protection for a given area or defence it is customary to take the lowest standard of protection defined by the point of lowest level of the defence, subject to the defence being of sufficient strength and condition to cope with the event in question. Therefore from the detailed spreadsheets, summary spreadsheets were produce presenting a standard of protection of each asset database defence unit based on the minimum value calculated within that unit. These summary sheets were then used to produce standard of protection plans which clearly show how the level of protection varies from defence unit to defence unit and around the project area as a whole. The standard of protection plans are presented in Appendix O and are labelled SoP – 1 to SoP – 12 to correspond with the lengths identified above. When using these plans it should be noted that these represent the minimum value as stated above. Therefore before prioritising works relative to issues such as Standard of Protection, consideration should also be given to the detailed spreadsheets to investigate whether this minimum value is a one off value in an otherwise relatively well defended area. This issue has been considered when summarising the current standard of protection across the study area in the following section by showing the maximum, minimum and "representative" values of standard of protection around the study area, see Table 15.9.

In addition to current standard of protection, an assessment was made of how that standard of protection will vary with time and due to climate change. These results are shown in the summary spreadsheets also presented in Appendix O, however only the present day standard of protection has been shown on the standard of protection plans.

A general overview summary of the results of this assessment for the current (2004) standards of protection are presented in the section below. For more detail around the project area see the spreadsheets and SoP plans (Figure no's SoP1 to 8) presented in Appendix O.

Summary Overview of Existing Standard of Protection

A summary of the current Standards of Protection around the study area are presented in Table 15.9 below. The range of standard of protection across the area is indicated by the highest and lowest values for that section of coastline. Because the standard of protection varies from one defence to the next and indeed along some defences and also due to the fact that the lowest value may be due to the existence of a single gap or low spot in an otherwise relatively well defended section of the coastline, a representative value has also been presented and is considered to reflect the overall levels of protection afforded across this region of the project area. These values are not derived from complex calculations of average standard of protection but are based on a common sense overview of the variation in the level of protection as identified within that area. For example if the majority of the coastline reflects the highest standard noted then the representative value has been biased to this higher value etc. It is felt that this value gives a relatively effective overview of the representative level of protection around the project area.

This should be borne in mind when looking at the standard of protection plans and summary spreadsheets for each asset data base defence unit presented in Appendix O, which present only the minimum standard for each defence unit.

The last column in the table makes a simple statement as to whether assets are at risk. This is quantified in more detail through the production of the flood risk maps, see Section 15.4.6.

Table 15.9 - Summary of Current Standards of Protection

Overall Locations	Description	Existing SoP (2004) Yrs			Property at Risk
		Highest	Lowest	Representative	
Merrion to Great South Wall	South Merrion Gates	>200	130	175 (0.57%)	Railway Line
	Merrion Gates	-	6	6 (16.6%)	Yes
	Property North Merrion	>200	70	125 (0.8%)	Yes
	Promenade to Martello Tower	>200	30	100 (1%)	Yes
	Martello Tower to End Promenade	>200	5	40 (2.5%)	Yes
	Promenade to Sean Moore Park	>200	1	25 (4%)	Yes
	Sean Moore Park to GSW	>200	40	180 (0.55%)	Power Station
GSW to East Link Toll Bridge	GSW to DPC Quays	>200	20	>100 (<1%)	Power Station & STW
	DPC Quays	>200	>200	>200 (<0.5%)	No
	Eastlink Toll Road	130	10	50 (2%)	Toll Road
East Link Toll Bridge to River Tolka	DPC Quays & East Wall Business Park	>200	180	>200 (<0.5%)	No
River Tolka to Central Causeway Bull Island	Alfie Bryne Road	>200	>200	>200 (<0.5%)	No
	Clontarf Road (To Bull Bridge)	>200	5	10 – 20 (5 – 10%)	Yes

Overall Locations	Description	Existing SoP (2004) Yrs			Property at Risk
		Highest	Lowest	Representative	
	Bull Bridge to Central Causeway	>200	66	190 (0.53%)	No
Central Causeway to Howth South	Central Causeway to End of Bull Island	>200	126	190 (0.53%)	No
	Sutton Cross Properties	>200	10	50 – 100 (1 – 2%)	Yes
	Greenfield & Strand Road	>200	<5	5 – 10 (10 – 20%)	Yes
Howth North to Baldoyle Estuary		>200	20	150 (0.66%)	Limited
Baldoyle Estuary	Baldoyle Town	>200	10	100 – 175 (0.57 – 1%)	Yes
	Coast Road	>200	5	10 – 50 (2 – 10%)	Yes
Portmarnock		>200	10	(50 – 100) (1 - 2%)	Yes
Royal Canal	Right Hand Bank	>200	6	10 – 50 (2 – 10%)	Yes
	Left Hand Bank	>200	10	10 – 25 (4 – 10%)	Yes
River Dodder	Right Hand Bank	>200	25 - 50	50 –200 (0.5 – 2%)	Yes
	Left Hand Bank	>200	25 – 50	50 – 200 (0.5 – 2%)	Yes
River Liffey	Right Hand Bank	>200	25 - 50	100 -200 (0.5 – 1%)	Yes
	Left Hand Bank	>200	25 - 50	100 – 200 (0.5 – 1%)	Yes
River Tolka	Right Hand Bank	>200	180	190 (0.52%)	Yes
	Left Hand Bank	>200	>200	>200 (<0.5%)	No

Note: SoP's presented above for the Rivers Liffey & Dodder are based on tidally dominated scenarios.
Right and left banks are defined as looking downstream
100 year SoP defined as the level of protection against which the defence affords, i.e. flooding will not occur for extreme events with a return period of less than or equal to 100 years.

SoP as a % is the average percentage risk of an event occurring which would be on the threshold of flood risk.

In Section 15.4.2, it was considered that an initial assessment of the appropriate level of protection against flooding for Dublin and the surrounding area should be taken as a 1 in 200 year event (a percentage risk of 0.5%). When considering the results presented above it can be seen that whilst most of the coastline and rivers contain defences which will provide a standard of protection to this level, there are significant weaknesses within the defences which ultimately govern the level of protection provided to the land behind. In some places the lowest levels of protection are very low indeed, with values of less than 10 years being seen in many locations. In respect of representative levels of protection, only the Dublin Port Quays and the coastal frontage at Portmarnock meet this initial indicative standard of protection in their entirety.

In respect of the rivers, the standard of protection in parts of the upstream reaches could be considered to be lower than the values set out in the representative column, which are based on tidally dominated conditions. At a number of locations the standard of protection with respect to fluvial flooding, could be considered to be between 10 and 50 years. This is particularly the case on the River Dodder, upstream of New Bridge where some low earth banks exist on the left hand bank. Furthermore on the upper reaches of the Liffey around Sara Bridge and the Rail Bridge downstream, there are a number of private houses and home complexes which are at risk from fluvial events in excess of around 25 to 50 years. These flood risks should be further quantified and considered in the context of the wider catchment.

15.4.5 Assessment of Potential Flood Damage Function

In earlier sections of this chapter potential flood compartments have been identified through use of the DTM and extreme water levels developed as part of the probabilistic analysis. The locations at risk of flooding around the coastline were then identified and the standard of protection provided at these locations evaluated or in other words the level of hazard that could be contributed to likely flooding.

The identification of the locations at risk through inadequate standards of protection, combined with low coastal zone topography, provides an effective means of establishing the overall level of risk and the areas at risk around the study area. Detailed predictive flood hazard maps have not been produced as part of this report. This aspect of work is later to be progressed through the SAFER project.

Nevertheless, it was necessary in order to evaluate the risk of flooding further to help attribute a level of priority to particular areas at risk, to roughly investigate likely areas of flooding. This allowed for the investigation of economic damage to help in the prioritisation process.

These flood extents were established considering the extent of flooding that could result within each of the identified flood compartments, through weiring and wave overtopping at each of the flood risk locations identified.

Around the coastline the extent of flooding has been modelled by considering direct weiring over the defences where it could occur and/or the volumes of water which could overtop due to wave action. Where sections of the coastline could be affected by a wave climate from different directions, the 1 year climate was assessed from each direction together with the overtopping that would result. Subsequently the condition providing the worst case scenario was taken to represent the flood extent.

Based on the flood extents identified, estimates of properties and infrastructure at risk were obtained from the councils gazetteer databases and other sources and these used to evaluate flood impacts. A further explanation of this work is give in section 15.5 below.

15.5 Flood Impacts and Assessment

15.5.1 Methodology

As noted above predictive flood maps will be developed as part of the SAFER project once national and international protocols have been established. Within this project general flood damage functions have been derived for the 1 in 200 year events estimated now and with climate change up to year 2051. The flood damage functions determine the potential economic benefits that are available and can be used to evaluate the benefits of the improvement works set out in Chapter 17.

When considering damages and hence benefits associated with any flood event scheme, two types of benefits are available: direct benefits and indirect benefits. Direct benefits include such things as direct flood damage to property due to depth and duration of flooding avoided. Indirect benefits can arise from preventing such things as: transport disruption due to road or rail; cost of responding to and dealing with a flood event, i.e. local authority/fire brigade cost etc.; environmental damage and health and safety issues including loss of life.

Within the scope of this project, only the direct damages have been assessed in monetary terms. Use has been made of data obtained from the 2002 flood. These concern repairs to properties, replacement of goods, provision of temporary accommodation and basic medical requirements linked to flood events.

Regarding the indirect benefits, only the environmental benefits have been dealt with, see chapter 18 for a qualitative description of the potential benefits associated with implementing the options.

For the purposes of this study the direct damages have been determined as set out below.

15.5.2 Damages as function of probability

For any given event the actual direct damage associated with that event can be calculated. However, that is not necessarily the damage against which improvement works can be justified, although it is an important consideration. These damage values should be turned into “annual average damage” values by considering various events with known return periods. By doing this and integrating the damages as a function of return period (and therefore probability), the average annual damages may be derived.

Within the scope of the project four flood hazard extents have been determined:

1. Standard of Protection (SoP) at present, i.e. the maximum event with a certain return period at which no flooding will take place now;
2. 200 year event present day, i.e. the event with a 200 year return period at which flooding will take place due to weiring and overtopping;
3. Standard of Protection (SoP) in 2051, i.e. the maximum event with a certain return period at which no flooding will take place in 2051;
4. 200 year event in 2051, i.e. the event with a 200 year return period taking sea level rise into account up to 2051 at which flooding will take place due to weiring and overtopping.

By setting points 1 and 2 out in a graph (damage as a function of probability) and assuming a linear relationship between the points, the average annual damage can be assessed for the present situation by integration. Analogous, this can also be done for 2051 using points 3 and 4. The net present benefits for the design life of the options, i.e. up to 2031, can subsequently be calculated. For this a net discount rate of 3% has been used.

However, in essence the actual net present benefit may well differ. Two reasons are:
- this approach assumes that the damage that is shown now for events above the 200 year will still occur with the new defences in place, whilst in reality this is unlikely to be the case, unless your new defence failed completely under events greater than the design condition; this results in an underestimating of the benefits;
- the linear relationship has been adopted to keep the analysis simple and transparent; this approach could however result in an underestimation or an overestimation of the benefits.

Throughout the lifespan of the flood defences, these average annual damages could potentially occur in any given year. When discounted to the present day, the net present damages are found, i.e. the net present benefits that may be derived from the flood defences in monetary terms. This value can then be set against the whole life costs of any improvement scheme over the design life of that scheme to determine the economic viability of the works in benefit cost ratio (BCR) or net present value (NPV) terms.

Based on this approach, the following table has been derived:

Table 15.10 – Summary of 200 Year Event Flood Damage

No	Area	Flood damage assessment [NPV benefits in €]
1.	Baldoyle Estuary	2.2 million
2.	Baldoyle Town	0.6 million
3.	North Howth	-
4.	Howth South West	13 million
5.	Clontarf	40 million
6.	East Link	200 thousand
7.	ESB Poolbeg Power Station	-
8.	Sandymount Strand (north)	17 million
9.	Sandymount Strand (promenade)	2.3 million
10.	Merion Gates	35 million
11.	River Liffey/Royal Canal	Right bank: 6.7 million

		Left bank: 75 million
12.	River Dodder	15 million

It should be noted that the assumption that the benefits are limited to the design life of the options, i.e. up to and including 2031, has a significant impact on the outcome. In the event that the design life would be up to and including 2051, the benefits mentioned above could increase by more than 50%. Obviously this would have to be offset against the additional costs of the higher and stronger defences.

16 STRATEGY AND POLICY FOR STUDY AREA [2]

16.1 Introduction

16.1.1 Background

The investigation into appropriate policy and strategy began with a series of face-to-face interviews with staff from Dublin City Council and Fingal County Council. This led to the need for a definition of both terms in order to set the context for this work.

The Concise Oxford English Dictionary defines Policy and Strategy as follows:

Policy: *"a course or principle of action adopted or proposed by a government, party, business or individual ".*

Strategy: *"a plan of action or policy in business, politics, etc.".*

The two terms are often confused and used incorrectly. Within the context of this study, "policy" is defined as the principle or intent, whilst "strategy" is about the means of carrying it out, i.e. the plan. Thus, in the context of the Dublin Flood Protection Project, policy would include, for example, the adopted approach to the provision of flood warnings, whilst strategy would propose the concept and timetable for its implementation.

Ownership is an important element of the above definitions; i.e. whose policy, and whose strategy? Both policy and strategy confer obligations, almost certainly upon the body that is proposing them, and probably upon others who are required to comply with any regulations that are thus derived. This is discussed further, together with other ground rules or "business drivers", in Chapter 2.

The purpose of this chapter is to explore issues of policy and strategy, and hence to make recommendations that may be acted upon by the authorities responsible for this project.

Outline policies are suggested, but the report is not, and neither should it be, construed as a policy document. The aim is to promote discussion relating to the direction and ownership of policies and strategies proposed. There are formal outlets for the dissemination of the policies, e.g. council development plans, bye-laws, etc. and the authorities may elect to utilise these and other statutory vehicles to adapt and implement the recommendations given herein.

16.1.2 Process for Defining Policy and Strategy

The process for defining policy and strategy for the Dublin Coast Flooding Protection Project has comprised five stages, as described briefly below:

[2] This Chapter was written before the publication of the OPW Review of Flooding Strategy which document addresses many of the observations on national issues. This Chapter is to be reviewed for compatibility with OPW Policy.

1. Establishing the **business drivers** (Chapter 2.0): in this section, the basic parameters that define policy and strategy in the context of Dublin's flood protection are set out. This includes the "ownership" of the policies, which is essential in terms of setting policies at the right generic level, so that they can be acted upon by the same bodies, or others. Timescale and geographic context are also important parameters, and these too are defined;

2. A number of **key references** are reviewed (Chapter 3): the purpose of this is twofold, firstly to learn from well tested cases, and secondly to derive a "long-list" of headings covering relevant aspects of flood protection;

3. Dublin's **present practices and aspirations** are overviewed in Chapter 4; using this list of headings, present practices are mapped out together with aspirations for future policy;

4. **Dublin's needs:** by comparing the list of headings with present practice and aspirations (Chapter 5), it is clear to see where policies are already being effected and where they are not, or to what extent they are in evidence. From this exercise, a clearer picture of Dublin's needs emerges. This is refined to take into account generic, legal and statutory obligations;

5. Following directly from Chapter 5, the final step (Chapter 6) is the formulation of **recommendations** for policy. The scope of policies is given together with advice on those areas requiring quantification elsewhere (e.g. flood risks).

16.2 The Business Drivers

16.2.1 The Statutory and Legal Context

Policy can be set at several levels by different agencies:

- Central Government;
- Government Department (e.g. Office of Public Works, Department of the Marine);
- Local authorities (i.e. Dublin City Council, Fingal County Council); and
- Private sector e.g. the Insurance industry or developers.

The setting of a policy does not necessarily mean that the body which set it must also carry it out, although this may indeed be the case. The policy setting agencies may themselves be tasked with reviewing the accomplishment of a given policy (this obligation being part of the policy). In addition to the public bodies referred to above, policies might also be set and influenced by the private sector, for example the Insurance Industry and developers.

Dublin City Council and Fingal County Council are executive organisations. That is, they are empowered to undertake works in compliance with policies set by Government, its Departments and indeed by the Authorities themselves. Recently, the Office of Public Works has been responsible for a review of policy relating to flood prevention. This has been through the consultation stage and is now before the Government Finance Committee for ratification and approval.

To put such policies into effect there must be in place, a Strategy for Flood Protection (or Strategies if the responsible authorities are separate entities). Whereas there may be a number of policies to contend with, originating from various sources and motivations, there possibly only needs to be one Strategy (or one for each Authority). This chapter of the report therefore deals essentially with policies and how these relate, or might relate, to the four generic levels described in the first paragraph above.

16.2.2 The Geographical Context

In terms of policy, the geographic boundaries are wide and variable, to the extent that global policies might apply, e.g. emanating from international advice on climate change. On the other hand, the Strategy for Flood Protection can relate to jurisdictional boundaries (Dublin and Fingal) providing that it recognises any influential physical boundaries that may extend beyond the Authorities' "territories".

16.2.3 Time Frame

Policy for flood protection should relate to sustainable practice. In this respect, the timescale should reflect the long term physical development of the waters' edge, climate change, and socio-economic development. Thus, policy should be considered to apply for a minimum of 50 years. This does not preclude the possibility of altering or adding to it, merely that a long term view should be made.

The timescale for Strategy should therefore accord with that of Policy. Notwithstanding that, there will be a need for Strategy to evolve in line with changing circumstances, and to a lesser degree, Policy. A workable timeframe in which to review, and revise Strategy would be a 5 yearly cycle.

16.3 Review of Key Documents

In this section, a number of key documents, including some important international references, are discussed. The purpose of this review is to learn from other cases of good practice of policy, and its application, in flood protection. The process also serves to identify a list of headings that are relevant to Dublin.

16.3.1 International References

Some key international documents are reviewed in respect of their relevance to policies for the Dublin Coast Flooding Protection Project. Whilst a broad range of literature has been examined for the project, the review given here concentrates on important and definitive references rather than attempting to appraise the greater mass of material.

The documents reviewed cover different aspects of flood protection (policy and strategy) and, as such, should not be compared one with another. It has not been the intention to appraise various policies with the view to picking the best; rather, the cases that have been selected represent good examples in respect of the subjects they cover. With this in mind the following table shows the key documents examined, together with the main subject matters covered:

Table 16.1 – Key international documents and main covered subjects

A	**Strategy for Coastal Defence, MAFF, UK, 1993**	- **scope of policy on flood protection** - **prioritisation**
B	**Fundamentals on Water defences, Technical Advisory Committee on Water Defences, the Netherlands, 1998**	- **institutional framework** - **legal framework**
C	**Planning Policy Guidance Note 25 (PPG25): Development and Risk , Office of the Deputy Prime Minister, UK, 19xx**	- **planning** - **precautionary principle and sequential test**
D	**US – National Flood Insurance Program**	- **relationship with insurance** - **integrated approach**

(A) Strategy for Flood and Coastal Defence in England and Wales: MAFF, 1993

The opening lines of the "Strategy" state *"this strategy document sets out a comprehensive policy framework for the Ministry of Agriculture, Fisheries and Food (now DEFRA), Welsh Office and relevant authorities to work within while carrying out their responsibilities for flood and coastal defence".*

The Strategy's stated aims and objectives are as follows:

"The aim of the Ministry and Welsh Office policy is to reduce the risks to people and the developed and natural environment from flooding and erosion:

- *By encouraging the provision of adequate and cost-effective flood warning systems;*
- *By encouraging the provision of adequate, technically, environmentally and economically sound and sustainable flood and coastal defence measures;*
- *By discouraging inappropriate development in areas at risk from flooding or coastal erosion."*

The document emphasises the importance of safeguarding life as the highest priority. In line with this, the stated priorities are, in descending order:

- *Flood warning systems;*
- *Urban coastal defence (sea defence and coast protection);*
- *Urban flood defence;*
- *Rural coastal defence and existing rural flood defence and drainage schemes;*
- *New rural flood defence and drainage systems.*

Research and development is given a high priority, through pledged funding and dissemination.

The Strategy also encourages the setting up of coastal defence groups for the preparation of shoreline management plans, together with the development of river catchment plans. A corollary to this would be the Flooding Partnerships being established by the DCC as part of the DCFPP.

Maintenance operations that are sympathetic to the environment are recommended, together with Water Level Management Plans, which take account of both operational and environmental requirements and opportunities.

Flood Warning Systems and Emergency Procedures are outlined in the document, with particular emphasis on the Ministry support for the Storm Tide Warning Service. In contrast, very little is said about local flood warning systems.

The Strategy document gives an overview of project appraisal procedures (which are detailed elsewhere, in particular in the PAG series of guidance notes). Other aspects of coastal defence that are discussed include: monitoring (essential element of any scheme or warning system); post project evaluation (checking the value for money of schemes that have been effected; communications (consultation and discussion between interested parties); review (Ministry monitoring of progress towards achieving the strategy).

(B) Fundamentals on Water Defences, Technical Advisory Committee on Water Defences, the Netherlands, January 1998.

This document ("Fundamentals") describes:

(i) the systems that lead to the need for water defence in the Netherlands;
(ii) the systems whose function it is to retain water and how this function is allocated; and
(iii) the decision-making systems.

Whilst the document is, to all intents and purposes, a textbook, it draws substantially on Dutch legislation on flood policy and strategy, and in this respect provides a useful overview. Indeed, the introductory text of Fundamentals describes it as a "hinge" between legislation/policy documents at strategic level and the specific guides/project documents at implementation level. In this part of the report, it is the higher level references from Fundamentals that are or specific interest.

It should be noted that fundamentals uses the term "management" in the same sense that the term "strategy" has been used herein. It describes policy as the *"integrated whole of administrative choices".* It points out the importance of the interaction of policy with its implementation, saying *"….management is the realisation of decisions that have been taken, whereas management experience plays an important role in policy preparation"*. Thus, it recognises the circular logic of policy and its implementation – so often viewed as the "chicken and egg" that is a characteristic of flood defence strategy.

Administrative framework in the Netherlands is outlined as follows:

Organisation

Care of the flood defences is spread over three administrative layers: the water boards, the provinces and the state:

- Water Boards are responsible for the construction, management and maintenance of primary flood defences that surround a dike ring area (an area that must be safeguarded by a system of flood defences). Water Boards are controlled by elected representation of landholders with interests in the protected area. Water Boards have the powers to issue byelaws needed to carry out flood defence. Flood management and maintenance is largely financed through landholder taxation, however, the costs of current dike reinforcement are too high for the majority of water boards, which are therefore subsidised by the State. By comparison, subsidy for construction of river dike improvements was transferred to the "provincial fund" in 1993. The financing of integral flood defence management is (at time of publication = 1998) a source of discussion; the aim is to form large and decisive Water Boards – this is connected to the increasing demands placed on the administrative and technical capabilities of Water Boards.

- The Provinces oversee the Water Boards. They have two specific legislated tasks: monitoring the technical quality of management, and; supervising proper harmony between municipal and water board policy. The Provinces play an important role in the organisation of the system of Water Boards.

- The State has a number of responsibilities, including: legislation; supreme control of the system of Water Boards; management of the primary water defences, and; management of large rivers and lakes. The state is also responsible for maintaining the location of the coastline. The powers and responsibilities of government are affected through the Rijkswaterstaat.

In addition to the three bodies mentioned above, the Municipalities draw up zoning plans in which flood defences must be located. Whilst the Water Boards are principally concerned with flood protection, the Municipalities have other functions and responsibilities in the case of a flood, including contingency plans, maintaining public order, etc.

Legal Framework

Legislation governing flood defence in the Netherlands is effected through a dozen or so main acts. In the context of flooding policy and strategy, further consideration may be given to two of these Acts, viz: the Flood Defences Act (1995), and the Spatial Planning Act (1962):

The aim of the **Flood Defences Act (FDA)** is to guarantee security against inundation. The FDA provides the legal mechanism for construction, improvement and maintenance of flood defences. It also designates the safety norm (standard of defence) of each dike ring (flood compartment). An important and very relevant (to Dublin) aspect of the Act is that it requires the flood defence manager to report on the state of defences in relation to the norm, every five years.

Some relevant headings and aspects of the FDA are bulleted below:

- applies to "dike ring areas" and flood defences (see below*);

- designates the safety standard, being the average annual overtopping of the highest high-water, which the primary flood defence of each dike ring is designed to withstand (see below **);

- the actual capacity of each defence must be assessed and estimated on a five yearly basis by the flood defence manager; the findings are reported to the provincial executive;

- primary flood defences are effected according to a plan adopted by the manager and approved by the provincial director; in any case, the preparation of the plan by the manager, shall involve the provincial executive and senior members of the municipality;

- the Act facilitates the preparation of technical guidelines on design, management and maintenance of primary flood defences;

- to counter a landward movement of the coastline, the works that are considered necessary are funded by central government.

The Netherlands is divided into 53 so-called dike ring areas, together with "high" ground mainly in the east and south which is not at risk. The norms listed against these flood risk areas range from 1/1250 to 1/10000 annual average probabilities of flooding. These high standards of defence are related to the potentially high level of damages that would be incurred in the event of a flood, because of the severity and depth of inundation. The highest standards are in built-up areas at the coast, whilst the lower standards are for the upper river areas, where damages due to fresh water flooding would be less than for sea water.

The **Spatial Planning Act (SPA)** is concerned with land use plans. The primary function of the areas occupied by flood defences is regarded as hydraulic, and in this respect, a municipal building permit is needed for building on (or the building of) a flood defence. The permissible uses of given defence are assigned in a byelaw prepared by the water board.

Other acts deal with: ground acquisition, defence improvements including EIA, use of materials, ground contamination, water pollution, river discharge.

(C) Planning Policy Guidance Note 25: Development and Risk, Office of the Deputy Prime Minister, 2000

This document takes the form of a guidance note. It applies to England and is principally concerned with the reduction of flood risk in respect of land-use planning. As such, the guidance is directed at those with responsibilities in the development process, which can include developers and responsible authorities alike. In respect of authorities' obligations, PPG 25 states that *"local planning authorities should use their existing powers to guide, regulate and control development in accordance with the guidance……, in particular the sequential test".* The sequential test is described later in this section.

At this point it is worth repeating the tenets of PPG 25 as stated in the Foreword to the document:

"This guidance (PPG 25) states that:

- *the susceptibility of land to flooding is a material planning consideration;*
- *the Environment Agency has the lead role in providing advice on flood issues, at a strategic level and in relation to planning applications;*
- *policies in development plans should outline the consideration which will be given to flood issues, recognising the uncertainties that are inherent in the prediction of flooding and that flood risk is expected to increase as a result of climate change;*
- *planning authorities should apply the precautionary principle to the issue of flood risk, using a risk based search sequence to avoid such risk where possible and managing it elsewhere;*
- *planning authorities should recognise the importance of functional flood plains, where water flows or is held at times of flood, and avoid inappropriate development on undeveloped and undefended flood plains;*
- *developers should fund the provision and maintenance of flood defences that are required because of the development; and*
- *planning policies and decisions should recognise that the consideration of flood risk and its management needs to be applied on a whole-catchment basis and not be restricted to flood plains."*

The "precautionary principle" referred to above states that, *"where there are threats of serious or irreversible damage, lack of full scientific certainty shall not be used as a reason for postponing cost-effective measures to prevent environmental degradation".* In essence this means acting on that information which is available including the estimated effects of climate change, and recognising how risks can change with time. Having done that, to proceed with caution according to the estimated level of risk.

Of particular relevance to the setting and administration of policy is a clear understanding of the roles of the relevant "players" in the field of planning development and flood risk. The points listed above outline certain responsibilities; PPG 25 goes on to expand on the powers and responsibilities of: the owner; Government; the Environment Agency; local authorities; Internal Drainage Boards; and the developer. In respect of local planning authorities, PPG 25 states that they are *"responsible for preparing development plans and controlling development, principally in relation to location and amenity, both in the flood plain, where it may be directly affected by flooding or affect flooding elsewhere, and elsewhere in river catchments, where changes in run-off characteristics may increase flooding downstream"*.

The opening paragraph to this section mentions the "sequential test"; the purpose of this is to enable planning authorities to apply a risk based approach to the preparation of development plans. The sequential test is central to the PPG 25 guidance note, being cross-referred substantially throughout the document. It is set out as Table 1 in PPG 25 which, for completeness, is reproduced in Appendix 1.

With respect to Insurance, PPG 25 advises developers to seek the views of insurers at an early stage, whilst authorities should consider consulting their own insurers to ensure that flood defence and other measures are adequate and indeed satisfy the insurers' requirements.

PPG 25 discusses the importance of considering the regional significance of flood risk, as opposed to a local or parochial view that is confined by administrative boundaries. At a regional scale, flooding should be considered at the level of the whole catchment in the case of river flooding, or on the basis of a coastal cell for coastal flooding. The PPG 25 Guidance goes on to advise on the level of detail appropriate at a regional level (Regional Planning Guidance), together with that which is appropriate to structure plans and local plans.

In Scotland an equivalent guidance note to PPG25 is used; Scottish Planning Policy (SSP7), Planning and Flooding covers similar content to PPG25 and includes a form of Sequential Test. In SSP7, the latter is referred to as The Risk Framework. It is fundamentally the same as the Sequential Test but has slightly different thresholds and criteria.

(D) US – National Flood Insurance Program

To complete this overview of international practice of policy and strategy, reference is made to the United States' approach to matters of Insurance, as set out in the National Flood Insurance Program (NFIP).

NFIP is managed by the Mitigation Division of the Federal Emergency Management Agency (FEMA). FEMA is an independent agency of the US government, founded in 1979. FEMA's mission, which is relevant to the present project, is reproduced below:

"to reduce loss of life and property and protect our nation's critical infrastructure from all types of hazards through a comprehensive, risk based, emergency management program of mitigation, preparedness, response and recovery".

In 1968, Congress created the NFIP in response to the rising cost of taxpayer funded flood disaster relief. The NFIP is self-supporting for the average year. This is because the operating expenses and claims payments are funded from the premiums received rather than the tax payer. However, the NFIP has authority to borrow from the US Treasury in the case of severe losses.

The NFIP deals with flood plain management and flood risk mapping, and advises on a wide range of flood mitigation measures including flood proofing, building codes, laws and regulations, training and workshops, storm warnings, etc. Thousands of communities in the US participate in the NFIP; in conjunction with this, the NFIP makes government backed insurance available to residents and the commercial community.

The NFIP advertises its benefits over the alternative (i.e. a scheme that is reliant on state financed disaster relief) thus: *"Flood damage is reduced by nearly $1 billion a year through partnerships with communities, the insurance industry, and the lending industry. Further, buildings constructed in compliance with NFIP building standards suffer approximately 80% less damage annually than those not built in compliance".*

In the US, in order to secure finance to buy, build or improve structures in special flood hazard areas, it is a requirement to purchase flood insurance. The lending institutions are responsible for determining if a structure is located in a flood hazard area, and must give notice if flood insurance is required. Thus, there is considerable encouragement for communities to subscribe to the NFIP scheme.

16.3.2 Irish Flood Review Group Recommendations

At the time of drafting this document, it was intended that a précis be included. We understand that whilst comments received and incorporated during the consultation process have been incorporated, the document has yet to be ratified and approved.

Early indications from a member of OPW suggest that there will be a greater emphasis on self-help and flood warning systems. Dublin City Council and Fingal County Council can therefore be encouraged to continue their forward thinking approach to the development of an integrated flood defence strategy.

16.3.3 Policy Areas

The two previous sections give overviews of some very relevant documents concerning Policy and Strategy for Flood Protection. From these references can be distilled a number of headings that can be used as a basis for developing the discussion further in terms of current practice, present and future needs, and actions. Further to theses references, a policy item on Emergency Response (P14) is added. This is fundamentally important and ties in with Communications, Flood Warning Systems, and a number of other policy topics. The "policy" headings, together with sub-headings are listed below:

Table 16.2 - Policy Headings

Policy Subject	Main Heading	Sub-headings
P1	Institutional and legal frameworks	▪ Government ▪ Authorities ▪ Developers ▪ Owners
P2	Communications	▪ Public awareness ▪ Consultation
P3	Regional perspective	▪ Shoreline Management Plans ▪ Catchment Management Plans ▪ Early warning systems
P4	Climate change	▪ Rainfall ▪ Sea level rise
P5	Planning for development	▪ Importance of flood plains ▪ Precautionary principle ▪ Sequential test – risk based approach
P6	Flood warning systems	▪ Flood forecasting ▪ Flood warning
P7	Flood Resistant Construction	▪ Basement ▪ Building Fabrics
P8	Flood defence measures	▪ Urban sea defence ▪ Urban river defence ▪ Rural sea defence ▪ Existing rural river defence and drainage schemes ▪ New rural river defence and

Policy Subject	Main Heading	Sub-headings
		drainage schemes
P9	Maintenance and operations	▪ Water level management plans
P10	Monitoring	▪ Schemes ▪ Flood warning systems
P11	Post project evaluation	▪ Management ▪ Funding ▪ Implementation
P12	Research and development in flood protection	▪ Funding ▪ Dissemination
P13	Relationship with Insurers	▪ Legal obligations ▪ Integrated approach
P14	Emergency Response	▪ Lead in time ▪ During and after the event
P15	Review	▪ Policies ▪ Strategy

16.4 Present Practice and Aspirations

This section outlines the outcome of a consultation and document review exercise, to identify Dublin's present practices in policy and strategy, as applied to flood protection.

16.4.1 Consultation

The study team conducted a number of meetings with key consultees on the subject of Policy and Strategy. Those consulted included:

(i) Dublin City Council, GDSDS Study Manager;
(ii) Dublin Port Company, Harbour Master;
(iii) Fingal County Council;
(iv) Dublin City Council, Technical Services;
(v) Department of the Marine;
(vi) Office of Public Works;
(vii) Dublin City Council, Planning.

Some recurring and/or dominant issues that stemmed from these meeting are outlined below:

At the present time, the **institutional framework** for managing Dublin's flood protection is not effective. This does not imply any criticism of the individual authorities and departments who are tasked, in some way, with flood protection; rather that the co-ordination of duties between these agencies is not clearly defined. In particular, the roles of the Department of the Marine and the Office of Public Works appear to overlap, and their respective geographic boundaries of jurisdiction are not firmly established. However, this issue is the subject of a "National Policy Review Group" whose purpose is to clarify the roles of the various authorities. The findings of the Review are due to be released in the summer of 2004.

With respect to the classification of **flood risk areas**, there is a recognised need for varying levels of flood defence according to (economic) priority. The potential depth of flooding is clearly a critical factor in assigning the required level of protection. Although there is clearly a demand for this kind of information, there is also a “nervousness” about public dissemination of flood risk maps.

The integration of flood protection with coastal zone management, through **planning initiatives** is in its infancy. There is, however, a positive will to develop on this and to incorporate flood defence philosophy into the upcoming Fingal County Development Plan and the Dublin City Council Development Plan. In addition to the flood defence function, the Councils are understandably concerned about the social and the other aspects (impacts) of any protection measures that might be proposed.

An **early warning system** is considered to be essential. There is a desire for this to be a regional system that will integrate warning of flooding with respect to both river and sea over the Greater Dublin Metropolitan area.

Further to the consultation exercise described above, a Workshop was held on 5 September 2003 at which flood forecasting, flood warning, and flood risk mapping were debated. Of particular relevance to the subjects of policy and strategy were debates on what were considered to be important flood warning triggers (criteria for having/issuing flood warning), and what would be the purpose and application of flood risk maps. The following condenses the discussions on these particular issues:

The most important flood warning triggers were considered to be:

- loss of life;
- social and personnel disruption:
  - shock - disease
  - stress - pollution
  - evacuation - post flood accommodation;
- property damage and loss of personal possessions;
- commercial/economic loss;
- loss of services and utilities;
- insurance impacts;
- concerns over adequacy of defences;
- traffic disruption.

With regards to the purpose and application of flood risk maps, the Workshop identified:

- as historical record;
- to identify the risk areas;
- to identify the degree of risk, i.e. the probability;
- to show the effectiveness of flood prevention measures;
- to facilitate Planning guidance;
- influence on Insurance cover and premium;
- to facilitate emergency planning including emergency access and evacuation;
- to inform the public on areas of risk and highlight measures to be taken;
- to identify risk to strategic infrastructure;
- to prioritise investment in flood protection;
- to predict areas affected by future extreme events;
- as a design tool;
- as a real-time information source with which to manage response.

Flood risk maps are being produced as part of this project.

16.4.2 Forum and Workshop

A Forum and a Workshop on the subject of Policy and Strategy were held on 2nd and 27th April 2004 respectively. Both gatherings provided valuable input, and specifically to the topics: Planning and Flood Defence.

On the topic of Planning, emphasis was placed on development and the developers and in particular, their responsibilities in respect of demonstrating flood risks, and the steps taken to alleviate them. It was also considered important to understand the effects of development both at the local level and the regional level.

On the topic of Flood Defence, the workshops identified the zoning and prioritisation of flood risk areas as a priority. A close link between flood defence measures and flood warning systems was seen as desirable.

16.5 Policies and Strategies in Place

This section briefly discusses those policies that are currently in place in respect of Dublin's flood protection.

16.5.1 Emergency Response

The guiding document for emergency response in the Dublin Region is the Dublin Metropolitan Region set of Major Emergency Procedures (MEP). First issued in 1999, as the ''Dublin Metropolitian Region, Major Emergency Plan'' the first major amendments were undertaken in 2003. Since then the title of the document has been changed to the DMR Major Emergency Procedures.

The Procedures outline the co-ordinated approach to managing major emergencies by:

An Garda Síochána,
The Health Boards,
The Local Authorities,
The Irish Coast Guard.

The document also details other bodies and agencies, which might be called upon to provide assistance in dealing with major emergencies.

The document defines a Major Emergency as *"any occurrence which causes a threat to the health of the community, disruption of essential services or causes or is likely to cause such numbers of casualties, or damage to property or to the environment as to require special arrangements by An Garda Siochana, the Eastern Health Board, the Local Authorities or IMES"*. As such, the MEP applies to a wide range of possible incidents including chemical spills, fire and nuclear accidents. Flooding is not singled out, but is mentioned specifically in relation to evacuation of the disabled.

The MEP is a necessary "strategy" for dealing with a serious flooding incident. It is probably fair to say that the document is largely focussed on the procedures to be adopted once the emergency has occurred, which is understandable given that many incidents (e.g. explosion) occur without warning. In respect of flooding, however, there is likely to be a foreseeable lead in time to the onset of damage; whilst the MEP advocates the principle that PREVENTION IS BETTER THAN CURE, it does not deal specifically with the lead in time to a given incident. Critical aspects of flood response that are not dealt with in the MEP are that of the public's role in protecting themselves, the prospect of a number of geographically dispersed incident locations and the probability of the evacuation of large numbers of the population from affected areas.

To complete the strategy for flood response, there needs to be further guidance on actions to be taken during the lead in time prior to the flood, on the role and response of the public, on the management of a number of dispersed incidents sites and the actions required after the water has retreated. The executive summary of the Major Emergency Procedures document specifically states that it is a framework document for the Dublin Region and is designed to complement existing operational Plans of the various authorities listed in it. Hence it is important that in addition to a future revision of the MEP containing guidance on the response to flood events that the Local Authorities ensure they have a specific plan to cover all aspects of the response to potential flooding affecting their communities.

16.5.2 GDSDS Policies

The Greater Dublin Strategic Drainage Study (GDSDS) has produced a number of documents referred to as Regional Policies. At the stage of preparing this report, these Regional Policies were available as Consultation Documents. The reports cover the following headings:

- Volume 2 New Development
- Volume 3 Environmental Management
- Volume 4 Inflow, Infiltration and Exfiltration
- Volume 5 Climate Change
- Volume 6 Basements

The scope of these five documents and their relevance to Dublin's coastal and river flooding, is outlined below:

Volume 2 New Development :

The Consultation Document Volume 2, New Development, sets out the following Objectives for drainage planning of new development:

- New development shall be controlled in areas at risk of flooding, erosion and other natural hazards;
- New development shall be controlled in order to reduce the risk of serious danger to human health or the environment;
- New development shall be controlled in accordance with the principles of proper planning and sustainable development;
- New development shall include the provisions and siting of sanitary services;

- New development shall protect and preserve the quality of the environment;

- New development shall control the deposit or disposal of foul sewage and surface run-off, the disposal of sewage and the pollution of waters;

- Existing and new development lands shall be categorised in terms of risk of flooding with appropriate planning controls.

In may be noted that there is some considerable similarity with the tenets of PPG25 as shown in italics in 3.1(C). Given the scope and purpose of the New Development report, it is not surprising that it emphasises the distinction between, and the need to separate, foul and storm drainage. This is an important consideration also for coastal flooding as ultimately it is the storm water drainage system that conveys the residual flood waters away.

The New Development report goes on to suggest policy for Development in Flood Plains:

- Flood risk mapping for the Dublin Region to be produced;

- Planning and Drainage Departments to categorise existing and future development areas in terms of low, medium, high and unacceptable flood risk, and state on Development Plans;

- Planning permission for development in areas of flood risk to be subject to satisfactory Flood Risk Assessment;

This uses a qualitative yard stick for the categorisation of flood risk. This can be compared, in a more approximate way, to the Sequential Test described in PPG25, and shown in Appendix 1.

Volume 3 Environmental Management:

There are a number of Directives that concern drinking water, fish, shellfish waters etc. In time these various Directives will be superseded by the Water Framework Directive (WFD).

The WFD seeks to achieve "good status" for all waters by 2015. The main concern of Volume 3 Environmental Management is that for the WFD to be met, sustainable (urban) drainage systems (SUDS) must implemented.

Drainage systems can have serious impacts on the natural water environment in respect of both storm water run-off and foul sewerage. The former is closely related to coastal flooding, the major factors being water quality (e.g. pollutants from paved areas etc) and volume (e.g. increases due to new development).

The Environment Management report describes policy proposals for Storm Water Drainage, under a number of headings, viz:

1. Sustainable drainage systems (SuDS);
2. River Management and Conservation;

3. Environmental assessment;
4. Drainage management and maintenance systems;
5. Health and Safety;
6. Public awareness.

The first policy, SuDS, entails the application of best environmental practice to mitigate the impact of development on the water environment. This objective could be affected, positively or negatively, by measures put into place for coastal flooding protection (e.g. with respect to inflow) and as such must be recognised in policy P8 Flood Defence Measures.

The second policy proposes that "it should be a requirement that all rivers and streams, comprising as a minimum all tributaries marked on 1 to 50,000 OS mapping, be subject to conservation of their natural channels and a significant riparian strip (typically 10-15m on either side depending on the size of the water course) within any development." Clearly, this is very significant in terms of flood protection, as it could directly affect works that might be located in the "riparian strip". This is a specific requirement that would need to be addressed in any Environmental Assessment (No 3 above) in which the specific circumstances of each case would have to be examined.

The fourth item relates mainly to authorities' internal administration of management and maintenance systems,

The fifth item discusses the minimisation of risks to health and safety, and further, that "such considerations should be an integral part of the planning, design, construction and maintenance phases of these systems". Clearly, these considerations, as a matter of policy, should apply to works and their maintenances, for coastal flood protection. It might also be noted that all works of this kind would be subject to the EC Construction, Design and Management Regulations.

The last item, Public Awareness, is equally applicable to coastal flood protection. It advocates "public awareness and public participation in resource management and conservation generally".

Volume 4 Inflow, Infiltration and Exfiltration:

Of particular relevance to Coastal Flooding is the issue of "inflow", as this can occur when adverse water levels in a river overwhelm the outfall, thus causing backing-up of the system. Infiltration can also occur due to tidally saturated ground, higher tide (sea) levels causing more infiltration.

The report promotes the "application of Best Management Practices" from international experience, so that the following objectives are achieved:

- I/I/E in the region's sewerage systems will be identified and flow quantities estimated;
- Survey and reduction works will be carried out with optimum cost-benefit;
- Specifications and practices for sewerage construction will be imposed to minimise I/I/E;

- Asset management systems will be targeted to minimising I/I/E and its adverse effects on the operation of the sewerage system and the overall environment."

Policy for Coastal Flood Protection should recognise the above listed objectives, particular with regards to identifying possible sources of inflow (e.g. river outfalls) and estimating the flow quantities involved. In essence, this represents another risk to infrastructure through coastal flooding and should be identified as such under Policies P8 Flood Defence Measures, P9 Maintenance and Operations, and P10 Monitoring.

Volume 5 Climate Change:

The report presents a technical overview of climate change and gives recommendations of the relevant issues for drainage engineers. Using the UK Climate Impacts Programme 2002, and a "medium-high" scenario (described as precautionary), the report goes on to discuss the influence of climate change on temperature, rainfall, sea level and groundwater. The report develops these subjects, culminating in recommendations for incorporating climate change issues into drainage design criteria. The report is educational and advisory; it approaches the subject of climate change informatively, and the application of the knowledge so gained in an instructive and directive way, along the lines of a design guide. Its style could be described as "bottom up" and, as such, possibly too specific to be classed as policy or strategy. It does, however, contain some useful recommendations that are, in effect, statements of policy; these are reproduced below:

"Sea level design criteria must take into account that sea levels are forecast to rise for several centuries. Strategic planning of the Dublin area should take an appropriate precautionary position."

"Detailed joint probability analysis should be used for storage and where solutions are very expensive."

"Simple combinations of events for pragmatic assessment of joint occurrence should be used for outline design and inexpensive schemes".

Volume 6 Basements:

The report is principally concerned with the flooding of basements due to sewer capacities being exceeded. It covers aspects of policy that relate to:

- Whether further development should be restricted because of the effect on the drainage system.

- If anti-flooding devices should be retrofitted to the existing basement drainage.

- How the policy should be financed.

The work is a close relation to that of the present project and the specific issue of basements can come into play in respect of the backing-up of sewer outfalls due to high water levels at the outfall. However, this issue is of less significance in cases of land inundation when water levels are in any case, above ground level (and inundation is more of less certain).

This policy on basements is important for certain types of flooding but is less relevant for more severe coastal flooding, being the principal subject of this present report.

As such, the Policy on Basements is not elaborated on further here, but policy concerning flood resistant construction should observe the advice given in this Volume 6 report.

16.6 Summary of Present Practice and Aspirations

From the preceding part of Chapter 4, various elements of present flood protection policy and strategy in Dublin have been outlined. For consistency with the system devised in the last section (Table 16.1) these "elements" (both tangible polices that are in place, and aspirations) have been collated against the relevant Policy Headings 1 to 15. The outcome is shown in Table 16.3.

Table 16.3 - Outline of Present Situation

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
P1	***Institutional and legal frameworks*** Government; Authorities; Developers; Owners.	Present Practice There are three kinds of "player" involved in Dublin's flood defence. These are: the authorities (Fingal CC and DCC); the Office of Public Works (OPW) and the Department of the Marine. Department of the Marine deal with serious problems of coastal defence, both capital schemes and maintenance projects. Technically, their responsibilities extend up estuaries to the tidal limit. The OPW is a central government department being part of the Department of Environment, Heritage and Local Government. Until 1995, the work of the OPW was entirely concerned with arterial drainage. Whereas previously the work of OPW was only considered on a catchment wide basis, after 1995 there was greater freedom to examine more localised schemes. Despite these demarcations in duties, there is very little in the way of any formalised institutional legal framework. The division of responsibilities between the Department of the Marine and OPW, It is anticipated will be set out in the forthcoming OPW National Review of Strategy.
		Aspirations There is a need to be mindful of the obligations of the authorities with respect to those of the owners, especially where there is a single major benefactor involved.

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
P2	**Communications** Public awareness; Consultation.	Present Practice The GDSDS Draft Regional Policy on Environmental Management advocates public awareness and public participation in resource and management of conservation. This project has led to the creation of a Web site on coastal flooding.
		Aspirations **It is h**oped **that lines of communication will** be improved with better institutional framework**.** It is intended that local Flood partnerships be established as part of the EU SAFER project.
P3	**Regional perspective** Shoreline Management Plans; Catchment Management Plans; Early Warning Systems.	Present Practice The present institutional structure does not favour the promotion of regional plans (SMPs and CMPs).
		Aspirations Early warning system sought on a regional basis.
P4	**Climate change** Rainfall; Sea level rise.	Present Practice GDSDS advises impacts on temperature, rainfall, sea level rise and groundwater. It offers some policy statements but advice is mainly directed at drainage matters.
		Aspirations To extend the present situation to include coastal flooding concerns.

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
P5	**Planning for development** Importance of flood plains; Precautionary principle; Sequential test – risk based approach.	**Present Practice** Dublin has expanded very quickly due to the "Celtic Tiger" economy. The GDSDS Draft Regional Policy on New Development advises, in respect of drainage: • Control of new development in order to reduce risks to people and the environment; • new development lands shall be categorised in terms of risks of flooding with appropriate planning controls; • delineation of the Dublin Region into flood risk maps, and the categorisation of existing and future development area in terms of low, medium, high and unacceptable flood risk.
		Aspirations The Fingal Development Plan, scheduled for June 2004, will include in some form, coastal zone management considerations, currently being devised with OPW. Information of flood risks can influence what is regarded as permitted development. The new City Development Plan comes into effect on Monday 14th March 2005. The plan will be available to view at the Planning Public Counter in Civic Offices from 14th March during normal opening hours and the Written Statement can be viewed on the website. See P6 and P7. The plan itself has a "shelf life" of six years but it does not itself include time parameters (strategy).

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
P6	**Flood warning systems** Flood forecasting; Flood warning.	**Present Practice** The interim flood forecasting systems is in operation, operating on Met Office forecasts and linked into real time monitoring.
		Aspirations Some areas are impossible to improve so there will always be a finite risk. An early warning system is therefore considered necessary. It would make sense for the needs of a coastal flooding early warning system to be combined with those of the GDSDS. Also the early warning system should operate on a regional basis. Early warning are sought by Dublin Port Company so that they can alert their tenants who in turn can take steps to reduce damages. Early warning system sought on a regional basis.
P7	**Flood Resistant Construction**	**Present Practice** This subject is discussed in the GDSDS Volume 6 in respect of the protection to basements.
		Aspirations Non identified
P8	**Flood defence measures** Urban sea defence; Urban river defence; Rural sea defence; Existing rural river defence and drainage schemes; New rural river defence and drainage schemes.	**Present Practice** Present underlying policy implicit in GDSDS is to provide protection for areas at risk in the case of the 1 in 100 year return period event being exceeded. The GDSDS Draft Regional Policy on Environmental Management recognises the requirement to comply with the Water Framework Directive. The most significant policy arising from this makes it a requirement that all rivers and tributaries be subject to conservation of their natural channels and significant riparian strip (10 – 15 m). Health and safety considerations are also proposed in the policy document.

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
		Aspirations Depth of flooding is a critical factor but this is not being elaborated upon at this time. Various levels of defence will be needed to cater for the different land uses. The National Coast Protection Study (currently underway) is to develop a more strategic way of tackling the issue of prioritisation (in Ireland).
P9	**Maintenance and operations** Water level management plans.	Present Practice The GDSDS Draft Regional Policy on Environmental Management recognises the requirement to comply with the Water Framework Directive. The most significant policy arising from this makes it a requirement that all rivers and tributaries be subject to conservation of their natural channels and significant riparian strip (10 – 15 m). Health and safety considerations are also proposed in the policy document.
		Aspirations Non identified
P10	**Monitoring** Schemes; Flood warning systems.	Present Practice A number of monitoring systems are in place but policy on monitoring is lacking.
		Aspirations To extend monitoring to include flood warning.
P11	**Post project evaluation**	Present Practice Non identified
		Aspirations Non identified
P12	**Research and development in flood protection** Funding; Dissemination.	Present Practice Dublin City Council are currently engaged in the EU funded Safer project.

Policy Subject	Main Heading and Sub-headings	Responses concerning present situation and future needs
		Aspirations Non identified
P13	**Relationship with Insurers** Legal obligations Integrated approach	Present Practice Non identified.
		Aspirations The policy Forum and Workshop identified insurance as a key issue that needed to be integrated with flood risk management. The possible outcomes are limited as the Irish Insurance Federation, the umbrella organisation for the industry has mainly a consultative/information role with its members – each Insurance Company has its own business drivers and criteria for determining its acceptable level of risk for individual cases. This is at odds with the Aspiration for an integrated approach.
P14	**Emergency Response**	Present Practice The Emergency Response Procedures contained in The Dublin Metropolitan Region MEP provide essential guidance on procedures and lines of communication in the event of a disaster having occurred.
		Aspirations To extend emergency response procedures to include measures taken during the lead in time to a flood. In accordance with best practice to undertake, develop and maintain comprehensive flood specific emergency planning.
P15	**Review** Policies; Strategy.	Present practice Non identified.
		Aspirations Non identified.

16.7 Dublin's Needs

Table 16.3 in the last section summarises the level of attention given to various aspects of flood management policy in Dublin. This present practice can be grouped into four categories:

Category 0: where there is no defined policy or documentation;

Category 1: where there is some form of policy but it not formally documented as such;

Category 2: where there is some form of documented policy;

Category 3: where there is comprehensive policy that is fully documented.

Table 16.4 shows the present status of the policies according to this categorisation, as perceived by the study team from the available information, and as discussed at the Forum and Workshop on Policy and Strategy.

Table 16.4 - Categorisation of Present Policies

Category 0: P12 Research and Development in Flood Protection P15 Review	**Category 1:** P1 Institutional and Legal Frameworks P3 Regional Perspective P9 Maintenance and Operations P10 Monitoring P11 Post Project Evaluation P13 Relationship with Insurers
Category 2: P2 Communications P4 Climate Change P5 Planning for Development P6 Flood Warning Systems P7 Flood Resistant Construction P8 Flood Defence Measures P14 Emergency Response	**Category 3:**

It may be recalled that the Strategy for Flood and Coastal Defence in England and Wales cited "flood warning systems" as a top priority, accompanied by coastal and flood defences. It is significant that these priority areas only appear as Category 2 in the above table, albeit, they are in the highest category achieved according to the system devised here. Clearly, it is essential that these important matters of policy be formally documented, which would thus place them in Category 3.

Two of the issues, as listed in Category 0, were not noted as being existing policy initiatives. This is also significant, because all the issues, though not of the same priority as those referenced in Category 2, are important elements of a strategic flood management regime. It therefore follows that all the policy items (1 to 15) should receive no less than a minimum level of formal recognition through documentation (i.e. minimum Category 2).

The relationship with insurers (P13) has been discussed at workshops but there is apparently no formal linkage with the flood defence community. US experience in this matter has demonstrated quantifiable benefits in having an integrated and transparent approach. Having an integrated approach means that the flood risks are known and catered for, both through the provision of practical advice on flood protection, and the imposition of premiums that accord with the risks. This item attracted considerable interest at the Forum and Workshop, and should be dealt with at Category 3 level.

16.8 Recommendations for Policy

16.8.1 Preamble

This section describes the criteria against which recommendations for policy are given (Section 6.2).

As noted in the introduction, this report is not a policy document, although the recommendations may be adopted and adapted for the purpose of formalising policies. It is useful, however, to consider the likely scope of each policy item and this is done in the next section of the report.

One of the fundamental policies to be concluded on is that of Institutional Framework, and so at this point it is worth reconsidering the question "who's policy"? This was the subject of examination by the National Review Group. The work of that group has clarified the roles and responsibilities of the various "players" in Dublin's flood defence strategy, that there are basically three levels of control, i.e.:

- Central government – through the direct powers of the appropriate Department/s;
- Government agencies with technical and regulatory remits – e.g. OPW and the Department of the Marine;
- Authorities with executive powers – DCC, Fingal County Council.

Other groups, e.g. Dublin Metropolitan Region, may also initiate and promote policy, although these are likely to be spearheaded by one or other of the agencies listed above.

In making recommendations it is important to identify the likely "owner" of each element of Policy. This means the body who proposes and intends to adopt the policy. They are likely to be responsible for publishing the policy but are not necessarily obliged to carry it out themselves.

It can be concluded from Chapter 5 that there is a need to document and disseminate policy. Indeed, documentation must surely be a prerequisite of any policy. The recommendations indicate possible vehicles for dissemination.

16.8.2 Recommendations

A tabular format is used to summarise, discuss and advise on each of the identified policy items.

Table 16.5 - Outline Recommendations for Policy

<table>
<tr><td>P1</td><td colspan="3">Institutional and Legal Frameworks</td></tr>
<tr><td>Current Category 1</td><td colspan="3">Current Status
Has been defined in the OPW report on the Flood Policy Review Group.</td></tr>
<tr><td colspan="4">Discussion
At the time of writing, the National Review Group Report was not available. However, it is expected to advise on the roles of government, their agencies, OPW and Department of the Marine, and local authorities.</td></tr>
<tr><td colspan="2">Advised Scope
The proposed framework should include, but not be limited to:
▪ Roles and duties and geographic remits of the various agencies, in respect of flood defence management (all P2 to P15);
▪ lines of communication;
▪ legal obligations and advised relevant Acts;
▪ mechanisms for funding flood protection works;
▪ criteria for funding flood protection works.</td><td>Advised Owner
Central Government</td><td>Advised Vehicle
Policy Note on Flood Defence</td></tr>
</table>

<table>
<tr><td>P2</td><td colspan="3">Communications</td></tr>
<tr><td>Current Category 2</td><td colspan="3">Current Status
The present project has engaged public consultation and introduced the setting up of an information website.</td></tr>
<tr><td colspan="4">Discussion
The implementation of communications has outpaced the formalisation of policy on it. Whilst this might seem to obviate the need to “transcend” to a higher level of consideration, there is still a very real need to establish policy, especially when considering the emotive issue of the publication of flood risk maps. Moreover, there is a close connection between three of the policy items which can be approached collectively, viz:
▪ P1 communications
▪ P6 Flood Warning Systems
▪ P13 Relationship with Insurers
US experience has demonstrated advantages in having a transparent approach, and this is the premise for the advice given below.
The most effective communications are those that can be related to from a personal perspective. This implies that the local community and the local authorities are the most effective correspondents. It follows that the local authorities should set policy for Communications within the guidelines of P1.</td></tr>
<tr><td colspan="2">Advised Scope
Policy on communications should advise on:
▪ Clear lines of communication between the various interested agencies, pending the outcome from the National Review Group;
▪ Dissemination of flood risk maps (supposed universally available);
▪ Press and other media; communicating with customers;
▪ Pledges regarding the creation and upkeep of public communication systems (web, leaflets, etc);
▪ Provisions for public feedback/response and timing pledges;
▪ Protocol governing the management of flood warnings; i.e. who is responsible (not advice on specific details of the system itself);</td><td>Advised Owner
DCC, Fingal County Council.

Insurers</td><td>Advised Vehicle
Joint Policy note.</td></tr>
</table>

▪ Scope and mechanism for dialogue with insurance industry, in particular, advice to customers.		

P3	**Regional Perspective**	
Current Category 1	Current Status At the time of writing there is no formal initiative or requirement to undertake Shoreline Management Plans (SMPs) or Catchment Management Plans (CMPs).	
Discussion Because SMPs and CMPs are dictated by natural, rather than jurisdictional, boundaries it would be inappropriate for policy regarding these initiatives to be driven by local authorities whose boundaries do not necessarily accord with geomorphological features. That does not mean to say that these authorities cannot take an executive role in the implementation of such plans, only that the directives for the plans, including geographical demarcation, should be envisaged from a national/catchment perspective.		
Advised Scope Policy on the preparation of SMPs and CMPs should include: ▪ Geographical demarcation; ▪ Purpose of preparing a plan; ▪ Objectives of a given plan; ▪ Technical scope; ▪ Legal status of SMPs and CMPs; ▪ Provisions for funding plans; ▪ Uptake (steps after plan preparation).	Advised Owner Central Government	Advised Vehicle Advisory note on the Preparation of SMPs Advisory note on the Preparation of CMPs

P4	**Climate Change**	
Current Category 2	Current Status GDSDS Regional Policies – Volume 5 Climate Change	
Discussion The GDSDS report provides useful information on climate change and policy advice concerning its recognition in strategic planning. It goes on to advise on the levels of analytical sophistication that different situations warrant. As a policy note it is possibly rather detailed in its treatment of models and methods and focuses particularly, and understandably, on matters concerning drainage. Although much of the GDSDS document is generic (to Ireland), in parts it is regionally focussed. More policy guidance is needed on the impacts of climate change on sea level and, hence in turn, on flood defence levels.		
Advised Scope Essentially this is a technical issue, with long term economic consequences. To plan ahead for (or avoid) those consequences there needs to be a policy regarding the recognition of sea level rise in flood defence management and design. The uptake for this advice would be realised particularly in Planning for Development (P5), and Flood Defence Measures (P8). A policy on Climate Change (for Flood Protection) could be generic to Ireland, which would suggest that it should be developed on a national, governmental, basis. It would cover: - advised latest estimates for relative sea level rise (which will vary around the Irish coast); - overview of groundwater and fluvial implications (which will refer to the need for regional studies, see P3); - recognition of climate change in respect of sustainable development (planning); - recognition of climate change in respect of economic evaluation of flood protection schemes;	Advised Owner Central Government	Advised Vehicle Advisory Note.

- recognition of climate change in respect of engineering (design) (adaptable construction, long term risks, uncertainty – also in relation to storminess).		

P5	**Planning for Development**	
Current Category 2	Current Status The GDSDS includes new development policies in respect of drainage. Intended inclusion of flood risk considerations in forthcoming Development Plans by DCC and Fingal County Council.	
Discussion This subject can and should be approached at two generic levels. There is a need for national guidance (similar to the UK's PPG 25, and as well as recognition of flood risks in individual development plans. Whilst the subject is included in the GDSDS Draft Regional Policy on New Development, the guidance given, whilst being directed principally at drainage, is too qualitative for coastal flooding purposes and needs to be developed further for this purpose.		
Advised Scope At a national level there is a need to advise policy on the relationship between flood risk and development. This might follow similar lines to the UK's PPG 25, the tenets of which are: ▪ planning control in respect of flood risk; ▪ recognition of uncertainty; ▪ use of precautionary principle; ▪ obligations of authorities and developers (e.g. the cases for which the developer has to fund study, designm or construction works) ; ▪ recognition of value of flood plains; ▪ recognition of regional perspective (see P3); ▪ adoption of a "sequential test" (see Appendix 1); Planning policy needs to recognise local and regional perspectives. At a local level, DCC and Fingal County Council should embody the principles listed above into Development Plans. In particular, this will categorise zones according to the flood risks including (by the same token) the nature of the existing or planned development. This, implies that the physical risks of flooding are known by way of flood risk maps. To apply the sequential test it is an implicit requirement to have the risk of flooding broadly categorised into three probabilities, say.	Advised Owner Central Government DCC Fingal County Council	Advised Vehicle Policy Note Development Plans

P6	**Flood Warning Systems**
Current Category 2	Current Status An interim flood forecasting system has been installed as part of the Flood Protection Project . This uses Met Office forecasts for input and is verified against real time tide observations.

<table>
<tr><td colspan="3">Discussion
Considerable loss can be avoided by taking action before a flood has occurred. In this respect, the study group is taking a pro-active approach in setting up a flood warning system. Consultation with relevant agencies (see Section 4.1) indicated a wish for a regional flood warning system that would encompass risks of flooding both from the sea and from rivers. This aspiration is in itself a policy, if formally documented as such.
The Workshop of 5 September 2003 also identified the relationship between Flood Warnings and Insurance as being important, as well as the importance of flood risk maps in the identification of risks to people and strategic infrastructure. Other human impacts resulting from floods, as discussed at the workshop (shock, stress, evacuation, etc) are generally included in the ERP.
Whist it is not the purpose of this report to state policies, it can be noted that the overwhelming objective of flood warning systems is the avoidance of casualties.
The direct connection between flood warning and loss mitigation implies, as noted above, that the Insurance Industry might be interested in participating in policy development. This also applies to policy P7, Flood Resistant Construction. Thus, in this way, a form of integrated approach can be adopted in which (flood) insurance premiums can be determined from an informed perspective.</td></tr>
<tr><td>Advised Scope
As mentioned in the discussion above, there would be merit in participation of the Insurance Industry in preparation of this and related policies (see P7 and P13).
Policy on Flood Warning should include:
▪ clearly defined geographic area of operation;
▪ commitment to advance warnings to the public;
▪ commitment to funding the setting up and maintenance of flood warnings;
▪ protocol for the issuing of flood warnings (e.g. varying alert levels);
▪ advice to public on self protection and minimising loss;
▪ advice to public on emergency measures (related to ERP);
▪ advice to public on post flood measures and provisions;
▪ liaison with Insurers.</td><td>Advised Owner
DCC/
Fingal County Council

Insurance Industry</td><td>Advised Vehicle
Published Policy Statement</td></tr>
</table>

<table>
<tr><td>P7</td><td colspan="3">Flood Resistant Structures</td></tr>
<tr><td>Current Category 2</td><td colspan="3">Current Status
The subject is referred to in the GDSDS Draft Regional Policy Document in respect of protection to basements.</td></tr>
<tr><td colspan="4">Discussion
From a policy perspective, perhaps the most relevant outlet for advice on Flood Resistant Structures is in the authorities' Planning and Building Regulations. Technical advice on the installation of “flood resistance” might be the subject of an advisory note or public information leaflet, but this would not be policy – however, the commitment to provide this information would be.
As with Flood Warning, there is potentially considerable merit in collaboration with the Insurance Industry.</td></tr>
<tr><td colspan="2">Advised Scope
Policy on Flood Resistant Structures could include:
▪ commitment to public education;
▪ planning and building restrictions ;
▪ grants (if any) for flood proofing;
▪ insurance premium bonuses for installation of flood proofing (see also P13).</td><td>Advised Owner
DCC/
Fingal County Council
Insurance Industry</td><td>Advised Vehicle

Planning and Building Regulations

Advisory Note</td></tr>
</table>

<table>
<tr><th>P8</th><th colspan="2">Flood Defence Measures</th></tr>
<tr><td>Current
Category
2</td><td colspan="2">Current Status
This is possibly the biggest subject listed, and the one with potentially the largest financial implications. At present local policy (GDSDS) recognises and promotes defence where there is a probability of flooding in excess of 1 in 100 years. This is a very simplistic approach that must be upgraded in recognition of varying levels of risk.

It is anticipated that the protocol for managing the strategy for flood protection (i.e. the institutional framework) may be set out in the forthcoming National Policy Review.

The GDSDS Draft Policy on Environmental Management advises on conservation measures with a view to compliance with the Water Framework Directive.</td></tr>
<tr><td colspan="3">Discussion
Policy on Flood Defence Measures must start fundamentally with Government policy. Pending the outcome of the National Review, the role of the OPW and the Department of the Marine can be compared to that off the Water Boards in the Netherlands i.e. that of regulation and administration.

Empowerment for the implementation of capital works must be read in conjunction with maintenance works. Common policy (UK and NL) is for capital works to be part funded by the state, having proved national economic benefit, whilst maintenance works (see P9) are funded out of local authority budgets.

The motivation for flood defence works should come from the local authorities (DCC and Fingal CC) based on the knowledge of their own flood risk areas and state of defences (see P10). This implies the maintaining of a programme of flood defence priorities, tied into a planned expenditure stream (i.e. a Strategy).</td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>Advised Scope
Government policy on Flood Protection Measures should advise on the empowerment of its departments (OPW and Department of the Marine), and of local authorities. The protocol so defined should include:
▪ policy in respect of protection of life (first priority) and property;
▪ policy on responsibilities of the agencies and property owners too; possibly this extends to the concept of “flood partnerships”;
▪ indicative priorities for protection (e.g. existing urban development in high risk areas,…….new development in low risk areas);
▪ availability and criteria for State grant aid (including conditions regarding: technical correctness, economic robustness, environmental acceptability); pledged budgets should include R+D ventures (see P12);
▪ policy to encourage the promotion and undertaking of works within a broad geographic context (catchment, coastal cell);
▪ compliance with EC Construction, Design and Management Regulations;
▪ policy on protection of the environment including compliance with Water Framework;
▪ Policy on impact on drainage, inflow and infiltration.</td><td>Advised Owner
Central Government

OPW and Department of the Marine.

DCC and Fingal County Council</td><td>Advised Vehicle
Government National Policy on Flood Protection

Advisory note of Regional Plans and Policies</td></tr>
</table>

Government departments tasked with managing the national flood defence programme should establish policies, founded on Department principles that are region specific. These should include the observance of regional perspectives (SMPs and CMPs), technical (including economic and environmental) audit, and review of the operating authorities' exercise of executives powers. As the bodies with executive powers to carry out works, the authorities, DCC and Fingal CC, may need to define no further policies beyond those which are incumbent upon them by virtue of Government policy and empowerment through statutory legislation. However, by the same token, these authorities need to develop strategies to put these policies into effect.		

P9	**Maintenance and Operations**	
Current Category 1	Current Status The authorities are, of course, active in the undertaking of flood defence maintenance, but documented policy on maintenance is not in evidence. However, the GDSDS Draft Policy on Environmental Management advises on conservation measures with a view to compliance with the Water Framework Directive.	
Discussion As discussed above, the empowerment and obligation to undertake maintenance works will largely rest with the authorities, DCC and Fingal CC, and as such they are probably in the best position to develop policy in this subject. There is "cross talk" between the carrying out of maintenance, other operational functions (e.g. flood warning systems) and protection of the environment. These three elements can be brought together in some form of maintenance plan (e.g. water level management plan). The initiative to prepare such plans, and their scope would be the policy.		
Advised Scope Policy to advise the preparation of Regional Maintenance Programme, the scope of which would include: ▪ geographic scope; ▪ time frame; ▪ scope to include flood defence structures, flood warning systems, records and archives, monitoring; ▪ liaison with OPW/Dept of the Marine ▪ policy on impact on drainage, inflow and infiltration; ▪ policy on protection of the environment and compliance with Water Framework Directive..	Advised Owner DCC and Fingal CC	Advised Vehicle Advisory Note

P10	**Monitoring**
Current Category 1	Current Status Various monitoring programmes are in being, however, policy on monitoring does not appear to documented.
Discussion ▪ Policy on Monitoring can be included within the Advisory Note referred to in P9.	

Advised Scope	Advised Owner	Advised Vehicle
Important details of policy that should be included in line with that for P9 are: ▪ scope of monitoring to include, flood defence structures, flood warning systems, relevant environmental parameters; ▪ policy should allow for update every five years.	DCC and Fingal CC	Advisory note, included within that for P9

P11	**Post Project Evaluation**
Current Category 1	Current Status This would appear to be a new venture, both in terms of policy and its actioning.
Discussion This item must be distinguished from Review, the latter dealing with Review of Authorities' Strategies . Generally there are two main motivations for carrying out Post Project Evaluations: (i) to assess whether a particular scheme was good value for money, and (ii) to explore ways of improving performance next time around. As value for money can relate both to grant aid (e.g. central Government or EC funding) or local revenue (e.g. DCC or Fingal) it follows that both levels of control have vested interests in carrying out post project evaluations. Similarly, both stand to gain from (ii) above, in the future. Possibly, therefore, policy on post project evaluation should be set and administered by OPW or Department of the Marine. In this way, not only would the performances of the consultant and contractor be the subject of evaluation, but also the executive authority and the funding process that led to the realisation of the scheme.	

Advised Scope	Advised Owner	Advised Vehicle
As the advice on Post Project Evaluation would be tied into funding and public works, it could be made a statutory requirement of grant aided schemes . As a policy it should include: • when a PPE is/is not required, ie it may not be obligatory for capital projects less than € 500,000; • Advise the stages of the PPE; • Advise the performance indicators; • Advise the dissemination of findings.	OPW or Department of the Marine	Statutory requirement as part of grant aiding policy.

P12	**Research and Development in Flood Protection**
Current Category 0	Current Status DCC are currently engaged on the EU Funded research project, SAFER, however R+D would appear to be new venture in respect of "policy setting".
Discussion In the UK, DEFRA have set out a comprehensive programme of research, organised into several thematic groups, all supported by a budgeted expenditure stream. Policy for Ireland's R+D on flood protection and management could be written along similar lines but with a budget that aligns with its general expenditure in this field, whilst taking on board UK R+D outputs and international practice.	

Advised Scope	Advised Owner	Advised Vehicle
Policy on R+D could include: • Declaration on budget; • Allocation of budget; • Prioritisation; • 5 year programme.	~~Government~~ ~~Ministry~~Central Government	Component of Government National Policy on Flood Protection.

P13	**Relationship with Insurers**
Current Category 1	Current Status Although there is no formal policy regarding the relationship with Insurers, the issue was raised at the Forum and Workshop and in consultation.

Discussion The value of having an integrated approach on certain issue including Flood Risks has been demonstrated by US experience. Advice concerning the areas of co-operation with the Insurance Industry have been outlined in : ▪ P7 Flood Resistant Structures ▪ P6 Flood Warning Systems The will to develop this approach stems basically from a policy of a transparent approach in which flood risks are understood and are available to those concerned: landowners, the authorities, the Insurance Industry. In this way, practical measures to counter the risk (of financial loss) can be put into effect and recognised in the assessment insurance premiums.		
Advised Scope See P6 and P7.	Advised Owner See P6 and P7.	Advised Vehicle See P6 and P7.

P14	**Emergency Response**	
Current Category 2	Current Status The Major Emergency Procedures produced by Dublin Metropolitan Region provides a first rate plan for the actions to be followed in the event of a major disaster. The advice given could apply in the event of a flood, however, the procedures relate essentially to the period following a disaster, which may be unexpected (as, for example in the case of an explosion).	
Discussion Policy on Emergency Response can be quite straight forward; Simply, there's got to be one. Whereas the current major emergency plan deals mainly with the aftermath of a disaster, new or appended guidelines would include the precautions that can be taken by homeowners and business to pre-emptively mitigate losses.		
Advised Scope This policy item should embody the guidance of the MEP. As a regional perspective is sought, consideration might be given to the policy document being prepared on behalf of Dublin Metropolitan Region. It is, however, supposed that it would be led and promoted by DCC/Fingal County Councils. The policy (not the plan) would include • Lead in time measures to be taken • Warning systems • Dealing with the aftermath.	Advised Owner Local Authorities within the Dublin Metropolitan Region	Advised Vehicle Policy Note and addendum to the Major Emergency Procedures. Local Authority Flood Incident Emergency Planning.

P15	**Review**	
Current Category 0	Current Status This last subject refers to the Review of Authorities Strategies for Flood Protection by the regulating departments OPW and Department of the Marine. There is no formal policy covering this at present.	
Discussion This form of review would take place on a time incremental basis. The purpose of it would be to check the progress and performance of the Authorities' Flood Defence Strategies so they can be advised on changes or improvements to Strategy updates. It would be logical for such Reviews to occur on a five yearly cycle, timed to take onboard the findings of the five yearly monitoring programmes.		
Advised Scope As a policy to carry out Reviews, the OPW/Department of the	Advised Owner OPW/Department of the	Advised Vehicle Advisory Note.

Marine would advise the Authorities on the scope and relevance of the Review process which might include: ▪ Frequency (five years); ▪ Performance indicators; ▪ Planned and actual expenditures, both in terms of capital and maintenance works; ▪ Flood events and responses to them.	Marine	

17 FLOOD ALLEVIATION OPTIONS

17.1 Introduction

17.1.1 General

Figures referred to in this section are included in Appendix Q

In chapter 15 it has been described how the study coastline has been split into different lengths for the Flood Risk Analysis. Using the results of this analysis, locations which are not currently defended to an acceptable level have been identified. Within this section a range of options has been developed for each of these locations that will reduce the risk from coastal flooding and improve the standard of protection to the hinterland against events with a return period of up to and including that of a 200 year condition. The nature of the options varies from location to location and each has been developed to take into account the nature of the existing coastline and the level of the risk. In addition each option is developed against specific design criteria relevant to that location and this is described in more detail below and in the description of each option.

The options that have been considered at this stage are based on a visual inspection of the location and the topographic survey undertaken as part of this study. No site investigation work has been undertaken, whether geotechnical or environmental. In addition whilst a general overview of the structural condition of all the defences has been made as part of the condition inspection survey, some locations may require a more detailed structural investigation before the options can be finalised or carried through to detailed design. Whilst inspection work is sufficient for the level of detail required at this stage it is recommended that prior to implementation of any of these options a detailed geotechnical, environmental and structural investigation of appropriate locations are undertaken.

The capital costs, stated for each option that follows in section 17.2 include VAT. However, they exclude, site investigation, professional fees, site supervision and any land compensation issues. In the summary table that follows at the end of the section, Table 17.2, an allowance based on percentage of the capital costs has been made for these items and a summary of the total project costs also given.

17.1.2 Other Projects

Marine flooding via the Royal Canal flood pathway remains the main flooding threat to the East Wall area. Historically, the Royal Canal was extensively used by sea traffic which entered from the Liffey through a system of navigation locks into the inland at Spencer Dock. With the disappearance of this trade In the last century the sea lock gates, which never had a coastal flooding function, fell into disrepair and were later removed, so that now the water level in the canal is completely tidal as far as the 1st lock at Newcomben Bridge.

Since the 1st February 2002 flooding two initiatives have been promoted to:

- Provide flood defences
- Restore the navigation capabilities of the sea lock as part of a general redevelopment of Spencer Dock / Docklands area.

The four agencies/ authorities:

- Dublin Docklands Development Authority
- Waterways Ireland
- Ianrod Eireann
- Dublin City Council

which are variously responsible for landowner, flood defences, navigation, development and other functions have agreed to jointly promote a number of related initiatives covering:

- temporary flood defence;
- permanent flood defence and
- restoration of navigation.

Currently the DDDA acting as lead partner has let a contract to seal off the Royal Canal with a temporary coffer dam near its junction with the Liffey and thereby provide immediate flood defence.

While the temporary dam is in place proposals will be developed and implemented for the provision of permanent flood gates and defences and for the permanent restoration of the sea lock.

17.1.3 Assessment Criteria

Design Criteria

All new options presented below have been designed to provide a standard of protection against events up to an including a **1:200 year event**. This criteria has been deemed acceptable in light of the flood risk assessment work. Through discussions with the client it has been deemed acceptable to consider a **design life up to 2031** which is a strategic milestone being considered for a number of other projects within the area including the GDSDS. Therefore the 200 year design event has been considered to this time scale and appropriate allowances for sea level rise included at set out in chapter 8.

As discussed in chapter 15, where necessary appropriate combinations of various parameters have been considered to provide a joint probability combined 200 year event. These combinations have been set out in chapter 15 for both tidal and wave combination to be used around the coast and also for tidal and fluvial discharge combination to be used in the tidal reaches of the rivers.

Details of the actual parameters used in the design process are presented in chapter 15.2 to 15 .5 and the appropriate joint probability combinations presented in tables 15.6 and 15.7 for the coastal regions and intertidal river reaches respectively. A brief summary of the design philosophy is set out below.

Coastal Regions

Tide Levels:

	200 year Level (Now)	200 year level (2031)
Dublin Port Lighthouse –	3.13mODM	3.25mODM
North of Howth –	3.27mODM	3.39mODM

At all other locations tide levels were interpolated around the coastline as appropriate using the results of the FINEL tidal hydrodynamic modelling.

Wave Climate:

Wave conditions presented in Table 15.3 have been used for design in appropriate combinations set out in Table 15.6 and below.

- Structural design – considered the range of scenarios in Table 15.6.
- Flood defence design, i.e. appropriate crest level of defences – considered scenario 1 in Table 15.6.

Acceptable Overtopping

Various limits for acceptable overtopping have been considered around the coastline depending on the location and option being considered. The general values considered include:

- Road or infrastructure directly adjacent to defence - <0.1 l/s/m.
- Road or infrastructure >20m back from defence - <0.5 l/s/m.

Intertidal Regions

For the intertidal regions of the rivers, options have been design to withstand only the high tidally dominated scenarios as presented in Table 15.7, Chapter 15. The reasons for this have been set out in that chapter. Therefore for design of tidal defence options within the tidal reaches, scenario 1 has been considered as it provides the most extreme levels for the tidally dominated scenarios.

17.2 Principal Findings of Asset Database and Emergency Works Report

17.2.1 General

This section of the report summarises the early findings of the Asset Database. In particular it considers the locations where some form of urgent action or emergency works might be required prior to completion of the overall study and the implementation of any recommended long term risk reduction measures. These locations were identified through our familiarisation of the project area, through discussions with DCC and FCC staff and through use of the condition survey asset database developed as part of this project. In response to these finding an emergency works report was completed, which recommended minor works at a number of locations. These options have now either been incorporated within the long term options or been superseded by them.

17.2.2 Identification of Database Actions and Urgent Works

Using the completed database an initial assessment of the condition of the defences across the project area was made and preliminary actions and priorities identified for those areas. A comprehensive table of actions was created from the database, by requesting a report for these actions and sorting them by area. The table is presented in Appendix E1 and sets out an initial assessment of actions and their priorities. This was used as a basis for identifying the main areas where urgent actions and emergency works might be required and was supplemented and enhance through information obtained from other sources. The use of the database to create the table clearly demonstrates the power of the database as a management tool.

The table presented in Appendix E1, contains a considerable number of actions, with varying degrees of priority. Each action identified was not investigated in detail as part of the emergency works report or now, instead only those listed with priority as “urgent” and using judgement a number of those with priority “high”, were considered for further evaluation.

Those which were considered further were broken into two categories, which included flood risk and other. “Flood Risk” considered those locations where there could be a risk to people and property should there be a repeat of an event similar in magnitude to the February 2002 event. “Other”, considered other issues such as poor condition of non flood defence structures, coastal erosion, health and safety etc and generally consisted of locations while not posing a flood risk, should be brought to DCC & FCC’s attention. In some cases both categories may apply.

<u>Locations</u>

1. River Dodder Central Defence Code DDC LA1, 2 3 & 4, Figure 1.7 & Appendix E2.

These defence units are located on the left bank of the Dodder just upstream of Ringsend Bridge and over the location of the dog track, see Figure 1.7 and Appendix E2, Photographs E2.14, E2.15, E2.16 & E2.17. There are several actions highlighted and are mainly **flood risk** in nature. They include investigating the protection offered by a new concrete retaining wall immediately adjacent to the bridge and at the location where water escaped into the building site and then the South Lotts area. In particular the connection with the adjacent earth, as a low spot still looks to exist and it is possible that water could still escape through this location. Also an action has been flagged to investigate the stability of the earth bank on top of the masonry wall and generally its integrity as a flood defence structure. All of these have now been investigated as part of the detailed flood risk assessment work and risk reduction options have been developed and are presented in section 17.4.7.

2. River Liffey Defence Code LFQ LC5 & LFQ RB 9, Figure 1.8 & Appendix E2

The action at both these locations is to investigate the toe detail of the quay wall following the identification of a series of cracks in the wall near the low water mark, see Figure 1.8 for location and Appendix E2, Photographs E2.18 and E2.19. These actions are **not firstly flood risk** in nature but are more integrity and health and safety in nature. That said should failure occur a gap could be left in the quay wall posing a flood risk. Even at low water the toe detail can not be seen. Therefore it is suggested that the action should firstly involve a detailed desk exercise to obtain any archive data on the details of these quays, and pending the findings of that a more detailed site investigation and structural integrity survey.

3. Sandymount Strand Defence Code SMS OA1, Figure 1.7 & Appendix E2

This defence unit consists of the location at Merrion Gates, where the water escaped through the access gap from the beach and onto the nearby road and dart line, see Figure 1.7 and Appendix E2, Photograph E2.20. The action is due to **flood risk** and in particular the risk from future events and the possibility of temporary defences Following the February event, the gap was sealed with metre cubed sand bags. However, during subsequent visits to that location it has been noted that at times these bags had been moved and placed to one side, probably to allow access to the beach. This access is thought to be for maintenance crews who may need access to the beach occasionally. It is recommended that in the immediate short term these sand bags are kept in place and all personnel who use this access given instructions to seal the gap again as they leave.

Two options for this location were developed as part of the emergency works report and these have been presented in section 17.4.5.

4. Sandymount Strand Defence Code SMS OB 6, Figure 1.7 & Appendix E2

There are a number of actions and issues highlighted at this defence unit, see Figure 1.7 for location and Appendix E2, Photograph E2.21. The one identified as urgent is not flood risk but health and safety. It involves the access steps and hand railing at a location within this unit and the overall poor condition that both are in. The investigation should identify if they are still used by the public and if closed what alternative exists. If closure is not an option, then replacement of rails and cleaning and reforming of the concrete steps should be undertaken.

5. Sandymount Strand Defence Code SMS OB

The database highlights an action at many of the defence units within this length to investigate the **flood risk** through the back wall along Strand Road. This is known to be a problem and was indeed a problem in February 2002. However, blockage of all these gaps as at Marine Drive, would not be a viable option as the area is used daily by many people for recreational purposes. Early in the project as part of the emergency works report, a number of options were developed to close the gap which exists opposite Marine Drive leading into Sean Moore Park. Since then more long term options have been developed along this length to deal with the remaining gaps and setback walls and these together with the early emergency works options are presented in section 17.4.4.

6. Royal Canal General

The condition survey concentrated on the immediate canal channel, i.e the land water interface. However, there are a number of wider issues in this region, beyond the canal around the boundary of the CIE land. A report has been completed by Kirk McClure Morton (KMM), which specifically concentrates on the Royal Canal and in particular recommended the provision of a new flood gate within the canal to prevent further inundation's similar to that experienced in February 2002, see section 17.1.2. Whilst it is felt that this is the most appropriate long term option, it is worth mentioning a number of wider issues which should be given some consideration in the short term pending the completion of the flood gate and indeed even following its completion, as the effectiveness of any flood gate is only as good as the forecast and operational response procedures required to close it. These wider issues include:

- Royal Canal left bank: It is certain that water did in fact weir over considerable lengths of the left bank, significantly adding to the volume of water that had already escaped through the lift bridge, see Figure 1.6 and Photograph C5. Since the event part of the left bank wall, from Newcome Bridge to the Rail Bridge, has been increased in level by placing an additional two course of brick to the crest of the wall. Whilst this will help, the wall was noted to be a considerable high at a number of locations and the overall stability for a water level to the crest could be questionable in places. It may be that a number of buttress piers will need to be added to increase the stability. This should be investigated through discussions with CIE to determine whether they have any details of the canal walls. In addition it is certain that a considerable length of the canal downstream of the Railway Bridge was also overtopped is still at risk of being significantly overtopped and as such some further remedial measures should be considered to raise the levels in the short and even long term.

- CIE Boundary Wall Ossary Road: While the wall that collapsed during the February 2002 event has been repaired, see Figure 1.6 and Photograph A32, a number of issues were noted. Firstly the drop to the Ossary Road is considerable and if the wall was to retain water again on the CIE side, the pressures at the top of the wall and hence overall moment down to road level could be high. Therefore it should be checked with CIE if the repairs considered this in terms of the rebuilt. Secondly, the majority of the repaired wall two blocks wide, however there is a short section perpendicular to the main section along Ossary Road which is one block thick. The implications of this, if any, should be checked when considering the overall integrity of the repaired wall.

- CIE Boundary Wall Blyth Ave & Irvine Terrace: At these locations the metre cube sand bags are still in place, see Figure 1.6 and Photograph A34. However, it is noted that these would not be fully waterproof, but would reduce the flow at both locations. The situation could be improved in the short term pending the overall recommendations for the Royal Canal, by rapping an impermeable membrane around the bags. In addition the level and condition of the boundary wall along Blyth Ave is in question, as video footage of the February 2002 event showed considerable seepage through the wall from the CIE land. In addition more extreme events than that of February 2002 could fill the CIE land to such an extent that overtopping of this boundary wall could occur. This, combined with the seepage, could result in a breach in the wall resulting in considerably larger volumes of water and health and safety risks to those residents in Blyth Ave in particular. This further reinforces the consideration that some additionally raising of the levels along the left bank of the canal should be undertaken even in conjunction with a flood gate option.

- CIE Boundary Wall West Road: In Chapter 3, February 2002 Flooded Areas, an additional possible flood path in the CIE land was identified and involved water escaping through an unused car park and through the railway underpass into West Road, see Figure C1.5 in Appendix C. During a site visit the car park was inspected from the railway line that was reported flooded, and clearly it could be seen that if the track was flooded, then so would have the car park. A block wall was noted along the end of the car park adjacent to the track, but it had collapsed in a number of places. Therefore it is recommended that either the wall be repaired over its length or be replaced with a low earth embankment to seal this flood path.

7. Baldoyle Defence Code BAL OB 4, Figure A1.3 & Appendix E2

This defence unit consists of the masonry wall at the location of the roundabout where the Coast Road meets Strand Road, see Figure 1.3 for location and Appendix E2, Photograph E2.1. It has been highlighted with an action due to the identification of a low spot where the wall meets an earth bank on the Coast Road and is thus a **flood risk** action. Flooding occurred here in February 2002 affecting mainly the road and the action is to investigate whether this low spot was the flood mechanism. This action has been evaluated through the flood risk assessment work and remedial works to address the flood risk at this location have been developed and are presented in section 17.3

8. Howth South West Defence Code HWW, Figure 1.4 & Appendix E2

There are several actions categorised as high and a number as urgent along this defence length, see Figure 1.4 for location. Some of the actions are to investigate flood paths and a number are in relation to the condition of walls and suggest repairs should be undertaken. They include:

- **OA1, 3 & 4** which have been highlighted as requiring action in the form of repairs to cracking and undermining of the wall, particularly OA3, which suggests the wall is cracking and undermining and the action has been highlighted as urgent, see Appendix E2, Photographs E2.4, E2.6 & E2.7. In the short term the works along these units could consist of some concrete underpinning at length 3 and concrete repairs and re-pointing at other locations. However, implementation of any of the long term options recommended in section 17.3.4 for Howth South West would address these concerns.

- **OA1, 2, 10, 11 & 12** which have been highlighted as requiring action in the form of investigating possible flood mechanisms through openings in the wall and over the wall, see Appendix E2, Photographs E2.4, E2.5, E2.8, E2.9 & E2.10. These have generally been given a high priority. All have been investigated as part of the final flood risk assessment and alleviation measures developed and presented in section 17.3.4.

- **OB 9, 13 & 17** are identified as requiring action in the form of repairs to the walls, see Appendix E2, Photographs E2.11, E2.12 & E2.13. OB 9 and 13 have been classed as urgent, as severe cracking has been noted at these locations. While none of the above provide a flood risk, the main issue here is one of health and safety in that should the wall and hence bank fail there is a road located above which could be at risk. Actions - The surcharge on the wall should be investigated and concrete repairs to the cracks undertaken, pending more detailed long-term stabilisation works. Options to address these issues have not been included in chapter 17 of this report.

9. Clontarf Defence Code CTF OA 4, Figure A1.6 & Appendix E2

This defence unit consists of the low revetment at the western end of Clontarf Road, see Figure 1.6 and Appendix E2, Photograph E2.2 (& A26 to A29). The action here has been highlighted to investigate the need for raising the crest level in light of the flooding on 1st February 2002 and is thus **a flood risk action**. It has been noted that this was a vulnerable spot from very early in the project. However there is no quick fix short term or temporary solution pending identification of long term works and such long term options have now been evaluated and are presented in section 17.4.1.

10. Clontarf Defence Code CTF OB 2, Figure 1.6 & Appendix E2

This defence unit consists of a bandstand element which protrudes out from the promenade wall and is located to the west of the Clontarf Baths, see Figure 1.6 and Appendix E2, Photograph E2.3. Therefore it forms an integral part of the defence along that length. It has been highlighted mainly due to the poor condition of the wall at the seaward side. There are two issues here to consider. Firstly should the wall collapse, then there would be a hole in the promenade wall, which is the primary line of defence. Secondly, people can gain access to the bandstand and therefore failure could pose a health and safety risk. Therefore works to maintain the integrity of this wall should be considered. Repair works were identified as part of the earlier emergency works report and these have now been incorporated with the options set out in section 17.4.1.

17.3 FINGAL COUNTY COUNCIL AREA.

17.3.1 BALDOYLE ESTUARY OPTIONS

The options for this area extend northwards from Baldoyle Town around the Baldoyle Estuary to the existing road that leads to the Portmarnock Golf course (Chainage 1630 to 4633).

Several locations along this extent require different options, many of which are ‘soft engineering’. The main issues addressed by these options are:

- Reducing the risk of still water and wave overtopping onto the existing main arterial carriageway running from Baldoyle to Portmarnock. The crest levels of existing and /or new embankments will be raised to give sufficient freeboard for wave action and future sea level rise.
- Increasing the drainage capacity and water storage facilities at the northwest section of the estuary, adjacent to the Portmarnock Roundabout.
- Repairing, replacing existing masonry coastal walls and raising the crest levels

Each location and its options will be described in more detail below.

At this stage, options are not considered to protect the road leading to the Portmarnock Golf Course, as only the road is liable to flooding. The road levels along this section at one location are low, but rise up again to meet the golf club grounds, which are at a higher level and are not at risk of flooding. Along the road there is one property located at the entrance to the golf course, which would be at risk of flooding during more extreme tides.

An option has however been developed (Location 6) to consider a potential flood path from the glof course and along the access road to the residential properties in Portmarnock. This option has been described in more detail below.

Location 1 – North of Baldoyle Town Centre (Chainage 1630 to 2293)

The options for this area seek to improve the existing low lying flood defences, which lie along the Coast Road just north of Baldoyle Town. These defences comprise of a shallow grass bank with a slope of 1 in 5 and a crest level of 3.56mODM. At the southern side the bank connects into a high concrete wall just north of Baldoyle Town centre and into a wide vegetated area that contain a few residential properties further to the north. The Coast Road running behind this grass bank is the main arterial route between Baldoyle and Portmarnock. This route is busy throughout the day, so flooding of this road would cause considerable disruption to the local residents and businesses.

Option 1 – Bermed Earth Bank

This option consists of a bermed earth bank, on the seaward slope. The option contains a small toe berm at a level of 2mODM and at larger berm of around 2.5 metres at a level of 3.25mODM. The objective is to reduce any potential wave overtopping, which on extreme tides could cause significant flooding of this part of the road. The use of a bermed defence allows the design crest level of the earth bank to be kept to a minimum and a lower level than would be required with a traditional earth bank. The required crest level for this structure would be 4.1mODM. The reason for this is that waves will be forced to break over or onto the berm, which greatly reduces the wash energy and run-up seaward of the crest. The slopes should be constructed at 1 in 2 reducing the possibility of slips, and visual impact on the area.

It is intended that the bermed bank will be blended into the existing vegetated area, just to the north. Whilst the profile footprint of the bank is large, some 14m, it is felt that this bank will present a more environmentally sensitive option, than that of a more hard defence such as a wall. It will reduce wave reflections over a vertical structure and may encourage the growth of the existing saltmarsh in the area locally surrounding it.

The capital cost (including VAT) for this option is estimated at €365,000.

Option 2 – Earth Bank

Similar to Option 1, above, this earth bank is styled in the more traditional sense, and does not have a berm down its seaward slope. In order for this defence to be as effective in reducing wave overtopping as that of Option 1, the crest level of the bank is required at 4.4mODM. Although this raising of the bank will reduce the visual aspect into Baldoyle bay from the sea, it does reduce the overall footprint of the structure to 11m.

As with Option 1, the earth bank will be landscaped into the existing vegetated areas to the north and it is may help to encourage saltmarsh growth within the local area.

The capital cost (Including VAT) for this option would be €300,000.

Option 3 – New Sea Wall

This option consists of a new reinforced concrete sea wall with a crest level at 4.25mODM. The new vertical structure could have an influence on the foreshore in this area through increased wave reflections and hence scour during periods of high and extreme water level. It is interesting to note that that the foreshore fronting the walls to the south consists of shingle, with not saltmarsh and bed levels are much lower. The wall has therefore been buried sufficiently deep in the event that beach levels fronting the new structure dropped in the future. In addition to help minimise this risk some rock scour protection has been placed along the front of the new wall. This rock could also be covered over with topsoil, vegetation and new turf, on completion of the works to maintain the grassed natural of the area.

The southern end of this new wall would tie into the existing sea wall, which runs northwards from Baldoyle Town. Whereas at the northern end, the sea wall would tie into the existing vegetated bank.

The capital cost (Including VAT) for this option is estimated at €450,000.

Details of the works required within Location 1 are presented in the drawings 9M2793 – 100 to 101.

Location 2 – South of Mayne River (Chainage 2593 to 3011)

Location 2 starts at the northern end of the wide vegetated area (progressing north from Location 1) to the Mayne River. At the northern end of this vegetated area there are two residential properties within its boundaries, backed by the main carriageway, and each has threshold levels of 2.7mODM. During the February 2002 event this area became inundated and both the properties and road behind were flooded. Just beyond the location of these houses, where the vegetated area ends, the road is protected by a set of gabion baskets, with a crest level of 2.5mODM. To the seaward side of the gabions exists a wide expanse of saltmarsh, which would greatly reduce wave action on this frontage. The footpath level along the gabions is around 2.6mODM and effectively forms the defence in this area.

From the end of the gabion baskets to the Mayne River existing a masonry wall, which protects the arterial carriageway, with a crest level of around 3.5mODM. This wall does not provide an adequate level of protection and would require raising. Indeed during the recent event of the 28th October the defence was not overtopped but the road adjacent was still flooded. This was noted to be due to water leaking through its joints and as such the overall structure is considered to be in a poor stated of repair. It is not considered that this wall could be raised to the required level with out significant rebuilding and improvement works and it is recommended that it be replaced.

Option 1 – Earth Bank and Replace Existing Wall To Mayne River

The perimeter of the existing vegetated area surrounding the residential properties consists of vegetated banks which are in a poor condition and are considerably lower than is required for this area. This option proposes reconstructing the existing earth banks, and increasing their crest level to 4mODM reducing the flood risk to the properties and the road. It is believed that during February 2002 these properties were also effected by water entering through their septic tanks, and therefore in order to protect the properties further, is recommended that they have non return valves fitted to all their drainage sewage outlets.

Along the length of the vegetated area to the south of these properties wave overtopping is not a major problem, as the wide expanse of land to the rear will soak up any overtopping water. The main flood risk will occur when the still water level exceeds the defence and direct inundation will occur. The new earth perimeter bank will start in the middle of the existing vegetated area. The ground levels to the south of this are sufficiently high for there to be no flood risk.

The rebuilt earth bank should protect both the residential properties, and as a consequence the road running adjacent to them. To further increase the flood protection, the earth bank will run around the perimeter, and in front of the existing short length of gabions which run just to the north of the vegetated area. During the February 2002 flood event this area was a low spot in the existing flood defences, which contributed to the inundation of the residential properties and the road.

The existing masonry wall leading northwards to Mayne River will be removed and a new recurve reinforced concrete wall with a crest level of 4mODM, will be put in its place. This wall will not replace the existing gabions, which will form the landward toe of the new embankment. It is therefore proposed to provide a new wall only over the length of the existing wall. This new earth bank will overlap sufficiently with the new retaining and flood wall structure.

Details of this option are shown in drawing numbers 9M2793 - 110

The capital cost (Including VAT) for this option is estimated at €1,300,000.

Option 2 – Earth Bank and New Wall replacing Gabions and Existing Wall to Mayne River

This option commences in a similar way to option 1, with a reconstruction of the earth banks that run around the perimeter of the vegetated and residential properties. However, unlike Option 1 the earth bank does not continue over the length of the gabions. Instead the new recurve concrete flood wall will commence over the length of the gabions, with a crest height of 4mODM and continue north to Mayne River. Once this wall has been constructed, the earth bank will tie into the new wall to complete the flood defences along this section of the coast line.

Details of the works required for option 2 within Location 2 are presented in the drawings 9M2793 – 111.

The capital cost (Including VAT) for this option is estimated to be €1,550,000.

Location 3 – North of Mayne River (Chainage 3074 to 3378)

The existing flood defence along this location is a shallow vegetated earth bank, with a crest level of 2.8mODM, which runs up to the Coastal Road. It is not know if this section of road did flood in February 2002, however, the road and ground levels would suggest that this area is low lying enough to have been at considerable risk then, and again in the future.

Option 1 – Earth Bank

This option proposes to construct a new earth bank over this location and tie it into the natural vegetated earth defences, to the north and also the hard defences to the south at Mayne River. The earth bank will have 1 in 2 slopes and a crest level of 4mODM. It is not expected that significant wave action will exist here, nevertheless small freeboard is included to all for modest wave action, shrinkage cracks and settlement.

As with location 1, it is considered likely that the bank will help to encourage the establishment of new saltmarsh surrounding this area.

The capital cost (Including VAT) for this option is estimated to be €400,000.

Details of the works required within Location 3 are presented in the drawing 9M2793 – 120.

Location 4 – North Western End of Baldoyle Estuary (Chainage 3964 to 4174)

At this location there is an existing masonry wall which forms the boundary to the low lying Coast Road (with road levels of 2.3mODM), and the saltmarsh on the seaward side. The wall just stops short of the vegetated earth area to the south. This leaves a gap which flood waters can pass through when tide levels are sufficiently high as occurred during the February 2002 event, flooding the road. It is also believed that the wall, as with the wall at Mayne River, is not impermeable and allows water to seep through it on extreme tides, which provides an additional flood mechanism at this location. On the seaward side of the wall, there is a water storage area and an existing dilapidated drainage outfall, with a flap valve in very poor condition. FCC have indicated that this out fall serves a very small localised catchment in this area, which included some field drains from the adjacent field. The main flood issue in this area will be the inundation through still water level rather than through wave action.

Option 1 – New Earth banks, new flood water storage area and raise road levels.

Initially this option was developed as two separate options, however it is now considered that both should be combined but as a phased approach.

Phase 1 of this option considers the construction of an earth bank at a level of 3.75mODM to prevent ingress of tidal waters during extreme tides while at the same time creating a water storage area on the land ward side. The earth bank would extend from the vegetated area to the south across the gap at the start of the masonry wall and cut across the corner of this area to tie into the wall to the west of the Portmarnock Bridge.

At present there is a small water storage area between the drainage outfall and the wall on the seaward side, the condition of which is very poor. The outfall from this area to the estuary, whilst flapped, is no longer functional and tidal water inundates it on all tides. This storage area is connected to a small localised drainage system which services the adjacent low lying area. This option uses the existing drainage system, and creates a second larger area for drainage water storage behind the new earth bank. The outfall will be replaced and extended beneath the new earth bank. A new Tideflex outfall valve would be installed on the new outfall. Rock will be placed on either side of the new outfall to prevent scour and undermining of the new earth banks.

The idea behind this option is to make a storage area for surface water runoff from the small localised system, at times when the outfall is tide locked. In addition, the current masonry wall should be repointed and refurbished to improve its condition.

Phase 2 of this option concentrates on the road levels in the vicinity of the roundabout at the junction behind the masonry wall, which are particularly low, with levels at around 2.3mODM. During the February 2002 flood event this area flooded considerably, and this in the main part is due to the road levels becoming lower in this area. This option requires the raising of the existing road levels to around 3mODM at the roundabout, with the levels falling away to meet with the existing road levels approximately 50m away, and upgrading the drainage gullies in this area.

The added benefit of this above and beyond that of simply undertaking phase 1, is that the water levels in the storage area would need to get to a level considerably higher than would be required with the existing road levels, before surge charge of the system and over spill onto the road occurred. This phase could be brought into use within 5 to 10 years after completing phase 1.

The total capital cost (Including VAT) for this option if carried out in one contract now is estimated to be €1,075,000.

If undertaken in a phased approach the estimated capital costs are as follows based on 2004 rates.

Phase 1 – New embankment, outfall, storage area and repairs to masonry wall - €345,000
Phase 2 – Raise road and roundabout - €765,000.

Details of the works required within Location 4 are presented in the drawing 9M2793 – 130.

Location 5 – Southern End of Portmarnock (Chainage 4174 to 4633)

This location is at the most northern part of the Baldoyle Estuary and is located between the Portmarnock Bridge and the road, which leads down to the Portmarnock Golf Course. Due to its location at the top of the estuary and because of extensive expanses of saltmarsh and grassed recreational areas, which front the area, it is not considered to be widely susceptible to wave action. Any wave action will be restricted by the fetch length within the estuary and the level of the saltmarsh and grassed areas.

The main flood risk threat will be through the inundation of still water levels. The existing masonry wall, which continues round from Location 4, bounds the existing carriageway, and is protected on the seaward side by a very wide grass area, with levels of around 2.8mODM.

Option 1 – New short walls and raise ground levels

This option localises the flood risk protection to the gaps in the existing walls. There are two pedestrian gaps along the existing wall protecting the road and the southern parts of Portmarnock. These gaps present a flood risk to the main arterial road behind, leading to Portmarnock town centre.

Therefore, it is proposed to close these gaps, by raising the ground levels locally and building new masonry walls, to match the existing walls, across the gaps. The new wall crest levels will match the existing masonry wall and are around 3.75mODM, and the crest of the ramped ground levels should be at approximately 3.5mODM. The raised ground levels will be graded in such a way to allow wheel chair access, as well as pedestrian access.

At the eastern end adjacent to the golf club access road, there exists a public house whose car park is low and was flooded in February 2002. Indeed this presents a possible flood path into the hinterland. Therefore at the location where the existing masonry wall finishes, it is proposed to extend a new earth bank around the public house and adjacent buildings and tie into an existing high bank at the top of the golf club access road. The crest level for this new earth bank should be 4mODM.

Details of this option are shown in drawing number 9M2793 – 140.

The capital cost (Including VAT) for this option is estimated to be €475,000.

Option 2 – New Earth banks

This option maximises the use of soft engineering, and links in with the existing earth banks that already protect the road leading south to the Portmarnock Golf Club. Therefore this option is a continuation of that earth bank and should be built to have a crest level of 4mODM.

The earth bank will continue around the perimeter of the existing public open space and tie into the existing concrete walls at the north west corner adjacent to Portmarnock Bridge. The crest level of the bank can reduce to 3.75mODM over the frontage of the open space, since more overtopping over this part could be permitted. However it would need to rise in level again as it approached the road near the bridge. The bank will reduce the flood risk, not only around the public open space, but primarily to the road behind it and also the public house and many properties beyond the road which are at risk from a 200 year event now and more so in the future.

The capital cost (Including VAT) for this option is estimated to be €735,000.

Details of the works required for this option are presented in the drawings 9M2793 – 141.

Location 6 – Southeast End of Portmarnock

Location 6 is located at the most northern part of the Baldoyle Estuary. Parallel to the road leading to the Portmarnock Golf Club is an existing vegetated earth bank, which forms the boundary between the Coast Road and the saltmarsh on the seaward side. The crest of the earth bank decreases from 4.52mODM at the west end of the club access road to 3.40mODM at the boundary of the Portmarnock housing estates with the golf course. The earth bank truncates just south of the entrance of the Golf Club. South of the Portmarnock housing estates the earth bank has several drainage outfalls which extend through the earth mound and are not flapped. These drainage outfalls provide local drainage to the road leading to the golf course but during high tides the road can also flood by this path from the estuary.

The flood defence is not considered to be widely susceptible to wave action, since it is located at the end of the estuary where the wave action will be restricted by the fetch length within the estuary and the bed levels. The main flood risk threat in this area is the inundation through still water level.

Flooding can take place over the crest of the existing earth bank adjacent to the club access road at the southern end and through the drainage outfalls in the earth bank. In addition the earth bank ceases to exist on entering the Portmarnock golf club and over several lengths of coastline large volumes of water could inundate the golf course during an extreme event. Under these circumstance water could flow back from the south of the club access road and the golf course to the Portmarrnock housing estates. These flood paths have been established and will be cut off by the proposed option.

Option 1 – New earth bank, new road ramp and raise earth bank

This option is subdivided in three parts.

Part 1 of this option proposes the construction of a new earth bank perpendicular to the club access road at the boundary of the Portmarnock housing estates with the golf course. This new earth bank will prevent flood water from flowing back from the Golf Course to the Portmarnock housing estates. The earth bank will have 1 in 2 slopes and a crest level of 3.75mODM. This part will link in with the new road ramp (part 2) on the club access road.

Part 2 of this option concentrates on raising the road level, by constructing a new road ramp, at the junction of the above mentioned new earth bank with the already existing earth bank parallel to the club access road. The road ramp will have 1 in 12 slopes and a crest level of 3.40mODM. The road ramp prevents flood water from flowing back from the southern part of the access road and links in with the existing earth bank that already protects the road leading south to the Portmarnock Golf Club.

The third and final part of this option is the proposal to raise the crest level of the existing earth bank parallel to the access road to a level of 3.75mODM.

The total capital cost (Including VAT) for this option is estimated to be €615,000.

Further Investigation

In addition to the flood risk identified and addressed by the options presented above, an additional possible flood risk could exist through the small stream which discharges into the north end of the Baldoyle estuary under the Portmarnock Bridge. Assessment of the flood risk and collection of data with respect to this stream was not requested as part of this study. However, it is felt necessary to point out that there still could exist possible flood paths from the river into the back of the Portmarnock properties in this area. It is recommended therefore that initially a level survey along the banks of the river and to the rear of the properties which back onto it, is undertaken and the flood risk further evaluated before any potential options are considered.

17.3.2 Baldoyle Town Option

The options for this area can be split into two locations. The first extends from the masonry/concrete wall just north of Baldoyle Town, across a number of properties which back onto the estuary and finishes just north of the church in the centre of town where a new high masonry wall has been constructed recently. The second extends south from south side of the church in Baldoyle Town towards Sutton (Chainage 803 to 1630) and finishes just north of the dart station.

Location 1 – North of the Church

Over this frontage there are a number of possible flood paths, some leading out onto the main road and most leading into the gardens of the properties which back onto the estuary. For the most part the defences are reasonable good, as new masonry walls have been constructed over the frontage as part of the recently completed north interceptor sewer project. However a number of gaps still potentially exist.

Over the length of coastline on which the private properties back onto the estuary, it is considered that the responsibility for alleviating flood risk to these properties lies with the owners. Most of the properties appear to have benefited from a new boundary wall which was built as part of the north fringe sewer project and these walls do provide a relatively high standard of protection. However, each property has an access gap opening out onto the shore, which could pose a possible flood path. This could easily be solved by placing either a small flood gate or dam board across the access gap, which the owner could close or install as required. One property does not appear to have taken advantage of a new boundary wall and may require some new flood wall strengthening measures in the future. It is estimated for the purposes of this study that a sum of around €105,000 including vat would be sufficient to provide a number of flood gates and undertake some general improvement works in the future

Further south the property adjacent to the church has a door opening out onto the top of a slipway. Again this could pose a risk if wave action were to run up the slipway towards the property. Again this could be dealt with by installing a small flood gate or dam boards over the opening.

The other flood risk locations include

- gap in masonry wall to north of Baldoyle Town. Proposed to close gap and remove access steps.

- Gap in wall at location of North Fringe Sewer outfall. Proposed to install dam boards over this gap which can be left in place and taken out as access requires.
- Two gaps in secondary wall along road between properties and the church. Proposed to provide small flood gates which could be closed should the need arise.
- Gap adjacent to slipway which leads onto concrete promenade running along the front of the church. The gap is sealed off with a pallaside fence and a small wall is proposed inside the fence to prevent possible flood waters flowing back along the seaward front of the church.

It is estimated that the above ad-hoc flood measures, not including the private houses, could be put in place for around €55,000 including vat.

Location 2 – South of Baldoyle Church to Sutton Dart Station

The options along this section of coastline focus on closing the gaps in the existing wall and raising it locally in places. The main issues addressed by these options are:

- Reducing wave overtopping, through existing gaps in the primary and secondary walls, and onto the existing main arterial carriageway running from Baldoyle to Sutton.
- Maintenance of pedestrian and vehicular access to the foreshore.

Each location and its options will be described in more detail below.

Along this stretch of coastline a new masonry and stonework sea wall and concrete promenade, has been constructed in the last 2 years as part of the North Fringe Sewer project, with the older masonry wall retained at the back of the promenade. These existing walls are built to an average crest level of 4 to 4.4mODM for both the primary and secondary defences. A 4m wide concrete promenade exists between both of these walls, and a number of pedestrian gaps exist through both walls allowing access onto the promenade and then onto the beach. These gaps present weak spots in the overall defence.

At all locations these gaps are positioned in parallel with each other, effectively resulting in a flood path through the seaward wall, across the promenade and out onto the road and land behind. This flood risk will be especially prevalent during times of higher than normal wave action. Wave action along this frontage will vary depending on direction of wind and hence wave direction and also depending on the location with respect to the entrance to the Baldoyle Estuary. For example the location directly opposite the entrance will be the most exposed to wave action, and particularly so when waves approach from an easterly direction. Moving further north or south from this location the waves will be subjected to diffraction as they enter the estuary and the waves will be smaller. The effects of this process have been considered when assessing the flood risk and also in developing the options. Outside of this zone directly opposite the entrance the flood risk only arises at the gaps, where as within the zone opposite the entrance under certain conditions there is a risk that the new wall will also be overtopped.

This new wall is the only protection for the arterial coastal carriageway, running from Sutton in the south to Portmarnock in the North and the extensive number of properties, which exist to its rear.

The main option proposes to raise both the seaward and landward walls where necessary over the zone directly opposite the estuary entrance. The amount the walls need to be raised will vary depending on the location of the wall in relation to the mouth of the Baldoyle Estuary. In some cases the wall will not need to be raised, as the existing wall is to an acceptable crest level. The condition of the existing sea wall is good, as it was constructed within the last 12 months. However, the landward or secondary wall is much older and therefore may potentially require some maintenance and repointing to ensure it is water tight.

All of the seaward gaps will remain open, to allow the continual pedestrian access down onto the beach. Several of the gaps on the landward side will be permanently closed, and the others repositioned in alternative locations. Thus staggered access points will be provided onto the promenade and then onto the beach. Where gaps are repositioned a new 2m wide pedestrian gaps will be provided and a new swing shut floodgate will be installed. These gates will remain open at all other times, apart from during a flood warning at which point they will be close. This will allow continual access onto the promenade, and to the beach, as necessary.

To enable free draining of water from the promenade, the existing ground levels have a natural fall towards the gaps, which will mean that as water overtops the wall or the gaps, it will be encouraged to flow back through the seaward gaps from the promenade. Once the flood event has passed this water will simply flow back out through the seaward gaps.

At the access slipway, just south of Baldoyle town centre, there is a large gap, to allow free access up and down the slipway from the carriageway. This access must be maintained, but presents a route for wave run up on the slip and hence flood water to pass onto the road. It is proposed to use two alternative options to ‘close’ this large flood risk gap.

Option A

The first option involves narrowing the existing gap in the landward wall to 12m, and installing four 3m long Dutch Dam demountable flood defences. These flood barriers will remain collapsed until required during any flood event.

Option B

The second option is to again narrow the existing gap in the landward wall to 8m wide, and installing two sliding flood gates. These floodgates will meet at a demountable post halfway between the two walls. This post can be unlocked and dropped down for access down the slipway and will be stored in a socket beneath its erect position. In addition to the sliding flood gates it may be possible to use ‘swing shut’ flood gates as an alternative to the sliding flood gates but these may be more cumbersome to use.

A potential third option is to re-orientate the existing slipway to run alongside the seaward wall, rather than perpendicular to it. The present situation has the potential to allow direct waves run up on the slipway, through the gaps, and out onto the road behind. By moving the slipway parallel to the seawall and protecting it with a new masonry perimeter wall, the potential ‘rush up’ of waves could be negated. This option however would require major reconstruction work on what is a relatively new slipway and frontage and would also be costly. Therefore it is considered that it is not a viable option at this time and has thus not been considered further.

The capital cost (Including VAT) for this option with option A at the slipway is estimated at €725,000 and with option B at the slipway, €770,000.

Details of the works required are presented in the drawings 9M2793 – 200 to 220.

17.3.3 North Howth Options

The options for this area extend northwards from the western pier of Howth Harbour to the entrance of Baldoyle Estuary (Chainage 0 to 2933).

Over this frontage the coastline can be divided into three distinct areas. The first extending from Howth Harbour to the New Apartments which have been developed adjacent to and on the seaward side of the railway line (chainage 0 and 1263). The hinterland over this length is protected by a concrete promenade and set back wall and also by a short length of sand dunes and set back wall adjacent to the apartments. The second area extends from the apartments to the end of the high ground further west (chainage 1263 to 1970). Over this length the hinterland is high and most properties are set back on higher ground on the crest of coastal cliffs. However a number of properties are located at a slightly lower level and more near the foreshore and these are protected by high masonry and concrete walls as appropriate. The final area extends from the end of the high ground to the golf club at Gush Point (chainage 1970 and 2933). The hinterland over this length is lower and is protected primarily by sand dunes and secondarily by property boundary walls and an old sea wall much of which is now buried by the dunes. Much of these beach dunes are vegetated at the crest with marram grass, although they are susceptible to erosion and shift along their front face during extreme events.

The most significant issue along this frontage is one of coastal erosion, particularly over areas 2 and 3. Whilst most of the area is not at significant risk of flooding at present, continued erosion could change this situation in the future. From discussions with local residents, it is believed that the sand dunes along area 3 did not exist some 20 years ago and that the coastal defence back then was a masonry wall, which is now buried by the dunes. However, in recent times increased erosion of the front face of the dunes has been noted and they are becoming narrower. This project whilst investigating the flood risk as it exists with respect to the current conditions, has not investigated the geomorphologic processes that are at work in this area. This would require a specific coastal evolution and geomorphological study, which is beyond the scope of this current project.

Nevertheless, a number of options have been considered along this frontage in the context of minimising further impact on and thus helping to maintain the current defences. Furthermore consideration is given to improving the current boundary walls along the exiting properties and road in respect of flood defence.

The main issues addressed by these options are:

- Reducing flood water ingress into the set back properties along beach.

- Maintaining public amenity access to the beach and foreshore.
- Reducing dune erosion, where applicable, to provide natural flood protection to the area.

Area 1 – Howth Harbour to Apartments (ch 0 to 1263)

Over this area a promenade and high concrete wall at the back of the beach protects the DART coastal railway line, between Sutton and Howth. The promenade itself is protected by a concrete sea wall with a crest level of 3.3mODM, with promenade levels of 2.7mODM. The wall at the back of the promenade has a crest level of 5mODM.

At the extreme west of this section of coast, the promenade stops, with the back wall continuing along the line of the railway to chainage 1263. Over this area between the end of the promenade and the apartments the back wall is fronted by low level sand dunes, which meet the wall just west of the end of the promenade and which becomes considerably wider moving west. The level of the dunes around the toe of the wall, are approximately 3mODM. Flood risk evaluations of this area have shown the defences along this length to provide an acceptable level of protection and as such no flood risk exists. Therefore at present no flood alleviation options are proposed.

The location where the low sand dunes meet the railway wall near the promenade should be monitored, as this is the most vulnerable spot along this length in terms of coastal erosion.

Area 2 – Area of High Ground

Over this length of coastline the general hinterland is high and there are few flood related issues. Instead the main problem would be one of erosion and coast protection and as such most of the frontage is protected by masonry and concrete walls built along the toe of the cliffs. Behind the walls are several residential properties, many of these are set up on the crest of the cliff, some 20m above the beach level. The properties do have accesses down to the beach. For those properties located on the cliff crest there is no flood risk. Two properties however are located closer to beach level on the front side of the high ground. Both have a relatively high masonry wall fronting them and both have access openings onto the beach which could results in a potential flood paths. The risk of flooding in both cases would be localised to the houses in question and as such no options to address these individual properties have been considered in this report. To improve flood protection the properties in question could consider installing a flood gate across their access paths to the beach and in addition they should monitor the condition of their defence wall and the beach levels fronting the defences.

Area 3 – High Ground to Golf Club (ch 1970 to 2933)

Over this length the hinterland is protected by sand dunes fronting individual property boundary walls/defences. It is believed that these dunes have deposited within the last 20 years and now cover an old masonry seawall which used to be the coastal defence. During the February 2002 flood event the dunes prevented significant flooding along this length but were nevertheless severely damaged during the storm. In addition a number of low spots exist through the dunes at several locations which allowed ingress of flood waters through the dune system to a number of access gaps behind.

Currently the dunes are between 3.5m and 4.5mODM at the crest. The low spot levels are mainly due to pedestrian accesses across the top of the dunes. Most of the properties along this length have access gates or gaps leading onto the shore and low spots exist at most of these locations created by repeated pedestrian movements across the dunes to the foreshore. These furrows have the potential to allow flood waters to flow through to the road and properties behind. In addition there are a number of accesses leading out onto Burrow Road behind the properties. One of these is a large vehicular access gap which has a significant gap cut through the dune system. Flood water did escape through this gap in February 2002 and out onto Burrrow Road although the flooding was not significant.

Whilst the main form of defence is the dune system, this is integral with the property defences and as such some measures can be considered to help maintain the integrity of the dunes and improve the protection offered by property boundaries. Therefore a number of options have been considered and have been broken into two options, each of which will require varying maintenance, management and resources. In addition to these options it is recommended that a detailed coastal evolution and geomorphological study be undertaken across this region to investigate the issues more fully and further develop appropriate coast protection options for the area as necessary. This is discussed further later.

Options to Maintain Primary Defences (Dune System)

The dune system is the primary defence along this length and is the best form of defence provided it remains intact. However, there is considerable evidence of erosion along the front face and to a lesser extent over the crest. The erosion along the front face is primarily due to wave action during stormy conditions, whereas the crest erosion is mainly due to pedestrian activity.

Options to address the front face erosion of the dune system in the longer term could include,

- Offshore breakwaters
- Fishtail groynes
- Beach renourishment
- Sand dune management system

In reality it is likely that an appropriate scheme will involve a combination of some or all of these.

However, any such scheme would require a detailed geomorphological study to investigate the present sediment movements in the region and also any impacts the proposed options might have on the region and further afield. Such investigations are beyond the scope of this study and it is recommended that they are carried out so that a better understanding of the processes in the area can be made which will aid the development of the most appropriate option to safe guard this area in the future. The costs associated with an appropriate scheme are likely to run into millions of euro and the cost of an appropriate investigation study likely to be in the region of €250 to 350K, including for detail bathymetric survey work and modelling (including vat). Whilst detailed cost estimates for such options have not been evaluated here it is recommended that an allowance of at least €0.85 to 3.1m (including vat) be made in any future cost budgets to undertake appropriate investigations and works should the need arise.

Nevertheless, a number of short to medium term measures can be implemented to help maintain and encourage the dune system growth. In particular measures can be put in place to reduce recreational erosion. These measures would involve some form of management system and might include;

- Fencing
- Board walks
- Marram grass planting
- Dune feeding (moving sand around)

Fencing could be used to restrict access across these dunes and help direct pedestrians through controlled gaps, which could be changed from time to time. Additional sand trap fencing could be erected at existing gaps and extending out from the front face to encourage the build up of sand both on the crest and on the front face. Marram grass could be planted across the dunes and in the fenced off low crested regions to further help sand to build up and the dunes to stabilise and grow. Further erosion is like to occur during storms and by recreational users, nevertheless the process would require ongoing management to be effective and the repair of damaged or removed fencing as required.

Pedestrian access could be maintained through specific gaps, like the one out to Burrow Road, in the middle of the dunes. However many residents have direct access on to the dunes from their garden and at these locations low spots are clearly evident. The residents may not wish to be directed to specific access points and so a series of access points in the form of suspended timber walkways could be constructed to allow access from the residential properties to the beach. These walkways will go over the crest of the dunes and be supported by timber posts and columns. These walkways would help prevent user erosion.

During an extreme event in October 2004, it was noted that the front face of the dunes was heavily eroded as a result of severe wave action. Whilst these options will maintain the existing dunes from regular minor storms and weather occurrences, they will not remove the risk of extreme events damaging the integrity of the sand dunes.

It is estimated that a programme of dune management works could be implemented over a number of years for the sum of around €85,000.

The results of this management could be monitored and conclusions drawn as to whether it was proving beneficial or not and this could help build the case for a much more detailed geomorphological study and the investigation of longer term options. Before any sand dune management works are implemented a small design study should be undertaken to investigate the appropriate location and layout of the works.

Options To Improve Secondary Defences

Options along this stretch options to improve the secondary boundary wall defences could involve localised flood protection to close access gaps leading to properties and out onto Burrow Road, which are generally located directly opposite low areas in the sand dunes. These flood protection measures would take the form of flood gates or demountable systems which would only be put in place by the property owner when needed. The flood gates across access paths out to the Burrow Road would however fall under the operational response of FCC. This response would be triggered by use of the flood forecasting system developed as part of this study.

The use of temporary and demountable defences could take the form of pallet barriers, dam boards or mobile dams. These flood defences require a significant amount of resource management to mobilise and demobilise.

Flood gates, depending on the gap to be closed can either be slid or swung shut. In terms of the flood gate required for the access to Burrow Road, which is 5m wide, a central post to support the flood gates would be required if the gates were of swing shut type.

These options would be effective in the short to medium term provided the dune system remains intact over that time. However, these may need to be re-evaluated following continued erosion of the dunes and new more substantial forms of flood defence may be required on the back edge at that time. This could take the form of a new sea wall, or a beach regeneration scheme.

There are approximately 19 gaps along this frontage, the majority of which lead into private properties. Of the 19, possibly 2 are access gaps which are likely to be maintained by FCC. One is a large vehicular access gap leading down to the beach from Burrow Road. It is recommended that a sum of approximately €560,000 including vat should be allowed for if new flood gates were to be installed at all gaps. Of this approximately €70,000 including vat should be allowed for to deal with the two access gaps.

17.3.4 Howth South West Options

The area considered over this frontage extends from the eastern end of the private properties on the seaward side of Sutton Cross (Chainage 0), east along Greenfield Road and then south east along Strand Road for approximately 700 metres until the ground levels rise sufficiently high to be free from coastal flood risk (Chainage 1100). The main issues along this frontage which need to be addressed are:

- The poor condition of the rubble revetment and crest level of the revetment along Greenfield Road. The lowest crest level of this revetment is approximately 2.86mODM and is considered to provide a standard of protection of less than 1 in 10 years. This revetment was considerably overtopped in February 2002.
- The land drainage pipes through revetment which have been installed to help drain surface water build up at the lowest point in the region which occurs at the junction of Greenfield and Strand Road. These are currently blocked preventing free drainage onto the foreshore.
- The poor condition and level of the seawall running along Strand Road and access gaps in that wall. In general the crest level of this wall is around 3.4 to 3.5 mODM and raised to between 4 and 4.3mODM over the last 250m. In respect of condition the coastal inspection had identified that repairs are required to much of this wall which suffers from cracks and in places scour along the toe.

A number of options have been considered and have been broken into two lengths. These lengths include:

- Chainage 0 to 450, over the length of the existing rubble protection.
- Chainage 450 to 1100, over the existing seawall along Strand Road.

For Chainage 0 to 450 three options have been considered and these include:

1. New rock revetment with concrete crest wall on seaward side of existing footpath. New slipway access with flood gates and new outfall chamber at drainage pipes (drg no 9M2793 – 300 & 301).
2. New rock revetment with concrete flood wall set back behind existing footpath. New slipway access with flood gates and new outfall chamber at drainage pipes.(drg no 9M2793 -310 & 311).
3. New rock revetment with set back flood wall along Greenfield Road. New slipway access with flood gates and new outfall chamber at drainage pipes (drg no 9M2793 – 320 to 322).

For Chainage 450 to 1100 two options have been considered and these include:

4. New raised concrete seawall with rock scour protection along the toe. New slipway. Beyond Chainage 850, the existing masonry wall to be repointed and raised slightly to a uniform level (drg no 9M2793 – 330).
5. Repaired existing seawall and/or new existing seawall with rock scour protection along the toe and a new set back flood wall with provision of promenade facilities. New slipway. Beyond Chainage 850, the existing masonry wall to be repointed and raised slightly to a uniform level (drg no 9M2793 – 340 & 341).

For the complete frontage any of options 1, 2 or 3 can be combined with options 4 or 5 to provide a composite and uniform defence and reducing the risk of flooding considerably from less than a 1 in 10 year event to an event of greater magnitude than and 1 in 200 year return period.

A more detailed description of each option is presented below together with an estimate of the capital cost of each. The following text should be read in conjunction with the drawings mentioned above which can be found in Appendix Q.

Option 1 – New Rock revetment and crest wall.

This option considers the replacement of the existing rubble protection along the location of Greenfield Road with a new formal revetment to prevent the risk of future coastal erosion. In addition it is proposed to construct a new RC crest wall at the back of the revetment and on the seaward side of the existing footpath.

The new revetment would have a crest level of 3.5mODM, a 1:2 slope and consist of primary rock armour in the range 0.25 to 1.5 tonnes. The toe of the revetment should be buried below the existing beach level and consist of a minimum of three rocks >= 1.5 tonnes. The existing beach profile should be reinstated over the toe on completion. The armour layer would be placed on a small underlayer of rock which in turn would be placed on a suitable geotextile. The new crest wall would have a crest level of

4.25mODM and should have a re-curved front face to help deflect any wave run-up over the revetment slope and crest, see Drawing Numbers 9M2793 – 300 & 301.

A beach access exists around Chainage 100, which allowed vehicular access to the foreshore. This will need to be maintained for use by FCC works department to maintain the existing or new structures and also for emergency vehicle access to the coast. It is proposed to construct a new concrete slipway with a 1:8 slope and edge walls to tie the revetment into on either side. The toe of the new slipway should extend below the existing beach level and the beach should be reinstated. In addition short concrete skirts should project down into the beach from the deck slab to prevent undermining should the beach levels drop under storm conditions. To ensure adequate access it is proposed to make the new slipway 3.5m wide. There would also be a requirement for a flood gate at the top to control wave run up under storm conditions. Due to the width of the proposed slipway it is proposed to provide two swing flood gates which can be closed onto a central support. It is proposed that the gates be left open under normal conditions and only closed when a flood warning is received. The central support can be removed when access is required to the beach, however it is recommended that it be left in place during normal conditions to act as a traffic barrier and hence restrict vehicular access to authorised vehicles only. This set up would maintain pedestrian access at all times.

At approximate Chainage 350, there exists two large concrete drainage pipes through the existing bank and rubble protection. The seaward side of these pipes open out onto the beach with no outfall chamber and are not flapped. At present they have been blocked with timber boards and are almost submerged in sand and shingle. Drainage through the pipes is therefore restricted to a slow seepage. On the landward side a drainage ditch has been cut into the ground from the road at the junction of Greenfield and Strand Roads, to allow any build up of surface water on the road to drain towards the pipes and out onto the beach. Levels at the road end of the channel are around 2.5mODM falling to about 1.6mODM at the inlet to the pipes. It is believed that the current channel, which also acts as a small storage pond, has managed to contain storm water run off in this area, however no analysis in terms of catchment runoff has been undertaken as it is considered to be beyond the scope of this project.

Considering the need to maintain this outfall, it is proposed to improve its efficiency by providing a new concrete outfall chamber on the seaward side with non return flap valves fitted to each pipe. To achieve this, the existing concrete pipes would need to be extended slightly and a new head wall and outfall chamber constructed within the revetment. However a potential problem with the outfall chamber is the build up of sand and shingle within the chamber, due to wave action, which eventually would prevent drainage through the pipes as the flap could become blocked preventing it from opening. Regular maintenance would therefore be required to remove the shingle and keep the flap free from build up. To help minimise this maintenance it is proposed to have a fairly long (2.2m) chamber and provide a number of shingle filter traps or deterrents. These include:

- Ensuring the armour stones in the top layer of the revetment are placed in front of the end of the outfall chamber and above the invert level. This way the stones will help to prevent wave driven shingle from being washed into the chamber, although this will not completely prevent the problem but it will reduce it.

- Placing timber dam boards across the chamber approximately 0.8m back from the front edge. These dam boards should extend the full height of the edge wall and would help to catch additional shingle which has been projected over the rock at the front. To maintain drainage through the boards it is propose to cut several small holes, approx. 125mm diameter, into the bottom two rows of boards. This will maintain drainage but reduce the quantities of shingle which can be blown through the boards and into the rear chamber.

Whilst these measures will help reduce the build up, they will not stop it completely, and maintenance will still be required to clean out the chamber. Removal of the timber boards will help to make this job easier.

In addition to the new outfall chamber, the existing drainage channel could be widened to provide additional storage capacity during tide locked conditions. The storage pond shown is indicative only and had not been developed through runoff or hydrological analysis.

With this option it is proposed to close the informal access to the beach at Chainage 0 and allow access to the beach only via the new slipway. This reduces the need for an additional flood gate and beach access structure. A new footpath should however be provided behind the new wall between Chainage 0 and the new slipway, which will still allow pedestrians access to and from Greenfield Road at Chainage 0. If Option 1 connects to Option 5 at Chainage 450, then a floodgate will be required across the footpath. This would not be required with connection to Option 4.

The estimated capital cost of Option 1 is €3,530,000 including VAT.

Option 2 – New Rock Revetment and Flood Wall Landward of Existing Footpath.

As with Option 1, this option considers the replacement of the existing rubble protection along the location of Greenfield Road with a new formal revetment to prevent the risk of future coastal erosion. The difference with this option is that the proposed new RC crest wall would be set back behind the existing footpath rather than immediately to the rear of the revetment crest. The advantage being that the amenity in respect of view by those persons who use the footpath is less obstructed. In addition setting the wall back further from the revetment would result in a slightly lower crest level for the new flood wall of 4.0mODM, a reduction of 0.25m from Option 1. The only exception to this would be between Chainage 0 and 100, where it is suggested that a similar construction to Option 1 should be used. The reason for this is to prevent the need for two additional flood gates which would be required at Chainage 0 and also at Chainage 100.

The ground levels along most of this section are above 3mODM, however from change 300 to 400 the top of the existing ground levels are below 3mODM. It is therefore recommended that in that area the ground levels are built up slightly and the new revetment should have a crest level maintained at 3.5mODM over the full length. At the rear of the new rock crest a small concrete edge detail would be required to segregate the fines from the larger rock voids. The revetment should have a 1:2 slope and primary rock armour in the range 0.25 to 1.5 tonnes would be required. The toe of the revetment should be buried below the existing beach level and consist of a minimum of three rocks >= 1.5t. The existing beach profile should be reinstated over the toe on completion.

The armour layer would be placed on a small underlayer of rock which in turn would be placed on a suitable geotextile, see Drawing Numbers 9M2793 – 310 & 311.

As with Option 1 a new vehicular access ramp/slipway would be constructed at Chainage 100 and similar modification made to the outfall pipes at Chainage 350.

With Option 2 it is anticipated that a flood gate would be required at Chainage 450, for connection to both Options 4 or 5 in order to maintain access across the footpath.

The estimated capital cost of Option 2 is €3,300,000 including VAT.

Option 3 – New Rock revetment with Flood Wall Along Greenfield Road

Option 3 is similar to that of option2, however it is proposed to set back where possible the flood wall along the Greenfield Road in line with the location of the existing intermittent wall. This setup could also be used between Chainage 0 and 100, although a floodgate would be required at ch0 to allow access to the new footpath on the seaward side of the wall. The flood gates at Ch 100 at the location of the slipway could also be move from the top of the slipway to the gap in the new wall.

However, between Chainage 300 and 400, the wall could not be maintained along the road edge otherwise a flood gate would be required at the back edge of the drainage channel, which if closed on a storm surge tide, would prevent any surface water run off into the channel. Therefore in order to maintain some drainage capacity in this area for surface water run off from the road, there is a need to turn the wall back towards the seaward side of the grassed area. However, the wall would be kept as far back behind the existing footpath as possible. In order to achieve this, the inlet chamber to the pipes would have to be extended and rebuilt as well as a new outfall chamber similar to that proposed in Options 1 and 2.

The revetment proposed for this option would be the same as that for Option 2. However because the flood wall is set back much further from the crest of the revetment, i.e. it is not yet dependent on the revetment to ensure its integrity, it may be possible to develop this option in two phases. The first being the provision of the flood wall to reduce the risk of flooding. The second, say within 5 to 10 years, being the construction of the revetment to halt erosion and ensure the area is maintained for amenity use and also to safeguard the flood wall in the longer term. However, if possible it is recommended that both be implemented together or at least within a few years of each other to provide a complete and secure defence in this region.

As with options 1 & 2 a new slipway to maintain access to the beach at chainage 100 would be provided. At chainage 450 a flood gate would be required for connection into either of options 4 or 5, which extend beyond this point.

The estimated capital cost of Option 3, as a complete package, is €3,355,000 including VAT.

Option 4 – New Concrete Seawall and Repointing of Existing Wall

Option 4 is the first of two options proposed to improve the flood defence beyond Chainage 450. This option proposes to improve the defences along the line of the existing seawall. However, the existing sea wall is only about 3.45mODM and to provide a level of protection up to the 200 year standard recommended, the new defence would need to be constructed to a level of 4.4mODM. This would require a rise of around 1m over much of this frontage. It is not considered that this can be undertaken viably or effectively by raising the existing wall, which in places consists of a block work wall and in others a concrete wall which required some repair works.

Therefore it is proposed to remove the existing wall and replace it with a new more secure structure. Details of this wall are shown on Drawing Number 9M2793 – 330. The wall would have a recurved front face to further defect waves impacting on the structure. The beach would need to be excavated down to a firm footing and placed on a layer of well compacted granular fill. At present no site investigations have been undertaken and so the condition of the ground in this region is not known. However, it is expected that the wall would need to extend to a minimum of about 1.5m below the existing bed level.

In order to protect the wall from scour in the future a layer of rock scour protection should be placed along the toe. This should be buried beneath the existing bed level and the beach profile reinstated. The rock should be placed on suitable geotextile to prevent the wash out of fines. The rock scour protection should be between 0.25 and 1 tonne in weight and be placed two layers thick on a core of compacted granular fill.

At around Chainage 450, the ground levels behind the existing wall and the road levels to the rear, are low, at approximately 2.8mODM. The levels rise along Strand road and at Chainage 600 the levels have increased to around 3mODM. Therefore between Chainage 450 and 600 the wall would stand some 1.6m high with respect to existing ground levels. This would obstruct pedestrian views across the bay. At Chainage 600 and beyond this wall the height would reduce to below 1.2m with respect to the ground levels and this would be more acceptable. If necessary some localised raising of ground levels between Chainage 450 and 600 could be undertaken to improve pedestrians view over the wall, although this would require a step down to the lower road levels.

At Chainage 575, there is an existing slipway gap. It is proposed that this be maintained and improved in a similar manner to the new slipway at Chainage 100. New walls would need to extend around the rear of the slipway and a new flood gate provided which could be closed when warnings are issued. Beyond Chainage 850, there is an existing masonry wall which is quite a bit higher than the adjacent wall and varies between 4 and 4.4 mODM. The wall is in reasonable condition, however it is suffering in places from scour at the toe. It is therefore proposed that the crest of this wall be broken out over its complete length and raised to a uniform level of 4.4mODM. In addition where voids and cracks exist between the existing blocks, these should be pressure pointed. The scour toe protection should also be continued over the length of this wall to ch1100.

This option can be connected into either of Options 1,2 or 3 as described earlier.

The estimated capital cost of Option 4, is €4,060,000 including VAT.

Option 5 – Repaired/New Seawall with Set Back Flood Wall

This option considers providing a new setback flood wall between Chainage 450 and 850. In this way the existing front seawall can be maintained at its original level of around 3.45mODM. By setting the wall back the overall level of the defence can also be reduced and the new set back defence level would be 4.1mODM, which is some 0.3m below that of Option 4. The new flood wall should be set back a distance of 6m from the existing seawall. In setting back the wall there is also the added benefit that a new raised pedestrian promenade can be provided which can be landscaped with suitable recreational features as required. To achieve this a new concrete slab can be provided between the new flood wall and tied into the top of the existing seawall via a new concrete cap. At this stage it is considered that the existing seawall can be repaired and maintained in the short to medium term, however it is likely that it will require replacement within the next 10 to 15 years. However to enhance its integrity during this time some rock scour protection should be placed along its toe.

Beyond Chainage 625 the existing grass verge between the seawall and the road comes to an end and the seawall runs along the edge of the road. Therefore there is no room to continue the set back flood wall in relation to the line of the existing seawall. Therefore it is proposed to construct a new sea wall seaward of the existing line. This way the set back flood wall can be maintained along the line of the Strand Road and a 6m wide promenade maintained over the frontage. The new seawall would be of mass concrete construction and built in a similar way to that of Option 4, with scour protection provided along the toe.

At Chainage 750 the existing seawall steps out again from the road, thus creating a verge which is currently used for parking. This then gradually converges back to the line of the road at Chainage 850. Therefore it is proposed that the new seawall be continued over this frontage and the set back flood wall maintained along the back edge of this verge along the road. Over this frontage in order to provide a suitably wide car park, the distance between the new wall and set back wall should be increased to 7m. This should provide sufficient room for a number of longitudinal parking spaces with room along the flood wall for travel, provided the car park is operated as a one way system. New flood gates would need to be provided at each of the entrance and exit location to the car park. Beyond Chainage 850, works similar to that described in Option 4 should be undertaken.

The estimated capital cost of Option 4, is €2,975,000 including VAT.

Summary of Option Costs

Table 17.1 – Overview of costs per option

Combination of Options	Individual Option Cost €	Individual Option Cost €	Combined Cost €
1 & 4	1 - 3,530,000	4 - 4,060,000	7,590,000
1 & 5	1 - 3,530,000	5 – 2,975,000	6,505,000
2 & 4	2 – 3,300,000	4 - 4,060,000	7,360,000
2 & 5	2 – 3,300,000	5 – 2,975,000	6,275,000
3 & 4	3 – 3,355,000	4 - 4,060,000	7,415,000
3 & 5	3 – 3,355,000	5 – 2,975,000	6,330,000

17.4 Dublin City Council Area

17.4.1 Clontarf Options

The options for this area extend west from the junction of Alfie Byrne Road and Clontarf Road towards Bull Bridge (Chainage 550 to 3275).

The options along this section of coastline focus on raising the wall, construction of a set back wall or earth bund, construction of a new secondary wall and construction of new breakwaters in the bay. The main issues addressed by these options are:

- Reducing wave water overtopping, over the existing sea wall onto the linear park behind (except option 3 which uses the park as a flood storage area).
- Halting flood water egress onto the coast road behind the linear park.
- Closing the gap at the eastern end of the promenade, replacing the existing revetment.

Each location and its options will be described in more detail below.

The promenade and properties along the Clontarf coastal stretch were severely affected by the extreme tidal event of February 2002 and again by a slightly less severe one in October 2004. There were several weak points in the existing flood defences which allowed a deluge of water to penetrate onto the main road and into a number of properties along the road. As a result a flood alleviation option is required to protect this important part of Dublin's coastline and main road infrastructure. Therefore the following four options have been considered.

All the options will introduce a new sea wall of the same arrangement as the existing sea wall to the east, between Chainage 550 and 766. This will replace the existing low revetment, which has levels of 2.6mODM. This revetment created a pathway for a significant amount of flood water to egress out onto the Clontarf Road during February 2002. It is recommended that a full ground investigation be undertaken along the whole promenade to investigate ground conditions before any one options is finalised.

As part of the emergency works report, a number of options were developed to improve the structural integrity of the bandstand adjacent to the Clontarf Baths. The long term option to improve the integrity of this structure, which includes pressure grouting and encasing the lower section of the bandstand with a concrete skin, have been incorporated into each of the options presented below.

Option 1 – Raising the existing flood defences

This is considered to be a 'minimum impact' option, whereby the existing seawall is raised, with only minimal other sea defences needing to be altered or constructed along this stretch of coastline. Between Bull Bridge and the existing derelict Clontarf swimming baths, the seawall would be raised to 4.3mODM as this stretch is exposed to more wave activity, and may therefore suffer greater overtopping as a result. The lowest point along this length of the existing wall is 3.1mODM and the highest 3.5mODM, which would require the wall to be raised by 1.2m and 0.8m respectively. Between Clontarf swimming baths and the end of the sea wall adjacent to Alfie Byrne Road, the wall would be raised to 4.15mODM. This crest level is less than the other section, as this part of the coast is sheltered, and the wave action in this area will be less. Over this length the lowest existing wall level is around 3.25mODM, with the highest at 3.6mODM. Therefore the wall would need to be raised by 0.9m and 0.55m at each of these locations respectively.

The raised sea wall may affect the visual aspect across the water between Clontarf and the port, in sections where the existing promenade ground levels are low. To counter this, the promenade would be raised to ensure that the maximum height between the crest of the new sea wall and the immediate ground levels behind is 1m. This level would be maintained for a width of 8m to facilitate the new footway and cycleway proposed by the Sutton to Sandycove cycleway project and consist of a concrete slab over appropriate sub-base and fill material. Beyond this new promenade the new ground level would be topsoiled and grass seeded and be graded down to meet the existing park ground levels.

This option has been developed to allow for slightly higher volumes of overtopping on the design condition in order to minimise the level to which the wall would need to be raised and hence the impact on the visual aspect of the area. The quantities envisaged will be contained within the linear park, where the ground levels along the Clontarf Road are higher than the newly raised ground levels along the new promenade. However there are a number of locations along the park where the ground levels at the back of the park are lower. As such the linear park ground levels will require reprofiling at some locations in such a way that this overtopping will be contained along the front of the park and later drained away through the wall as the tide subsides. Drainage gullies connected to non return flap valves would be used to facilitate the drainage.

In addition to general overtopping by waves along this frontage, part of the frontage from around the Clontarf Baths to the Clontarf Yacht Club can be subject to extreme overtopping due to the phenomenon known as Mach Stem. This occurs when the incident wave angle strikes the wall at a particular angle and builds in height as it runs along the wall before overtopping. In order to reduce the risk from this type of overtopping a number of perpendicular rock groyne type structures will be required over this frontage.

The capital cost (Including VAT) for this option would be €11,200,000.

Details of the works required within Option 1 are presented in the drawings 9M2793 – 400 to 402.

It should be noted that the above costs include for a new 8m wide promenade over the full length of the frontage to Bull Bridge to accommodate the proposals of the Sutton to Sandycove cycle way project. From a flood defence point of view, it would not be necessary to provide an 8m wide promenade and a similar spec walkway could be provided as exists currently. This is likely to reduce the capital cost of this option.

Option 2 – New set back re-curve seawall with new promenade

This option limits the amount of construction undertaken on the existing sea wall, and sets back the sea defences. The proposal is for a new re-curve sea wall to be set back 8m from the existing sea wall. A new promenade would be created on the seaward side by raising the existing ground levels and hard landscaping the area in between to a level similar to the top of the existing promenade wall level. New safety railings will be constructed on the seaward edge of the new promenade. The new set back re-curve wall would have a crest level of 4.0mODM over the frontage from Alfie Byrne Road to the Clontarf Baths and 4.15mODM from the Baths to Bull Bridge.

As with Option 1, this increase in wall level is warranted due to the more exposed location of the frontage from the Clontarf Baths to the Bull Bridge. In addition it is proposed to widen the new promenade to 10 metres from the location of the Clontarf Baths to the Bull Bridge. This is considered necessary to minimise the impact from Mach Stem overtopping over the critical length of frontage west of the Clontarf Yacht Club and also because of the low level of the existing promenade wall in places, particularly along the section adjacent to Bull Bridge. Hence the new promenade levels are lowest over the frontage adjacent to Bull Bridge and could be subject to more extreme overtopping.

Openings along the set back sea wall would allow access to the promenade. These openings would be equipped with flood gates that will be closed as necessary when warnings are issued. The flood gates provide a means whereby the safety of the public can be enhanced as they would provide a physical barrier to access to the promenade during adverse conditions. The ground levels behind the set back wall would need to be raised slightly to support the wall and would be graded down to the existing linear park ground levels. The ground leading up to the access points on the landward side should be graded at a slope of 1:20 or less to facilitate all access requirements. At each of the access points the width of the promenade has been increased to 14m by gradually realigning the set back flood wall. This provides for easier access to and from the promenade as well as setting back the weakest point in the defence, making them less susceptible to wave impact loading.

The set back promenade would be free draining should overtopping occur, thus alleviating the need for drainage. In addition to being free draining, the set back option offers the Sutton to Sandycove cycleway a potential route along the promenade without the need for alternative routing.

Along the frontage adjacent to the junction of Clontarf Road with Alfie Byrne Road, the existing sea wall finishes and a low revetment exists. Over this location there will be a need to strip out the top of the revetment and construct a new RC edge wall to support the new promenade. The revetment would then be reinstated to the toe of the edge wall. The new set back seawall will be continued westwards over this length until the ground levels behind are sufficiently high enough to provide flood protection to the carriageway and properties behind. This occurs a short distance along the Alfie Byrne Road from its junction with Clontarf Road. Again all the existing pedestrian accesses will be maintained, or realigned, and protected using swing shut or sliding flood gates as appropriate.

This option has been developed to allow for minimal overtopping, although some wind blown spray is likely during extreme conditions. As such no re-landscaping along the Clontarf Road edge of the park is considered necessary.

The capital cost (Including VAT) for this option would be €8,050,000.

Details of the works required within Option 2 are presented in the drawings 9M2793 – 410 to 414.

In addition to the use of a setback concrete wall, consideration was also given to the use of a softer method of flood defence such as an earth bund. The earth bund would be constructed over much the same line as the concrete wall, although the actual footprint of the structure would for the most part be much greater. Some typical cross sections through the bund are presented in drawing 9M2793 – 414.

As can be seen from the drawing the bund has been developed with a front and back slope of 1:6. This relatively flat slope was chosen to minimise the height to which the crest of the bund would need to be built and also to allow for ease of access across the bund and minimise the amenity impact of the area. At all cross access paths, the bund would be flattened out even more to ensure that wheel chair access to the front promenade is also maintained. At other locations between cross access points, the footprint of the bund could be made smaller by steeping the slopes, however the crest would also therefore need to be raised to maintain its efficiency as a defence. For the design as shown in drawing 9M2793-414, the crest level to the west of the Clontraf Baths is +4mODM and to the east it rises to +4.2mODM.

Whilst the bund could be used as an alternative to the flood wall, it would not be possible to use it over the complete frontage due to space restrictions in a number of locations. This would certainly be the case of the length from about half way between the baths and Vernon Ave to the location of the Clontarf Yacht Club. Over this length it is likely that the re-curve wall would be required to facilitate the existing use of space in the area. For the purposes of costing this option, this has been considered to be the case and the estimated capital cost of this combined earth bund and re-curve wall option would be €9,635,000 including vat.

The above costs are based on insitu construction techniques. The option to precast the set back wall does exist and would probably reduce the construction costs indicated slightly.

Option 3 – Replacement of the Secondary Wall

Option 3 represents a variation on the set back wall described in Option 2 above. The current wall follows the existing ground levels along the linear park, rarely rising more than 400mm above the existing ground levels. Option 3 would involve replacing the low level wall with a new wall to a level of 3.75mODM at crest level, or 0.5m above the ground levels, where the existing ground levels are already high. Existing pedestrian openings would be closed thus reducing the current flood risk that exists along this stretch of coastline.

As with options 1 and 2 the existing low rock and rubble revetment that is located at the most westerly end of the Clontarf Promenade, adjacent to Alfie Byrne Road will be removed. A new toe sea wall detail to a level of 3.25mODM constructed with the rock revetment reinstated on completion.

Replacing the secondary low wall would still permit overtopping of the primary sea wall, as no major modifications would be undertaken. Minor repairs would however be necessary to maintain it’s structural integrity. Therefore under Option 3 the area between the set back wall and the Clontarf Promenade would become a storage area for flood water during extreme tidal events, protecting the main arterial carriageway behind, and minimising disruption to the area as a whole.

To combat the potential trapping of the flood waters within the promenade area after the flood event has passed, a series of linear drainage channels and gullies would be installed along the full length of the promenade. These will drain the water away from the promenade down through to a new discharge pipe installed under the footpath adjacent to the sea wall.

Existing openings and/or new openings would be fitted with flood gates. This approach, combined with the use of earth embankments would further protect against inundation of the carriageway.

The capital cost (Including VAT) for this option would be €7,950,000.

Details of the works required within Option 3 are presented in the drawings 9M2793 – 420 to 424.

It should be noted that as with the other options, the above costs include for a new 8m wide promenade to accommodate the Sutton to Sandycove cycle way. Again from a flood defence point of view, it would not be necessary to provide such a promenade and indeed at the new wall is set along the back of the park not alterations to the front walkway would be necessary. Some damage and reinstatement of the existing back cycleway is likely. For this option if the costs of the new promenade are removed the cost of this option from a flood defence point of view only would be €4,255,000. This cost reflects insitu construction and might reduce if a precast option was considered.

Option 4 – Offshore Breakwater

The initial three options along this section of coastline focus on raising the existing wall, construction of a set back wall, and construction of a new secondary wall along the back of the linear park. To minimise the required defences in any of these three options, a fourth option has been investigated. This option combines a breakwater or a series of breakwaters with defences as given in option 1. The breakwater(s) will reduce wave height along the frontage, thus allowing lower defences.

The design of the breakwater(s) should take a number of aspects into consideration:

- costs involved with the breakwater(s) versus costs saved due to lower defences along the promenade;
- visual / landscape impacts along the promenade as well as on Dublin Bay;
- impacts on boat recreation (Clontarf Yacht and Boat Club);
- impacts on tidal currents and ship movement in and around Dublin Port;
- impacts on local tidal currents, sedimentation and water quality;
- impacts on the SPA along the Clontarf frontage.

In designing the breakwater the hydraulics issues have been taken into consideration in basic terms, the impacts on morphology however have not been investigated. Obviously the breakwater(s) will affect the bathymetry: erosion near the breakwaters and sedimentation on the lee sides will occur to a certain degree.

To determine the critical design conditions, combinations of water levels and wave heights have been investigated: a tide with a return period of 200 years with waves with a return period of 1 year and a tide with a return period of 10 years with waves with a return period of 10 years. Nearshore wave climate was obtained from the wave transformation modeling work undertaken as part of the DCFPP. At this stage, development of the breakwater layouts and hence their impacts on the nearshore wave climate within the harbour have been considered using simple diffraction techniques set out in the Shore Protection Manual. Before such an option is taken forward to design, it is recommended that it is checked using appropriate wave diffraction models and also tidal hydrodynamic and water quality models.

Furthermore simple calculations were made to investigate wave increase on the leeward side of the breakwaters based on the fetch lengths available.

Four possible configurations have been investigated.

The first option (4A) is a breakwater that partly stretches from Bull Wall across towards the DPC's Terminal 1. This still leaves a relatively large opening for waves to approach the western end of the Clontarf frontage. To effectively reduce wave height along considerable sections of the promenade, the breakwater is taken at a length of 650 m. The opening left is therefore some 500 m wide, providing sufficient opening for tidal currents and recreational boat movement.

The second option (4B) replaces the continuous breakwater in 4A with a series of short detached breakwaters with a number of openings between each. This option extends further across the gap between Bull Wall and Terminal 1. The total length of the breakwater is taken at 4 times 150 m, with gaps of 75 m and a larger gap of approximately 250 m near Terminal 1.

The third option (4C) is similar to the second (4B), except the outer detached breakwater is moved in a more north westerly direction to further reduce wave action along the critical frontage between the yacht club and the Clontarf baths, which is subject to more severe wave overtopping through the Mach Stem phenomenon.

The final layout considered (4D) is a further variation of 4B & C, with the two outer breakwater moved in a north westerly direction. The impact on the wave climate for both options 4C & 4D is very similar and indeed by reprofiling the layout in this way to follow the orientation of the coastline, the maximum impact of the wave climate at the frontage can be achieved. However, from a visual perspective the breakwaters positioned further out from the coastline is considered to be better.

It should be noted that the breakwater(s) will need to be out far enough to minimize the total length of the breakwater, ie far enough out to protect most of the promenade. A (cost) effective location would be where the estuary is narrowest. The drawback of this however is that on the leeside of the breakwater the fetches are such that wave heights can increase again. For example: with breakwaters positioned several hundred meters out from the coast, design wave heights will still not be less than 0.2 to 0.3 m.

With option 4A substantial reduction of wave height to less than 0.4 m was achieved from chainage 550 m to roughly chainage 1100 m and from chainage 2600 m to 3275

m. Wave reduction from chainage 1500 m to 2500 m is marginal, therefore the design as presented in option 1 may not be minimized here. Furthermore Mach Stem may occur here.

With option 4B the wave reduction is somewhat more substantial, with wave height less than 0.4 m from chainage 550 to 1000 m and from chainage 2300 m to 3275 m. Wave height reduction, whilst better than option 4A, is still relatively marginal along the stretch from 1200 m to 2200 m with respect to the open situation, again with potential occurrence of Mach Stem.

The most promising options from a wave reduction point of view is options 4C & D. Wave heights will be less than 0.4 m from chainage 550 to 1650 m and from chainage 2200 m to 3275 m. Wave heights will be reduced from 1700 to 2100 m but will still be significant enough to require a marginally reduced variation of either option 1 as described before.

In order to break the wave energy sufficiently to allow for the abovementioned wave reductions, crest levels of the breakwater(s) should be at a minimum of +3.75mODM, with a 3 m wide crest and slopes of 1:2. The breakwater would consist of a 1.5 m thick rock cover layer with rock armour of between 0.75 to 1.5 tonnes, overlying on a 0.75 m thick underlayer of rock range from 75 to 150 kg. This underlayer would be placed on a suitable core, with a geotextile between.

It should be noted that all options will have considerable visual impact on the landscape. Obviously in places the flood defences along the promenade will be lower, but the outlook on the estuary will change considerably, irrespective of the view point. Measures (signage in particular) will also need to be provided with respect to navigational safety of the recreation boating that takes place in the estuary.

In respect of the impact of the breakwater options on the requirements along the existing promenade a comparison with option 1 only has been made, and an assessment of the reduced design level for the wall estimated together with cost saves as a result. When considering the improvements in wave climate that can occur for breakwater layout 4C or D, the following new design wall level have been considered appropriate.

- Aflie Byrne Road to Clontarf Baths (Ch 550 – 1650) – new wall level of 3.85mODM, this provides a reduction of 0.25m from that of option 1.
- Clontarf Baths to 500m east of the baths (ch 1700 – 2200) – new wall level of 4mODM, this provides a reduction of 0.25m from that of option 1.
- Ch 2200 to Bull Bridge (Ch 3275) – new wall level of 3.85mODM, this provides a reduction of 0.4m from that of option 1.

The reduction in wall heights will certainly have a beneficial visual impact on the character of the park with respect to the design levels for option 1 or 2, particularly in the region adjacent to Bull Bridge. However, this benefit is to some extent over shadowed by the detrimental visual impact created by the large offshore breakwater structures required a short distance from the coastline and within the harbour area.

Details of the four breakwater layouts considered are shown on drg no 9M2793 – 430 & 431.

Rough calculations have been made regarding the costs of the breakwater options. These are estimated at € 5.1 million excluding VAT for option 4A and €5.8 million excluding VAT for option 4B, C or D.

With respect to the cost of option 1, the savings that can be obtained through the implementation of the offshore breakwaters will be through reduced wall height and the removal of the rock groyne structures required over part of the frontage from chainage 1500 to 22200. An approximate estimate of the saving is €2 million excluding VAT. Therefore the net extra cost of implementing the breakwater option over and above option 1, would be €3.8 million.

Based on the net costs involved with the breakwater option and the impact of the breakwater(s) on the visual perception of the bay and navigational safety, this option is not seen as being feasible. This will also be the case when the breakwater(s) are combined with for example option 2 also.

Discussion

Each of the four option presented above will provide a standard of protection against coastal flooding of up to and including an event of magnitude 1 in 200 years i.e. the annual average risk of flooding will be reduced to less than 0.5% as opposed to a current risk in the region of around 5 to 10%.

Obviously each option has different impacts on the area, its character and to a lesser extent local coastal processes. Option 1 to 3 consider confining the risk reduction works to within the park, while option 4 looks to minimize the impact on the park by providing structures offshore which sufficiently reduce wave action and hence the defence requirements along the park. However, it should be noted that even option 4 does not remove the need for alterations to the park and the promenade, as the existing promenade wall is considerably low.

Option 1 and 2 address the issue of flood risk along the front edge of the park confining defence works and hence the problem close to the source. However option 1 has been design to allow for some overtopping as mention above. This would only occur during the design storm event and under such circumstances the park might flood up to 0.1m in depth, although this could be slightly deeper along the front of the park and shallower towards the back. This design criteria has been adopted for the reduced wall considered as part of Option 4 also. For option 2, when the flood gates are closed, which provide access onto the new promenade, the overtopping of the defence would be minimal and restricted to wind blown spray. However, in considering both options, it should be pointed out that whilst option 2 is better in terms of overall flood defence on the extreme storm conditions (when the promenade would not be in use), option 1 does provide improved protection on a day to day basis with respect to public use of the promenade. This is because the wall along the front will be higher than currently whereas with option 2 the promenade is set at around the level of the existing wall and so some overtopping may occur onto the promenade as occurs over the wall today. This will mainly impact on the SUS requirements with respect to public use of the promenade. Whilst the disruption will be greater for option 2, it is unlikely to pose a significant disruption for more than 5 to 10 days per year. It is also likely to be restricted to specific locations where mach stem occurs and following monitoring to determine these locations it is likely that some minor works could be implemented to improve the situation.

With option 3, the park is allowed to flood completely on a design condition and indeed it will flood to a lesser extent relatively regularly in a similar way to that experience along the park today. The flooding would however be confined to the park by the new wall and flood gates at the rear. This flooding would however become more frequent with time due to the impacts of climate change. It would have perhaps the least impact on the park's character in that the new wall would be the lowest of all the options presented and the front wall would be unaltered. Nevertheless it would have the most impact on the proposed new SUS cycleway and promenade and the park as a whole, as it would be flooded more frequently that the other two options and when flooded it would take some time for the water to drain away.

Another consideration is how flexible each option is in its ability to cope with future flood defence requirements due to continued climate change. For example the options have been designed making an allowance for sea level rise based on a review of the latest guidance. However, this guidance is reviewed every 5 to 10 years and it might not be unreasonable to consider what the impacts would be if the guidance suggested that a more severe allowance should be made. In this respect those options which are set back from the line of the existing front promenade wall will be most flexible in how they can be adopted to deal with either enhanced climate change impacts or simply modification to extend their design life. This is because there is more room for modification. With option 1 and to a lesser extent option 4, the primary defence has been raised. This to some extent restricted further modifications to further larger raising along the existing line and source of the problem. With option 2 and 3, the improved requirements in respect of wall raising or other modification will be less due to their set back nature.

In respect of whichever option is considered, it is recommended that the existing footpath and cycleway along the back edge of the park and the Clontarf Road, is maintained for access purposes when storm conditions restrict use of the park and new promenade along the coastal frontage.

17.4.2 East Link Toll Bridge Options

The options for this area extend west from the junction of East Link Toll Bridge to Ringsend Yacht Club (Chainage 2812 to 3634).

The options along this section of coastline focus on raising the flood defences to protect the carriageway behind the existing rock revetment. The main issues addressed by these options are:

- Prevention of direct flood waters spilling onto the road at the lowest crest level near the toll booths
- Mitigation against wave overtopping, over the existing revetment, out onto the road and by the Toll Booth.

This location is well within the harbour and as such wave action will be small. However some modest wave action will accompany extreme water levels which will reduce the freeboard against flooding.

Each location and its options will be described in more detail below.

During the flood event of February 2002 the East Link Toll road, around the location of the toll booths, experienced significant flooding across it. This caused a considerable amount of disruption to the motorists using this main arterial road and the toll booth facilities. As a result a flood alleviation option is required to protect this important part of Dublin's main infrastructure. The following two options have been considered.

Option 1 – Combined Traffic Barrier and Flood Defence

The current traffic safety barrier forms a protective shield stopping motorists from driving directly into Dublin Harbour. It is an Armco flexural structure that is anchored at either end, and has timber posts at regular intervals along its length. This option will utilise the existing safety barrier, and enhance it to provide an effective means of flood protection as well as a safety barrier for traffic.

This will be done by using aluminium sheets bolted onto the seaward side of the posts. These sheets will extend over the full exposed height of the traffic barrier for aesthetic reasons. Steel angle and T sections will be fixed to the seaward side of the timber piles, and the aluminium panels attached to these steel sections. The sections will be rubber sealed, to reduce the risk of leakage and the bottom channel section will be set into a concrete strip running along the length of the traffic barrier.

The ground conditions in this area are of made ground built up when the original road was constructed and it is not know as yet what the properties of the made ground are. Each of the timber posts in the traffic barrier can be considered to act as a laterally loaded pile when the structure would be subjected to hydrostatic loading. Form our calculations over the location around the toll booth where the low road levels are lowest, the length of the existing timber posts would not be sufficient to withstand the loads during a 200 year design event. Therefore around the area of the toll booth, two alternative options were required in this area, to provide sufficient resistance to the hydrostatic forces from flood waters.

The first of these alternatives would be to remove the existing timber piles and install longer timber piles in their place. The second alternative option would be to raise the existing road and pavement levels in the surrounding area to reduce the head of water acting on the defence. Both of these alternatives would provide better localised lateral resistance.

The capital cost (Including VAT) for this option has bee estimated at €440,000 for option 1 and €570,000 for option 1a.

Option 2 – New Flood Wall

This option requires the temporary removal of the safety barrier that protects the road and rearrangement of the rock revetment adjacent. The new wall will be placed on the back edge of the existing footpath. Due to the varying road and ground levels several different sections of reinforced concrete wall will be used to reduce the visual impact, but provide the maximum flood protection.

Once the wall has been constructed, the traffic barrier will be reconstructed on the landward edge of the existing footpath. This arrangement will allow pedestrians to move between the new flood wall and traffic barrier.

The new barrier will be fixed into new reinforced concrete ground anchors at either end, and will over lap with the existing set back traffic barrier at both the East Link Toll Bridge and at the Ringsend Yacht Club, to ensure adequate traffic safety.

The capital cost (Including VAT) for this option is estimated to be €1,690,000.

Details of the works required for both these options are presented in the drawings 9M2793 – 500 to 510.

17.4.3 ESB Poolbeg Power Station Options

The options for this area extend along the coastal stretch on the south side of ESB Poolbeg Power Station (Chainage 4356 to 5250).

The options along this section of coastline focus on raising the flood defences to protect the southern side of ESB Poolbeg power station grounds. The main issues addressed by these options are:

- Reducing wave overtopping, over the existing rubble revetment, out onto the road.
- Closing gaps, where applicable, in the existing earth embankment around the perimeter of the power station, to reduce water egress.
- Improve the existing rubble revetment to prevent coastal erosion.

Each location and its options will be described in more detail below.

This section of coastline comprises of a series of rock and rubble revetments at the top of a sandy beach, with a low earth mound, protecting a minor coastal road with levels along its length at around 4mODM. Behind the road the perimeter of the power station land has been protected using high vegetated earth mounds, which are between 1.5 and 2m above the road level.

During the flooding of February 2002, the power station land was not significantly flooded. However the perimeter road leading around the power station was significantly overtopped by wave action and water entered the power station land through various gaps in the earth mound, and most notably the water flowed back along the road and through the main entrance to the power station land.

It should be noted that the biggest gap in the high perimeter earth bank is around 140m long and exists along the frontage where a Bord Gáis depot exists. It is not known if this depot was flooded in February 2002, and if so what effect if any flooding would have on the depot. However, for the purposes of this study it is assumed that while it is acceptable for the road to flood, any waters entering the Bord Gáis depot and the power station land is not acceptable.

As flooding is purely a function of water overtopping at these locations the options have been developed around preventing significant overtopping occurring directly opposite these gaps. To achieve this, the options propose the use of overlapping earth mounds, positioned on the seaward side of the road, at the crest of the revetment. The mounds would have a crest level of 4.5mODM. On the seaward face of the raised earth mound a new rock revetment should be constructed. This will protect the new and existing earth mounds and provide a more stable flood protection barrier in the long term. In addition to prevent water from out flanking the new mounds and simply running down the road from adjacent stretches, it is proposed that small ramps should be constructed on the road within the foot print of the new mounds to prevent remove this risk. These ramps would not need to be very high, just higher than the road kerb levels to encourage the water to run back into the sea rather than along the road.

Where any minor gaps exist in the earth mound, bounding the ESB Poolbeg Power Station, these will be filled and vegetated to blend in with the surrounding area.

The capital cost (Including VAT) for this option is estimated at €2,910,000.

Details of the works required for these options are presented in the drawings 9M2793 – 600 to 610.

17.4.4 Sandymount Strand, North of Existing Promenade, Options

The options for this area extend from Sean Moore Park to the northern tip of the existing Sandymount Promenade (Chainage 2114 to 2808).

The options along this section of coastline focus on raising the flood defences to protect the existing carriageway and properties behind the existing masonry wall. The main issues addressed by these options are:

- Reducing wave overtopping, over the existing masonry wall.
- Re open the pedestrian gaps and provide better access to the beach, if possible.
- To provide an improved public amenity along this section of coastline, if possible.

Each location and its options will be described in more detail below.

During the February 2002 event, large amounts of water overtopped the existing masonry sea wall and flooding the footway, carriageway directly behind and in some locations many properties also. There are a number of existing pedestrian gaps along this stretch of coastline, which allowed the flood waters to pass through the wall and onto the road behind. These have since been temporarily blocked. In addition near the Sean Moore Park the existing masonry wall is very low and observation during the event confirm that significant volumes of water overtopped this low wall causing extensive flooding of the road and area adjacent to the park.

As a result of the floods an extensive amount of beach material was washed up onto the adjacent Sean Moore Park and out onto the road. The beach level itself was reduced significantly.

It is recommended that a full structural analysis of the existing wall is undertaken, prior to the selection of any of the following options. This analysis should take into account the loading and stress that these options may have to the existing wall.

Option 1 – Raise Existing Masonry and Stonework Wall

This option focuses on minimising alterations to the area by raising the existing wall, which would need to be raised to +4.6mODM. It is proposed to provide a new 'bull nose' on the crest of the raised wall to deflect wave action. The existing temporarily closed pedestrian accesses will also be permanently closed, using the same wall composition adjacent. However, as with the Clontarf frontage the issue of Mach Stem could be a problem along this stretch of defence and the design level of 4.6mODM is based on conventional overtopping criteria. Mach Stem could still result is some overtopping of this new defence and therefore to minimise this risk it would be necessary to provide some means of preventing or minimising the development of Mach Stem along this wall. This could be achieved by the construction of a number of short perpendicular rock structure along the length of the wall. These would encourage the Mach Stem wave to break thus preventing it from building to such an extent that would result is enhanced overtopping.

The existing wall composition and structural condition and capacity of the existing wall will need to be investigated in detail, prior to the raised crest being constructed. Depending on the findings of that investigation it might be necessary to beef up the bottom of the existing masonry wall which in places will be raised by over 1 m, and thus exposing the wall to significantly greater wave loads than is currently the case. In addition raising the wall by this amount will impact on the visual amenity of the area by significantly reducing the view across the bay for pedestrians and vehicles behind the raised wall and from the houses located nearby.

The capital cost (Including VAT) for this option is estimated at €2,260,000.

This cost does not include for works that might become evident following the results of the structural inspection survey. In addition it does not allow for any mitigation works with respect to loss of public amenity through the increased wall level, such as raising footway or road levels to maintain the view across the bay.

Option 2 – Raise existing wall to 4.2mODM and build new rock revetment at toe

In Option 1 the wall is raised to 4.6mODM, which could be unfavourable from an aesthetics point of view. In Option 2 it is proposed to build a rock revetment along at the toe of the wall. This would enable the crest level of the wall to be reduce to 4.2mODM. The wall will then be protected by a 5m wide rock revetment at 3.25mODM which would help break down the waves and would prevent mach stem build up along the wall.

Construction of the rock revetment is likely also to at least maintain and may even encourage some accretion of the beach level along the toe. Evidence of this is already in place at the toe of the rock revetment along the existing promenade. The beach levels along the toe of the existing wall are much lower.

This rock revetment will link the existing promenade in the south to Sean Moore Park in the north. However, the revetment is not designed to be accessed by the general public, so the existing pedestrian gaps that were initially closed as a result of the February 2002 flood event will need to be closed permanently.

The capital cost (Including VAT) for this option is estimated to be €3,100,000.

Option 3 – Extend Promenade North to Sean Moore Park and raise existing wall to 4.2mODM

In order to improve the public amenity and reopen the blocked pedestrian gaps that will be closed, as in options 1 and 2, and using the layout of the existing promenade, it is intended to extend the existing promenade northwards to meet up with Sean Moore Park.

The general arrangement of the new promenade will be the same as the existing promenade, with the revetment continuing as the primary wave breaking defence. Backed by a 3.5m wide footway, at a level of 3.25mODM, placed adjacent to the crest of the revetment. This in turn will be backed by a new linear park rising to a level of 3.35mODM at the toe of the existing wall. In various places the existing masonry wall will need to be raised to a crest level of 4.2mODM.

This option allows the previously blocked up pedestrian accesses to be re opened, and they will be protected from flood water ingress by using a combination of ramps and walls. These will ensure that any still water egress onto the carriageway behind the wall is avoided.

The promenade will stop at Sean Moore Park, with the revetment tying into the existing rock and rubble revetment already protecting the south eastern perimeter of the park. At present there is an informal vehicular access at this point. This access will be replaced by a concrete ramp, which will be protected either side by the rock revetment. At the crest of the ramp, facilities for the installation of damboards during extreme flood events should also be provided, to prevent it becoming a flood path out of Sean Moore Park to the road behind.

This option does not propose to protect the frontage of Sean Moore Park. Instead, the park will be left open, allowing wave overtopping into that area. However, the existing ground levels in the park fall towards the beach and should be sufficient to reduce any standing water that may occur. The options proposed in the emergency works report, see following sections, should be undertaken in line with this and any option proposed along this frontage to prevent any escape of water from the park through the existing access gap (currently temporarily blocked).

This option also provides considerable opportunity for the SUS project, as it will provide a new continuous promenade over much of the Sandymount frontage on which the newly proposed cycleway and footpaths can be constructed.

The capital cost (Including VAT) for this option would be €8,145,000.

Details of the works required for these three options are presented in the drawings 9M2793 – 700 to 750.

Access Gap Sean Moore Park

In addition to each of the options mentioned above, a number of options to close a pedestrian access gap into Sean Moore Park opposite Marine Drive were developed as part of an earlier emergency works report. This work should be incorporated within each of the above options. Details of the options are set out below.

Options Considered

Option 1

Option 1 (figure 6.1, Appendix Q) requires the removal of the original disabled ramp, and a larger system of ramps and platforms put in its place. This new ramp system should comprise a two staged ramp separated by an intermediate platform running parallel to and against the existing park wall. The width of the ramp should be 1.2 metres in line with the recommendations of the building regulations referred to above, with each stage of the ramp laid at a slope of 1 in 12. Access would be obtained from the western end, i.e. access along the ramp would be west to east. The top platform should be 1.5m long in the direction of travel from the ramp and should be at a level of +3.65m ODM. The platform would then turn through 90O (in the direction of the park) and cross the masonry wall to the top of a second ramp leading into the park. The ramp leading into the park should be at a slope of 1 in 20, which due to higher ground levels within the park still complies with legislation.

The option also includes for a set of steps leading up to the top platform from the road side. In addition the option includes for the removal of the existing vehicular access gate adjacent to the wheelchair access, which is currently blocked up, and replacement with a new masonry wall over that width. It is considered that this access is no longer required as other vehicular access to the park can be obtained from the Sean Moore Road.

For ease of construction and due to the small quantities of materials involved, costs for the structure have been based on the use of mass concrete.

The cost of this option is estimated to be €12,500 including Vat at 12.5%.

Option 2

The second option (figure 6.2, Appendix Q) consists of a parallel and mirror image system of access ramps either side of the existing wall. The road side ramp consists of a series of ramps at a 1 in 12 slope and separated by an intermediate platform. The ramp should be 1.2m wide and access obtained from the east, i.e. access along the ramp would be east to west. The top platform would be 1.5m long in the direction of travel. The top platform should extend 2.75 metres in width across the existing masonry wall to form the top of the inside ramp. The inside ramp would be a mirror image of the road side ramp, see Figure 6.2.

For non-disabled users, a step should be cut into the existing masonry wall at the location of the intermediate platform. As with Option 1, the existing vehicular access gate should be removed and a new masonry wall constructed.

As with Option 1, the cost estimate for this scheme is based on the use of mass concrete.

The cost of this option is estimated to be €13,500 including Vat at 12.5%.

Option 3

The third option would involve the removal of the existing ramp, and adjacent vehicular access gate and the construction of a new ramp system and masonry wall, as shown in Figures 6.3 & 6.4, Appendix Q. The start of the new ramp system would be similar to the existing ramp located on the pavement on the outside of the park. This ramp should be 1.2m wide and at a slope of 1 in 12. At the top of this ramp and just inside the park, a new intermediate platform should be constructed to provide a turning area leading onto the second section of the ramp system, which would run parallel to the existing masonry wall on the inside of the park. That ramp should be constructed at a 1 in 20 slope and would lead onto the top platform, constructed at a level of 3.65m ODM. The top platform would provide the second turning area leading onto the third and final section of the ramp system which would be orientated perpendicular to the second ramp and lead into the park. The third ramp should be constructed at a 1 in 20 slope and would exit onto a newly constructed tarmac path, which in turn would join with the existing path in the park. Each of the turning platforms, should be 1.5m long in the direction of travel of each ramp.

The removed vehicular access gate should be replaced with a new masonry wall, which should extend around the intermediate platform and along the second section of the ramp system and tie into the top platform. The extension of the masonry wall around the new ramp system is necessary to seal off the structure and remove all possible flood paths.

The cost estimate has been based around the use of mass concrete as with Options 1 and 2 above. From a safety perspective safety railings should be erected around the perimeter of the ramp structure.

The cost of this option is estimated to be €18,500 including Vat at 12.5%.

Option 4

The fourth option would involve the removal of the existing ramp, and adjacent vehicular access gate and the construction of a new parallel ramp system and masonry wall, as shown in Figures 6.5, Appendix Q. In this option the new ramp structure is entirely contained within the park, which is a considerable improvement from an aesthetic and a health and safety perspective, as the current perpendicular intrusion onto the road side pavement which is the extending ramp and railing system is a potential hazard. For this reason also, Option 3 would be an unacceptable option.

With option 4, access to the park would be via a series of parallel ramps systems separated by a new masonry wall. The entrance to the ramps system from the pavement would be through the location of the existing gap in the masonry wall and would lead into a ground level platform, which in turn would provide access to the first series of ramps. The ramps should be 1.2m wide and run parallel with the existing masonry wall at 1 in 12 slopes, with an intermediate platform midway. One series of ramps would lead onto a top platform constructed to a level of 3.65m ODM and the other lead down the other side.

The masonry wall separating the two ramps and tying into the top platform is required to seal the structure and remove any possible flood path. Safety railings should be placed around the perimeter of each of the ramps. Adjacent to the ground level platform and within the new masonry wall, a new set of access steps should be constructed for those more able bodied persons who may not wish to use the access ramps.

As with the other options this one has also be costed as a mass concrete structure and has been estimated at €17,500 including Vat.

Discussion

Each of the options presented above are viable access structures that comply with current disabled access legislation. Each is simply a different layout of the same overall principal. However, of the four options, Option 3 is the least favoured due to the obtrusive lead in ramp and safety railings that cut across the existing footpath. These could represent a hazard at night. For this reason this option is not recommended. Whilst not as obtrusive, Option 1 also has a set of access steps and safety railings that protrude onto the pavement making this option also unfavourable.

Of the remaining two options, option 2 could potentially present a similar hazard within the park over its lower ramp, although to a lesser extent. Whilst option 4 is slightly more expensive than 2, the whole structure is contained within the park making it visually pleasing and is very symmetrical removing any hazardous components. This option is therefore the most preferred.

17.4.5 Sandymount Strand, Existing Promenade Options

The options for this area extend from the northern tip to the southern tip of the existing Sandymount Promenade (Chainage 989 to 2114).

The options along this section of coastline focus on raising the secondary flood defences to protect the carriageway and properties behind the existing promenade and masonry wall. The main issues addressed by these options are:

- Reduce flood water egress out onto the road behind the promenade.
- To provide an improved public amenity along this section of coastline.

Each location and its options will be described in more detail below.

During the flood event of February 2002 this existing public promenade was inundated with floodwaters. This floodwater then flowed through the pedestrian and vehicular gaps in the secondary masonry wall, out onto the road behind, and in some case into the gardens of residential properties.

The following options are designed to deal with the flood impacts around the specific pedestrian and vehicular gaps, so as to maintain the promenade and its existing public amenity. These designs also allow continual beach access and where possible reduce any undue visual impact to the area. In most cases the existing masonry/stone wall is at a sufficient level, +4.2mODM, but the crest level of the wall north of the Martello Tower (Chainage 1662 to 2114) is below the required level set out in the design criteria and as such will need to be raised accordingly.

Pedestrian Gaps 1, 2, 6, 11 and 14

In order to maintain the public amenity of the existing promenade the pedestrian gaps along it will need to remain open. The gaps along the promenade will be protected from ingress by flood waters using a combination of walls, ramps, and where possible, steps.

Where necessary the existing masonry wall will be raised to +4.2mODM. A new offset parallel wall will be constructed across the existing gap, approximately 2-3m on the seaward side, again to a crest level of +4.2mODM. A new concrete platform will be constructed matching the existing pavement level, and a set of ramps and steps will be installed with the appropriate gradients for disabled access and railings for pedestrian access. The perpendicular footway will then be re-routed to tie in with the new access measures at the pedestrian gap.

The new parallel wall infront of the gap will prevent wave run up simply rushing through the open gap. The steps and the ramps will prevent any flood waters which build up on the promenade from escaping through the gap. Wave action at the back of the promenade will be significantly reduced due to the level and width of the promenade and any build up of water across the promenade will now be retained within it and return to the sea during wave troughs and on the falling tide.

Pedestrian Gap 5

This existing pedestrian gap has already been closed and is no longer open to the ingress of flood waters or the general public. The existing wall is of a sufficient standard and crest level, therefore no additional options will be considered.

Pedestrian Gap 7

This is a partially closed pedestrian gap that is not widely used as an access point to the promenade. Therefore due to its close proximity to pedestrian gap 6, it is intended to fully close this gap to the same wall crest levels adjacent. Again the crest levels of the adjacent walls are sufficient to reduce the potential flood risk to the road behind.

Vehicular Gaps 3 and 4, Vehicular Gaps 12 and 13 Options

There are three car parks located along the Sandymount Strand Promenade. Vehicular Gaps 3 and 4 are the gaps associated with the southern most car park, and vehicular Gaps 12 and 13 are the gaps associated with the northern most car park. The third car park near the Martello Tower is discussed in the next section. The current car parks have an informal entrance and exit set up. Using two separate options these existing entrance and exits will be formalised into a one way system for all the car parks. The one way system will ensure traffic enters at the northern gap and exits at the southern gap.

Option 1

The first option uses a perimeter wall constructed around the existing car park built to a crest level of +4.2mODM. At the two gaps a wide ramp is put in place across the full width of the gap and tied into the existing ground levels at either end together with a wall on either side. The wall around the gaps and across the front of the car park would prevent significant wave run–up across the car park. The ramps at either end would need to be build to a level that exceeded the tidal still water level and also the car park ground levels. Thus effectively 'sealing' the existing gaps. Two formalised pedestrian gaps would be required through the new car park perimeter wall to allow pedestrians to exit the car park and use the promenade. These gaps would obviously allow wave run up into the car park. However, the gaps would be small and sufficiently offset from the access ramps in such a way that wave action would be effectively nullified with respect to the ramp locations. The crest level of the new pedestrian access would need to be lower than the vehicular access to ensure and encourage any flood water build up in the carp to drain back out to sea. Obviously during a storm event the car park will be flooded and signage would need to be erected warning users in advance not to park there.

Option 2

The second option is similar to the first but aims at keeping the view from the car park more unobstructed by not having a new perimeter wall along the front of the car park. As such more careful detailing is required at the vehicular access gaps to ensure flood waters do not escape. To achieve this it is proposed to build a much narrower and slightly higher ramped system bounded on either side by a shorter localised wall. The ramped access structure would be curved at both the entrance and exit locations, with the outer wall overlapping slightly on the inner wall. The entrance and exit points on the seaward side would also be kept to a minimal width to reduce the risk of wave run-up in the location of the ramps. On the seaward side of the new ramps the adjacent area would be vegetated to disguise the new walls and also break up any wave run up in this area.

This option and layout would result in the loss of a number of parking spaces at either end of the car park but this could be offset by slightly extending the overall length of the car park. Costs for this have not been considered at this time. Whilst this option utilises a larger section of the car park, at either end, the narrowness between the walls ensures that the one way system is adhered to within the car park, and reduces any potential water egress out onto the road. As with option 1 it would be necessary to warn users when the car park was at risk of flooding.

An additional safety measure, which could be incorporated into either option 1 or 2 at minimal cost, would be the provision of slots for dam boards at the top of the ramps. This would also be a useful deterrent to block the entrance access ramp prior to the storm event, which would prevent users from accessing the car park, while allowing cars parked in the car park at that time to leave via the exit ramp.

Vehicular Gap 8 Options

This vehicular gap, positioned just to the south of the Martello Tower serves as an entrance and exit to an existing smaller car park. Like the other car parks along this coastal promenade the options to 'close' this gap involve using walls and ramps to stem any potential flood waters and reducing the risk of their egress onto the carriageway behind.

The **first option** uses a considerable area, within the car park, around the gap to construct a central wall, which two ramps are placed around. A rudimentary one way system is installed to reduce the flow of two way traffic.

A perimeter wall is constructed around the outside of the car park at a crest level of +4.2mODM at either end and +4.1mODM along the middle section. Two pedestrian access gaps would be required through this perimeter wall to allow access to the beach from the car park. Vehicles will enter and exit the car park from the same gap, but turn left into the car park and over a ramp, so that the central wall is on the offside. Whereas to exit, the vehicles will exit over a curved ramp, with the central wall on the offside, and then through the gap. As with the other car parks this system also ensures that the car park can be used to minimise wave effects.

The **second option** is similar to the initial option but requires land take of an area just to the south of the existing car park. Due to the large amount of area used by the new central wall and one way ramping system, it is intended to extend the car park into this new area, taking approximately 500m2. The new car park will still utilise the same one way entrance and exit and will effectively not lose vehicular spaces in an effort to reduce the flood risk onto the road.

The **third option** uses the curved entrance and exit design, similar to that for the second options designed for the north and south car parks. This option also requires using some of the adjacent promenade public open space to extend the car park southwards and involves breaking a second gap through the exiting masonry wall. The existing gap would therefore become the entrance gap and the new gap the exist gap. Thus the curved narrow entrance and exit gaps with ramps are constructed to restrict the movement of two way traffic and reduce any potential water egress out onto the road.

Pedestrian Gaps 9 and 10 Options

The area around the Martello Tower, at the central point of the Sandymount Promenade, is an open area that has no existing flood protection in place. This creates a potential flood risk in this area, and to the carriageway behind it. At present the current use is a café, which is only used during the summer months.

The options for this area maintain the existing course of the footpaths and use a series of low walls with a crest level of 4.2mODM and ramps with a level of 3.65mODM to allow continued pedestrian passage between the carriageway and the promenade. Two layouts have been considered.

For the **first layout** small ramps have been placed on the access paths through the new wall. Due to the nature and orientation of the access path and hence ramps it would be necessary to use dam boards across the gaps on the ramp platform, during extreme weather conditions, as wave run up is likely to still run over the low level ramps.

The **second layout** utilises the same arrangement for the first option, but re-orientates the access paths and hence gaps with respect to the critical wave direction. It is proposed that for very extreme flood events dam boards may need to be installed across the gaps to raise the height of the flood defences further.

The capital cost (Including VAT) for this option would be €1,365,000.

The costs above allow for sealing of the secondary defence along the line of the back wall and gaps. However, it is likely that following future storms damage to the existing light revetment along the front of the promenade will occur and repair or strengthening works will be required. It is therefore recommended that a further sum of around €900,000 including vat should allowed for future strengthening works.

Details of the works required for these three options are presented in the drawings 9M2793 – 800 to 861.

Merrion Gates

As part of the emergency works report a number of option were developed to improve the flood defence at Merrion Gates while at the same time maintaining the access to the beach. These options are presented below.

Options Considered

Option 1 – Flood Wall & Revetment

Option 1 provides a new vehicular access ramp together with a reinforced concrete wall and small rock revetment to protect against wave action. Details are presented in Figures 5.0 and 5.1, Appendix Q.

The ramp is orientated in such a way so that is exits onto the higher grass bank that currently exists parallel to the Dart Railway line boundary. The ramp has been designed with 1 in 12 slopes and a 2.5m wide crest. The level of the crest should be +3.9m ODM. The road side slope of the ramp and crest should be constructed from tarmac and tie into the existing road, while the seaward slope should consist of grass cell blocks such as Grassguard. The access part of the ramp varies from 7 metres wide at the road to 5m wide at the grass bank. The ramp on either side of this central access portion should be topsoiled, grass seeded and landscaped with some shrubs and trees.

The RC wall should be constructed a short distance seaward of the access ramp, as shown, and should be approximately 25m in length and constructed to a level of +4.4m ODM. A 10m length of safety rail should be placed on top of the wall over the central part to coincide with the crest of the landscaped ramp that backs onto the wall.

A small rock revetment should be constructed on the seaward side of the RC wall, as shown. The primary armour stone should be between 225kg to 375kg with a W50 of 300kg and a D50 of 0.5m. The armour layer should be 2 rocks thick and placed on an underlayer also 2 rocks thick of W50 equal to 30kg. The beach should be excavated out and the rock placed on a suitable geotextile prior to the sand being replaced and the toe buried as shown. This helps maintain the design should the beach levels fall during a storm event. The crest of the revetment should be at +4.0m ODM and be a minimum of 3 rocks thick, with the slope at 1 in 3.

Access from the existing grass bank down to the beach can be achieved a short distance beyond the toe of the new ramp and past the end of the revetment by grading a new slope down to the beach, which is approximately 400mm lower. Some additional rock should be placed either side of this new access, in a similar way to that placed around the existing grass bank.

The layout has been designed such that the crest of the new ramp will prevent still water levels from flowing through the gap and onto the road, while the revetment dissipates waves which would otherwise run up the beach on an extreme tide and overtop the ramp. On the southern side of the revetment, wave action could run up onto the grass bank, but will be sufficiently reduced that the exiting brick wall along the railway and the new ramp would trap it.

The cost of this option has been estimated at €110,000 including VAT at 12.5% but excluding design and study fee costs.

Option 2 – Flood Wall and Rock Berm

Option 2 is similar to that of option 1 in that it provides a new vehicular access ramp together with a reinforced concrete wall and wide low level rock structure to protect against wave action. Details are presented in Figures 5.2, 5.3 & 5.4, Appendix Q.

As with option 1 the ramp is orientated in such a way that is exits onto the higher grass bank which currently exists parallel to the Dart Railway line boundary. The ramp should have 1 in 12 slopes and a 2.5 metre wide crest at a level of +3.9m ODM. The road side slope of the ramp and crest should be constructed from tarmac and tie into the existing road, while the seaward slope should consist of grass cell blocks such as Grassguard. The access part of the ramp varies from about 7m wide at the road to 6m wide at the grass bank. The ramp on the railway side of this central access portion should be topsoiled, grass seeded and landscaped with some shrubs and trees.

The RC wall in this option should be constructed tight to the edge of the access ramp and tie into the corner of the existing property wall, as shown. It should be approximately 25m in length and constructed to a level of +4.25m ODM. A 15m length of safety rail should be placed on top of the wall over the central part to coincide with the crest of the new access ramp that backs onto the wall.

The rock structure in this option is a much wider lower structure, as shown in Figure 5.3, Appendix Q. The idea behind this type of structure is to provide a wide berm at a level around the design water level, across which wave action can dissipate before reaching the wall and ramp. For this type of design a single layer of riprap armour was considered to be more appropriate than the conventional two-layer design for rock revetment and breakwaters. Riprap allows for a much wider grading of rock, reducing the voids within the layer making it more suitable for single layer construction. The riprap armour stone should be between 30kg to 1000kg with a W50 of 250kg and a D50 of 0.46m. The riprap armour layer should be 1.25m thick. The beach should be excavated out and the rock placed on a suitable geotextile prior to the sand being replaced and the toe buried as shown. In this case the toe does not need to be buried as deep as with option 1, because the integrity of the overall structure is not as critical to a reduction in beach level, since there is a much wider berm structure in place. For example if the toe of the revetment should become exposed and move, it could result in slip failure down the front slope resulting in failure of the structure. In the case of the low berm structure, if the toe is disturbed, it will simply be replaced by more rock from within the wide berm.

The crest of the rock berm should be at +3.25m ODM and be a minimum of 4m wide at the seaward end of the ramp and a minimum of 7m wide from just in front of the crest of the ramp to the property boundary. The front face of the rock structure should be shaped at a 1 in 3 slope down to the toe level.

Access from the existing grass bank down to the beach can be achieved a short distance beyond the toe of the new ramp and past the end of the wall and rock protection by grading a new slope down to the beach, which is approximately 400mm lower than the grass bank. Some additional rock should be placed either side of this new grade access in a similar way to that which currently exists around the existing grass bank.

As before the design is such that the crest of the new ramp will prevent still water levels from flowing through the gap and onto the road, while the low rock structure will dissipate wave action which might otherwise run up the ramp. The wall will prevent wash through and collect spray. On the southern side of the wall and rock, wave action could run up onto the grass bank, but will be sufficiently reduced such that the exiting brick wall and new ramp would trap it.

The cost of this option has been estimated at €115,000 including VAT at 12.5% but excluding all design and study fees.

South of Merrion Gates

South of Merrion Gates there exists a masonry revetment with masonry crest wall. The southern limit of DCC's boundary is some 620m south of Merrion Gates and this masonry coast protection and flood defence structure protects the dart and rail line which runs south from the city. At present the crest wall is reasonably high with levels varying from about 4.14 to 4.39mODM. The higher level provide a standard of protection in excess of 200 year while the lower levels just fall below this recommended level of protection. The situation becomes worst with sea level rise and as such the wall would need to be raised slightly over its length to provide a new uniform defence level of not below 4.37mODM. This would provide protection against a 1 in 200 year event up to 2031.

The estimated capital cost of this work (including VAT) is €550,000.

17.4.6 River Liffey Options

Upper River Liffey – Island Bridge Weir to Sean Heuston Bridge

The options for this area extend from Islandbidge Weir to Sean Heuston Bridge (ch 0 to 1557).

The options along this section of the River Liffey focus on raising the flood defences and closing off potential flood paths to protect the adjacent businesses and properties. The main issues addressed by these options are:

- Reduce tidal flood water egress out into the waterside properties on the existing quays.

- To improve the condition of the existing quay walls as necessary.

Each location and its options will be described in more detail below. To give a full overview of the area a short description of the length is given first followed by an indication of whether there is a risk or not. This helps to be clear about the location for which the option relates.

Sean Hueston Bridge to Rail Bridge Upstream

The quays along this section of the River Liffey vary from hard flood defences to soft vegetated areas. Upstream of Sean Heuston Bridge on the right bank until the railway bridge, the defence consist of a combination of high masonry quay walls and natural vegetated banks. The level of these masonry quay walls and bank levels is high having a minimum crest level of around 5mODM, therefore no tidal flood protection options are required for this bank over this section

On the left bank between Sean Heuston Bridge and the Railway Bridge there is a mixture of hinterland uses. Just upstream of the Sean Heuston Bridge is a recently constructed apartment building. The quay levels fronting the apartments are between 2.6m and 3mODM. This level is lower than the 3.5mODM 1 in 200 year tidal flood risk. However, the secondary wall behind, which protects the apartments and provides the threshold levels of flooding for the apartments is 1.5m above this level. This means that this area is currently protected against tidal flooding.

Upstream of the apartments is an existing high vegetated bank, with crest levels of 10mODM. Between this high bank and the railway bridge is another new block of residential apartments. The primary flood defence is a concrete walkway with a crest level of 3.5mODM, situated above a gabion quay wall. The apartments have been set back, and raised up another 0.5-0.75m above the walkway and as such it is considered that no tidal flood risk exists at present. However due to access restrictions ground levels along the edge of the setback buildings could not be obtained. Whilst a tidal flood risk is not perceived here it is recommended that ground levels around these apartments are checked as part of any future fluvial flood risk investigations in this area.

Therefore the quays on both banks between the Sean Heuston Bridge and the Railway Bridge are sufficiently well protected against the 2031 1 in 200 year tidal flood risk event. Therefore improvement of tidal flood defence options is not required along this length.

Railway Bridge to Sarah Bridge

Upstream between the Railway Bridge and Sarah Bridge, the tidal 1 in 200 year flood risk in 2031 is 3.5mODM. The right bank upstream of the Railway Bridge has a primary flood protection in the form of a concrete wall with an approximate crest level of 2.6mODM. The secondary flood protection behind this wall is a high masonry wall, with an approximate crest several metres above the primary level. On the right bank just down stream of Sarah Bridge there exists a compartment complex. The defences along here consist of a gabion river wall with level of around 2.6mODM. However, the ground level rise away from the river wall and the lower compartments have been constructed such that the ground flood window levels form the lowest flood defence level. These are around 1.4m higher than the river wall level and as such it is not considered there is a flood risk at present. Actual levels of the property windows were not obtained for access reasons and these levels should be checked in respect of any future fluvial flood risk across the area.

The left bank is a residential area, with no obvious flood protection. The gabion quay wall has a concrete cap walkway running along the quayside, the level of which is between 2.5m and 3mODM. The raised grassed area behind has levels of +3.6mODM rising up to +4 mODM nearer the properties themselves.

Therefore the quays on both banks between the Railway Bridge and Sarah Bridge are sufficiently protected against the 2031 1 in 200 year tidal flood risk. Therefore improvement of tidal flood defence options is not required along this length. Potential does exist however for these areas to be at risk from fluvial flooding and this should be checked in the future.

Sarah bridge to Island Bridge

Between Sarah Bridge and the Islandbridge Weir the banks are a combination of vegetated quays and recently built concrete walls as part of new residential developments. The 1 in 200 year tidal flood risk level in 2031 is 3.5mODM. On the left bank, just upstream of Sarah Bridge there is a public open space, which has no existing flood protection. The ground levels are 3.25mODM and the existing wall upstream does not continue to Sarah Bridge.

This area forms a gap in the flood barriers, providing a flood path behind to the residential areas. Therefore flood protection for this section, a new earth bank is proposed, which is to blend in with the public open space, and form the flood protection. The crest level will be 3.8mODM, with 1 in 2 slopes either side. Once it has been installed the bank will be vegetated to match the existing surroundings.

Further upstream, a new concrete wall has been constructed as part of a new residential development. The crest level of this wall is at 4.8mODM, with the ground levels at 3.8mODM. Between this section of the River Liffey and the Islandbridge Weir there are several properties backing directly onto the river. Direct access to these properties was not possible during the project survey and so level information along these properties in respect of their flood risk is not known. Based on visual inspection and levels of adjacent defence it is believed that there may be an isolated risk of tidal flooding to some of these properties. However, it is much more likely from the fluvial river modelling results that there is a fluvial flood risk to these properties and this should be further investigated.

On the right bank upstream of Sarah Bridge there is a large building that backs directly onto the river, with high crest levels. Upstream of this is heavily vegetated masonry quay wall, with a private road and a new residential apartment complex behind it. The crest level of this quay is 3mODM, and consists of a 5m wide grass verge at the crest with the road behind. The option to provide flood protection along this section will use a new earth bank placed within the confines of the verge, to minimise the impact of the bank. The earth bank will have a crest of 3.8mODM and have 1 in 2 slopes either side. Once the bank has been installed it will be vegetated to fit in with the surrounding area.

Due to the heavily vegetated nature of this quay wall, there is some doubt in the stability and strength of this wall, should construction take place at its crest. In this case, a further detailed structural investigation of this wall will be required to ascertain the strength of the wall and ability to withstand alteration and construction directly adjacent to it. In the meantime some repointing / grouting has been assumed for this wall.

Just upstream of this section of the river, there is a newly constructed concrete wall with a crest level of 3.8mODM. This forms the perimeter of the new private apartment complex.

Upstream of this newly constructed wall and downstream of Island Bridge Weir is a masonry wall which is high enough but is in need of repointing / grouting.

One area where further investigation will be required is the underground car park beneath the apartment buildings. It is not clear at this stage whether there are any potential flood risk paths, which may put any vehicles that are parked in the underground car park at risk.

The capital cost (including VAT) for these options would be €400,000

Mid River Liffey – Sean Heuston Bridge to Matt Talbot Memorial Bridge

The options for this area extend from Sean Heuston Bridge to Matt Talbot Memorial Bridge (ch 1557 to 4308).

The options along this section of the River Liffey focus on raising the flood defences and closing off potential flood paths to protect the carriageway directly behind the existing quay wall and the businesses and properties adjacent. The main issues addressed by these options are:

- Reduce tidal flood water egress out onto the road behind the existing masonry quay wall.
- To reduce the visual impact and maintain the existing flood protection afforded by the river quay walls.

Three locations which are of particular concern are:

- right quay between Butt Bridge and Matt Talbot Memorial Bridge
- left quay between Butt Bridge and Matt Talbot Memorial Bridge
- left quay between Rory O'More and Frank Sherwin Bridges

Each location and its options will be described in more detail below.

<u>Right quay between Butt Bridge and Matt Talbot Memorial Bridge</u>

The majority of the quay walls along the River Liffey are high quay walls with crest levels above 4mODM. One major area that requires attention is the right quay between Butt Bridge and Matt Talbot Memorial Bridge. This section of the quays does not have a quay wall like the opposite bank. Instead the quay has a chain and metal safety rail running between the two bridges.

The option proposed along this quay involves removing the existing safety railing and constructing a new masonry wall to a crest level of 4.25mODM. This crest level will be consistent with the existing quay wall levels. This will then protect the road and financial district adjacent to this area.

The cope stone will be removed and the new wall will be constructed as part of the integral quay construction. On the downstream side of the Butt Bridge there is an existing set of access steps down to the river. At present it is not known whether this access is currently in use, but if this should be the case, a new demountable flood defence, in the form of a Dutch Dam will be installed. This Dutch Dam will be left permanently stored in the ground in a locked containment unit. It can then be bought into use prior to an extreme flood event occurring.

The Dutch Dam demountable defence will be used here, as the gap is only 5m wide, and therefore it is not cost effective to use another form of demountable or temporary flood defence. Also Dublin City Council are currently using the demountable Dutch Dam flood barrier to provide flood protection across the 2 new and 6 existing boardwalk gaps along the left quays of the River Liffey. These costs are not considered here.

The capital cost (including VAT) for this option is estimated at €575,000.

Left quay between Butt Bridge and Matt Talbot Memorial Bridge

There is also a similar stepped access gap on the left quay, just upstream of Matt Talbot Bridge, again the current usage of this access is not known. As this is the case, we will assume that the access is still required, therefore we will install a new Dutch Dam, which again will remain locked away in its housing unit on the quay until it is required in an extreme flood event.

In this section 10 existing drainage holes will need to be flapped.

The capital cost (including VAT) for this option is estimated at €50,000.

Left quay between Rory O'More and Frank Sherwin Bridges

Further upstream on the left quay, between Rory O'More and Frank Sherwin Bridges, there is a length of quay wall that has recently been reconstructed by DCC Roads, due to the wall separating itself from the quay side.

In one area, the quay wall crest level is 3.21mODM, and the 1in 200 year tidal flood risk is at a level of 3.46mODM (2031). Clearly the quay wall will need to be raised to reduce the risk of tidal water ingress, out onto the quays behind. The first two top layers of masonry will be removed, and the crest level rebuilt and raised to a new crest level of 3.5mODM.

Upon visiting the site, it is evident that there is a noticeable dip in the quay wall. The reason for this is not clear. It may be that the wall has simply been built like this to follow local topography of the river banks or it may be that some undermining or settling of the footings or that the ground beneath has occurred. This will need to be investigated further to establish whether there is an underlying issue. Additional costs may be required to address this issue following such investigation works.

The capital cost (Including VAT) for this option, not including further investigation works or issues that arise as a result, is estimated to be € 55,000.

The quays on both sides of the River Liffey between these bridges flooded in February 2002, see chapter 3. This was however not due to overtopping of the wall but through unflapped outfalls along this stretch of river. DCC drainage department has confirmed that this issue has been addressed and these have been since flapped.

It should be noted that is chapter 15 a detailed photographic inspection of all outfalls along the River Liffey has been presented. This inspection noted that many outfall exist which don't appear to be flapped on the exterior. In addition it was pointed out that the quay levels at a number of other locations do fall below the design water level and whilst protected by high wall, a similar flood path through unflapped outfalls could exist. Recommendations were made in chapter 15 that the DCC drainage division should confirm the level of protection to each of the outfalls identified. Costs to deal with this investigation or resulting remedial measure if found are not included here.

Lower River Liffey – Matt Talbot Memorial Bridge to East Link Toll Bridge

The options for this area extend from Matt Talbot Memorial Bridge to East Link Toll Bridge, at the furthest downstream end of the River Liffey (Chainage 4308 to 5905).

The options along this section of the River Liffey focus on raising the flood defences to protect the carriageway and properties behind the existing promenade. The main issues addressed by these options are:

- Reduce flood water egress out onto the road behind the promenade.
- To reduce the visual impact on this public amenity along this section of the River Liffey.

Each location and its options will be described in more detail below.

The options for this location were designed, within an area along the River Liffey that has become increasingly used by the general public, tourists and services industries (i.e. boat storage, etc). Both quays have recently been renovated or are currently under renovation by the Dublin Docklands Development Authority to improve the public amenity along this section of the river. At present both quays have been landscaped using block paving, various shrubs, trees and bushes to provide a pedestrian promenade, which is backed by a road.

The quays and the promenade are backed by new and existing businesses, and there is considerably more development taking place on the right quay just upstream of the East Link Toll Bridge. Off the right quay there also exists the entrance to the Grand Canal and the River Dodder. The entrance to the Royal Canal and the George Dock exists off the left quay.

During the February 2002 flood event, Sir John Rogerson's Quay (the right quay) became flooded as a result of floodwater egress onto the quays. In addition there was also some flooding over the quays at the entrance to the Royal Canal. The still water level was and is the main contributor of flooding along this section of the quays.

Looking downstream, the quay levels on the left quay are between 3.3m and 4.1mODM. Whereas the quay levels on the right quay are between 2.8m and 3.5mODM. Therefore the options considered below will primarily be for the right quay, and will be used, where necessary on the left quay. The maximum flood barrier height required would be around 0.85mODM, which includes an allowance for freeboard.

Option 1 – Demountable Flood Defences

In order to reduce the permanent visual impact of flood defences along this section of the quays, the first option focuses on using temporary or demountable flood defences. Using the flood forecasting (FFS) and flood warning systems (FWS) a trained team can be alerted to erect these flood defences in the hours leading up to the potential flood event. Each of the three chosen solutions for this area require a varying degree of expertise and time to set up and install these flood defences.

There are several main issues with these types of flood defence that, prior to their use, are required in an actual flood event. The first requirement is training. Training in the erection of temporary or demountable flood defences is required well in advance of any potential event. This training will need to be practiced regularly, much like a fire drill, so that prior to an actual flood event the defences are set up quickly and correctly.

The second is storage. Each of the demountable defences below will require different amounts of storage space. This storage needs to be local to where the defence is to be used and large enough for the flood defence to be stored.

The third is initial cost. Whilst other more permanent options may seem expensive, they are less so due to their permanent status. They are always in place and only require minimal maintenance if maintained regularly. Whereas, the capital cost for temporary or demountable defences can often be as expensive as permanent defences.

The final issue is the aesthetics of the flood defences. The permanent flood defences, such as walls, flood gates and barriers can alter the image of a designated tourist area. This is especially true along the quays between Matt Talbot Memorial Bridge and East Link Toll Bridge. This is an area of expanding development, with a future of businesses and residential properties. The quays are already promoted as a tourist attraction and as such altering their current aesthetics may have an adverse affect to that tourist appeal. This is where temporary or demountable flood defences would be ideal. They are only used primarily for practice runs and actual flood events. Therefore reducing the visual impact that permanent flood defences have.

It is intended that any temporary or demountable flood defences will be used along the full lengths of both the north (1.4km) and south (1.7km) quays. Therefore the total length will be 3.1km. The costs included within for these temporary and demountable flood defences do not include man hours to erect the flood barrier over its lifetime, or its storage cost.

Three specific options of a temporary or demountable flood defence are outlined below.

Option 1.1 – Board Barrier

The Board Barrier is a demountable flood defence that comes in varying heights, suitable for different flood defences. It is made up of steel support angles, wooden boards, a plastic membrane and anchors. It is constructed by placing the boards onto the support angles, which are fixed to the quay, and placing the plastic membrane over the whole structure and anchoring it at either end. It is a version of the demountable defence known as “pallet barrier”, but instead of pallets boards are used.

The barrier height use for the flood protection of quays in this option would obviously vary with the height of the existing quays. At present the smallest manufactured barrier height is 0.65m, with the next largest being 1.25m. For the Liffey quays it is likely that a combination of both these barrier heights will be required. This option will provide protection against a 1 in 200 year extreme water levels up to our design milestone of 2031 and indeed beyond given the minimum barrier heights available. The Board Barrier would be constructed at the back of the quays due to the limiting nature and street furniture locations along the quays themselves. The Board Barrier will require storage off site.

Whilst this option for the whole of the Liffey quays below Matt Talbot Bridge would be ideal from an aesthetics point of view, it would require considerable operational resources to install between 2.5 to 3 kilometres of this defence prior to a potential event. Literature with respect to this demountable suggests that a team of 18 people can install 100m in 1 hour. Therefore from the lengths of quay involved it can clearly be seen that a minimum of 2 and preferable 3 operational teams would be required to ensure an installation time of around 10 to 12 hours.

The capital cost (Including VAT) for this option has been estimated at €2,100,000.

This cost involves the purchase of the system and the cost of one trial run but does not include storage costs or whole life costs of likely operational responses. The operational response cost of installing and removal after the event this demountable system could be in the region of €40,000 to €50,000.

Option 1.2 - Mobile Dam

As with the Board Barrier the Mobile Dam comes in varying heights. It is made up of a PVC tube, which is laid out along the surface of the quays. Water is then pumped into the tube, expanding the tube and creating the flood defence. There is conceivable no limit to the length of tube as each tube is available in 50m to 250m lengths.

The barrier height required along this section of the quays would be 0.9m. Therefore this design uses a tube diameter of 1.15m. This is the smallest barrier height currently manufactured. It is expected that this tube would effectively provide flood protection against all the 1 in 200 year extreme water levels up to 2051. The Mobile Dam can be placed either at the front or back edges of the quays, depending on the access to the quays, as a truck is required to ‘roll out’ the tube in a continuous fashion. The Mobile Dam will require storage off site.

As with option 1.1, this system would require significant manual resource to install and remove the system over the full length of quays required. Guidance suggests that 100m can be installed in 1.5 hours not including delivery time to site. This would require a 5 man team with 2 no 4x4 vehicles and trailers. A minimum of 3 to 4 teams would be required to ensure the system was erected within a 10 hour period.

The capital cost (Including VAT) for this option has been estimated at €1,200,000. This includes for purchase of the system and does not include for the continued operational response or storage costs required through the life of the structure. The operational response costs are likely to be in the region of €30,000 to €40,000.

Option 1.3 - Dutch Dam

The Dutch Dam is a folding aluminium flood protection system, which requires a certain amount of construction in order to place it in position. It is a variation on a permanent and temporary or demountable flood defences. It is stored permanently in the ground in a locked containment unit. During a flood event the unit is unlocked and pulled up into place and then shored up. After a flood event the shores are removed and the Dutch Dam can be folded back down into the storage unit and locked away.

It comes in varying heights. For the quays along this length of the River Liffey we would propose to use 0.8m height, which like the Mobile Dam and Pallet Barrier should protect against extreme 1 in 200 year water levels up to 2031. The storage units come in 6m lengths and within that are two 3m lengths of flood defence. Presently Dublin City Council has these flood barriers in place to provide flood protection for the new River Liffey Boardwalk gaps between O'Connell Bridge and Butt Bridge, further upstream of this location.

The capital cost (Including VAT) for this option is estimated at €6,750,000. Of this almost 70% relates to the cost of purchase and supply for the Dutch Dam. The remainder relates to the capital works required to fix the new system into a small concrete footing along the existing quay. The costs do not include whole life costs for the operational response through the life of the structure.

In terms of operational response, literature suggests that 2 persons can erect 100 metres in 4 hours. This would mean that to ensure an operational response time of less than 10hours, 12 no 2 person teams (24 persons) would be required to ensure that the dam was in place before an event. The operational response costs are likely to be in the region of €15,000 to €25,000.

To provide a total demountable option over the complete length of the lower Liffey quays from Mat Talbot Bridge to East Link Toll Bridge, would require considerable operational resources to ensure a sufficiently fast erection time over what is a considerable frontage. To bring the response time down to below the 10 hours quoted above would required much more resources and expense. Given that the proposed forecast system developed as part of this study provides a maximum forecast window of 36 hours, this could be achieved but shorter response times would be desirable. For these reasons it is unlikely that it would be prudent to use a complete demountable system over the whole frontage and in reality some form of part permanent and part demountable solution would be more realistic.

Option 2 – New Masonry Floodwall

This option requires a permanent construction along both quays of this section of the River Liffey. The floodwall would be placed approximately 2-3m from the back edge, adjacent to the road, of the existing quays. The new floodwall will have a crest level of 3.45mODM, in the areas where the existing quay levels are low. However, where the quay levels are higher than 3.15mODM the crest level will be 0.3m higher than these crest levels. Access will be through steps over the wall, and through gaps. These steps and gaps will alternate at 100m intervals along both quays. The gaps will require a flood gate or other demountable defence which can be closed during an event.

When an alert comes from the flood forecasting system the gates can be unlocked and moved into place. Sliding floodgates are proposed. Either side of the sliding floodgates two raised flower beds will be constructed. The floodgate will be housed within one of these raised flower beds, and when required, will slide into place and lock into the adjacent raised flowed bed. To maintain continuity, these raised flowerbeds will be placed at every 200m along both the left and right quays.

The capital cost (Including VAT) for this option has been estimated at €1,875,000.

Details of this option are presented in drawing 9M2793-910

Option 3 – Glass Flood barrier

The concept of this option is to provide a flood barrier, but reduce the obvious visual impact of the wall in Option 2. This option is located on the seaward side of the existing safety fence that runs along the quays. The existing rail fence has posts that are at 2m spacing and 1.1m high. This option will use the space provided between the posts to fix new adjacent posts at the same 2m spacing, thus visually shielding the new posts with the existing posts.

It is proposed to bolt a base plate at the toe of the post, weld an angled steel 'T-section' vertically, at various heights depending on the quay levels, to the base plate. Running along the toe and top of the T-Section will be channel sections. The toe channel and base plate will be bolted into the existing quay to provide horizontal support, and the top angle will be bolted to the T-section, effectively creating a rubber sealed 2m long frame. The height of the frame will be between 0.6m at the easterly end of the quay and 0.9m at the western end.

Into the frame will sit 2 panes of 8mm Pilkington Optifloat heat soaked glass, laminated together with a 1.5mm clear PVB interlayer. This arrangement will reduce the risk of a full break down of any one section of the defence, whereby the two panes laminated together will provide extra strength and support, should any damage come to either one of the panes of glass. This arrangement will allow a single 2m long sections to be replaced, rather than a whole quay length. The manufacturers of the glass maintain that the glass should be able to withstand minimal impact and still provide a sufficiently strong flood barrier. However, large items striking the barrier may cause more severe damage.

Other draw backs of this type of flood barrier could include algae growth, if the glass is exposed to high water levels for long periods. In addition, using glass as a flood barrier may encourage vandalism to the glass, in ways such as graffiti, posters or stickers being attached. However, the installation of the glass flood barrier on the seaward side of the existing safety rail may reduce these types of occurrences.

The capital cost (Including VAT) for this option would be €3,130,000.

Details of the works required for these options are presented in the drawings 9M2793 – 900 to 930

17.4.7 River Dodder Options

The options for this area extends from Ballsbridge Weir to the Liffey Confluence, at the furthest downstream end of the River Dodder (Chainage 0 to 1924). This is the tidally dominated section of the River Dodder. Right and left banks refer to looking downstream.

The options along this section of the River Dodder focus on raising the flood defences to protect the properties and businesses behind the river flood defences between each of the bridges. The main issues addressed by these options are:

- To improve the standard of protection in respect to tidally dominated flood events to a 1 in 200 year standard in 2031.
- Reduce floodwater egress out into the areas containing business, residential properties and roads behind the flood defences.
- To refurbish and improve the condition of the existing quay walls and banks as necessary.

Each location and its options will be described in more detail below.

During the February 2002 flood event several key areas where inundated with floodwater, which caused considerable damage to both the quays and the adjacent land behind. This was most notable on both banks between the Ringsend Bridge and London Bridge and on the right bank just downstream of Newbridge.

Location 1 – River Liffey Confluence to Ringsend Bridge (Chainage 1636 to 1924)

The options considered for this section of the River Dodder are focussed on the left hand bank only. The crest levels in this area fall below the flood level used to design the options for this area, which are 3.3m ODM (200 year tidal event in 2031).

The current quay levels along this stretch are between 3mODM and 3.7mODM, rising on moving upstream to Ringsend Bridge. Therefore it is proposed to construct a stub wall with a crest level of +3.6mODM, rising to +3.9mODM, for the area adjacent to the Ringsend Bridge. The wall will be constructed within the grass verge on the edge of the existing quay, thus avoiding construction in the existing vehicular rights of way.

Located in the area adjacent to the Ringsend Bridge is a recently constructed apartment complex, which uses the renovated block paved quay as a residents car park. The wall will run along the quay side perimeter and replace the existing post and chain fence.

The crest levels on the right hand bank, which protect a residential block of apartments, are considered to be above +4mODM at its lowest point, and the current condition of the defences is moderate. Therefore no options have been designed for this stretch.

The capital cost (Including VAT) for this option would be €75,000.

Details of the works required for Location 1, Option 1 are presented in the drawings 9M2793 – 1000

Location 2 – Ringsend Bridge to London Bridge (Chainage 1162 to 1627)

Both banks on this section of the River Dodder were badly affected during the February 2002 flood event. As a result, Dublin City Council commissioned a new quay wall to be constructed on the right hand side of the river. This new quay wall is a sheet pile and reinforced concrete and masonry crest construction with a minimum quay crest level of +3.8mODM. The tidal flood level, for a 1 in 200 year standard in 2031, taken from the joint probability analysis is +3.30mODM. The design level of 3.8mODM for the new right bank wall was set prior to the completion of this study and certainly provides adequate protection for extreme tidal events well beyond 2031. Had the wall been designed now as part of this study using the latest extreme water level estimates this level might have been reduced slightly to perhaps around 3.65mODM.

In respect of the left bank, there existing low masonry wall with various walls or banks on its crest. The low masonry wall looks to be in poor condition and is in need of significant repair. Before any detailed option is finalised for this length, the overall structural condition of this wall should be further investigated by means of a detailed structural assessment and analysis.

Two methods of structural investigation could be used. The first, deep core drilling, requires drilling boreholes into the quay to establish the consistency of the material behind the quay wall. Or alternatively the construction of a temporary sheet pile cofferdam could be used to examine the condition of the wall whilst the tide is high. This section of river dries out during normal low tides.

Access to this section of the river is one of the main issues with this investigation, and with the construction plant for the options below. Access on the right bank is and was much easier for the construction of the new wall. However, access to the left bank and at other locations further upstream of the Dodder are considerably more difficult.

On the left bank of this section the main access points are through Shelbourne Park Greyhound Stadium, or through a gated access adjacent to London Bridge. Whilst these may be acceptable they still do not provide access at all places along the river bank. An alternative option would be to float a barge or preferably a jack-up rig, since the river dries out at low tide, into this section of the river and work from that. The soffit level of Ringsend Bridge is relatively high and it is thought that a small jack-up rig could be brought in to this section of the river at high tide and left there for the duration of the contract.

From a visual site inspection, there are noticeable voids along the surface of the river wall on the left hand bank, which may lead to voids behind its front face. In addition the current level of flood defence varies from just under 3mODM to around +4mODM, and therefore some sections are at risk. The risk is however made worse by the condition of the low masonry wall which could collapse in time similar to the collapse which occurred on the right bank due the February 2002 event. It is considered prudent to not only raise the level of the defences but also to carry out works to ensure the integrity of the river wall in the future. It is considered that the design level for the defence along the left bank should not be lower than those of the right despite the earlier statements and as such the minimum defence level should be 3.8mODM.

Each of the following options uses driven sheet piles in some way. The first two options use sheet piles as their main form of flood defence and stabilising the existing masonry wall and river bank behind. The remainder of the options for this section of the river will require the construction of a sheet pile cofferdam. The sheet pile cofferdam will be constructed using driven sheet piles.

Prior to driving the sheet piles a thorough investigation of all the services that could exist should be undertaken together with a detailed site investigation. Most notably along this stretch of the river would be the Rathmines and Pembroke No. 1 Trunk Sewer, which crosses the River Dodder just upstream of the London Bridge.

Therefore the following new options for the left bank are proposed.

Option 1 – Low new Sheet Pile Wall with Concrete Cap and flood barriers behind

This option uses the installation of anchored sheet piles to stabilise the existing masonry wall and the areas above it, along the full length of the river bank, between Ringsend Bridge and London Bridge. The piles will be driven in as close as possible to the existing wall, anchored in place and a concrete cap will be place on the crest, to a level of 2.5mODM. Dense granular fill will be placed in the gap between the sheet piles and the existing masonry wall.

The existing length of river fronting Shelbourne Park Greyhound Stadium, has an informal loosely vegetated earth bank, with the masonry wall at the toe as its flood protection. The use of the sheet piles will maintain the toe protection, but the existing earth bank will be raised to 3.8mODM, and vegetated.

Moving upstream, there is an existing boast house, storage yard and slipway, which are used by local residents and sea scouts. The storage yard and boat house is bounded by a low masonry wall, crest level at 3mODM, which is directly adjacent to the concrete slipway. Both these elements will be removed and a new masonry wall, founded just behind the new capping beam with a crest of 3.8mODM.

Upstream of the boat house is an existing vegetated earth bank at the crest of the existing low masonry wall. Whilst this bank is in fair condition, the crest level is too low at 3.15mODM. This embankment will need to be re-profiled and raised to a level of 3.8mODM, then re-vegetated. There are a number of large trees growing out of the bank and consideration should be given to their removal.

Further upstream, is an existing masonry pump house and perimeter wall, which are at the crest of the existing masonry wall. Neither the wall nor the pump house can be removed, as they are privately owned structures. Instead the sheet piles will be driven as close to these structures as possible. At the crest a new cap and reinforced concrete/masonry faced wall would be provided across the face of the buildings. This will protect the masonry buildings and seal the location in respect of flood defence as it is not known how water tight the buildings and area might be. An outfall exists in this section and this would need to be maintained through the sheet pile wall.

The section of the river directly downstream of London Bridge consists of a high earth bank, approximately 4mODM at the crest. This level will be sufficient to reduce the risk of flood inundation and it should be maintained at it current level. This embankment is at the crest of the existing poor condition masonry wall. The sheet piles will be driven as close to the wall as possible and a concrete cap will be placed at the crest. This section will also tie into the London Bridge abutment, sealing the new flood defences.

The capital cost (Including VAT) for this option has been estimated at €5,000,000.

Details of the works required for Location 2, Option 1 are presented in the drawings 9M2793 – 1010 and 1011

Option 2 - New Sheet Pile Wall with Concrete and Masonry wall at crest

This option is similar to the quay wall design recently completed on the right bank and is a slight variation from Option 1. This option requires the construction of a sheet pile wall with a concrete cap, as with Option 1. However, instead of providing intermittent lengths embankment and wall as the flood defence above the sheet pile wall, it considers the use of a continuous reinforced concrete/masonry clad wall on top of the pile cap.

The crest of the new wall will be at a level of +3.8mODM with the ground levels behind, in many cases, earth embankments and grass banks, will be re-profiled to a horizontal profile. The sheet piles will provide support to the toe of the existing river bank and the foundation for the new flood wall. The only location where this form of defence would vary is between the masonry pump house and London ridge, where the earth bank is sufficiently high and in a reasonable condition. Here it is proposed to maintain this bank and use similar profile to option 1, with toe piles to support that bank.

As with option 1 the services along this stretch of the river should be investigated prior to construction.

The capital cost (Including VAT) for this option would be €4,970,000.

Details of the works required for Location 2, Option 2 are presented in the drawings 9M2793 – 1012 and 1013

Option 3.1 – New Reinforced Concrete Wall

For this option, rather than using the sheet pile wall of previous options, it is proposed to construct a new reinforced concrete toe wall, with a crest level similar to the existing masonry wall, approximately 2.5mODM. Again, as with Option 1 the ground levels behind the wall, in many cases earth embankments and grass banks, will be raised to a level of 3.8mODM or maintained at their existing level if sufficiently high. Again the services along this stretch of the river should be investigated prior to construction.

For this option an extensive amount of excavation would be required both into the existing banks and river bed to construct the base detail and a sheet pile cofferdam would be required to allow the work to take place. It is not thought that the cofferdam would be required over the full length of the river but shorter lengths could be installed and moved as work progresses. In addition to this it is expected that a jack-up rig or similar would be required to facilitate the construction process and provide a working area during construction.

The flood defence option proposed in combination with this retaining wall, are similar to those proposed for option 1 and include intermittent walls and earth banks. At the masonry pump house and perimeter wall extensive excavation will not be possible to provide the required toe detail. Therefore over this length a slight variation would be required and it is proposed that a vertical concrete wall will be place close to the buildings, and tied either end into the full reinforced concrete wall.

The capital cost (Including VAT) for this option has been estimated at €5,050,000.

Obviously the suitability of this option would depend on the ground conditions and a detailed site investigation would need to be undertaken before this option could be fully recommended.

Option 3.2 – New Pre Cast Reinforced Concrete Wall

Similar to Option 3.1, this reinforced concrete wall will again require extensive excavation to enable it to be constructed. However, whereas with Option 3.1 the wall will be constructed in situ, this option can be pre cast away from site, then delivered and constructed at site.

The wall takes the form of ‘post and rail’ construction and sits on a cast in-situ reinforced concrete base. From this base 0.5m by 0.5m posts are inserted at 3.5m spacings, and set in place. Between the posts a series of ‘reinforced concrete rails’ are slotted into place, down vertical grooves in the posts. Connected to the back edge of the posts is an angled reinforced concrete flange, which provides lateral support to the front face of the posts and rails.

The wall will have a crest level of 3mODM, to enable it to support the existing quay at the toe, and rely on vegetated earth banks and short masonry walls to bring the standard of flood defence up to 3.8mODM or higher in the case of some existing defences.

As with option 3.1, this option is subject to confirmation of ground conditions through the completion of a detailed site investigation. It would also be necessary to construct a cofferdam as well as have the use of a jack-up rig on site.

The capital cost (Including VAT) for this option has been estimated at €5,700,000.

Details of the works required for Location 2, Option 3 are presented in the drawings 9M2793 – 1014 to 1016.

Option 4.1 – New Masonry Gravity Wall

This option proposes to makes use of materials similar to those currently in place and to main the appearance of the river bank as a masonry structure. Th option would require the use of a cofferdam and jack-up rig. The masonry wall will be broken out and any suitable material kept for reuse in the new wall. The riverbed will then be excavated at the toe down to suitable foundation level and a concrete foundation slab constructed to build the new gravity masonry wall off.

The wall will then be rebuilt using the good quality stockpiled stones and new masonry stones. The resultant cross sectional area will be prismoidal in shape and for the most part have a crest level of 2.5mODM. Behind the new masonry wall the defence level would take the form of an earth bank with a crest level of 3.8mODM where it can be fitted in. However, in two locations, namely adjacent to the existing boat storage and the masonry pump house and perimeter wall, earth works will not be suitable. Instead at the boat storage yard the existing masonry perimeter will be removed, and the new masonry wall will be extended to a crest level of 3.8mODM.

At the pump house and perimeter wall the new masonry wall will be built to a crest level of 2.5mODM. The existing buildings will be waterproofed, to reduce the risk of flood water leaking through the old masonry structure. The vegetated earth banks either side will be tied into the structure.

The capital cost (Including VAT) for this option would be €7,350,000.

Option 4.2 – New Masonry Block Wall

As a variation to Option 4.1, this option looks at using a new masonry block wall in place of the existing masonry wall. As with Options 3.1, 3.2 and 4.1 there will be a need for considerable excavation along the various different lengths of the river bank and the use of a cofferdam and jack-up rig.

This structure will be a blockwork wall at a 1 in 10 slope back, founded on a concrete base. The crest of the wall will be at 3mODM with the vegetated earth banks at the crest completing the flood barrier to 3.8mODM. Again the two areas where the cross sectional area will need to change will be at the existing boat storage and the masonry pump house and perimeter wall, where no earth works are possible. Instead as with Option 4.1 the crest of the wall will be raised to 3mODM by the boat storage area, and a low masonry wall will be provided over the pump house and perimeter wall which will also be waterproofed.

The capital cost (Including VAT) for this option would be €7,500,000.

Details of the works required for Location 2, Option 4 are presented in the drawings 9M2793 – 1017 to 1019

Location 3 – London Bridge to New Bridge (Chainage 761 to 1150)

Between the London Bridge and the New Bridge are two contrasting riverbanks. Looking downstream the left bank is a vegetated bank, with a footpath behind. The vegetated bank has a crest level of between 3.6 and 4.1mODM, with the lowest crest levels being located just upstream of London Bridge. Behind the footpath and the bank is a secondary perimeter wall over much of the frontage. The wall is higher surrounding one side of Lansdowne Road Stadium. The design tidal 200 year water level for 2031 has been estimated at 3.35mODM. Therefore it is considered that the existing flood defences on the left bank are sufficient to protect against a 1 in 200 year flood event to 2031. At most some minor raising of the footpath just upstream of London Bridge to +3.8mODM could be undertaken to increase the freeboard slightly.

Looking downstream on the right bank the level of the flood defences are considerably lower than those on the left bank. The primary flood defence is a concrete and masonry quay wall with a pedestrian walkway at the crest. Over part of the length a low brick wall exists which is around 0.5m high. The lowest defence level along this bank is around 2.9mODM, some 0.45m lower than the design tidal event.

Adjacent to New Bridge is a public open area, which is bounded by two walls, the primary wall closest to the river is a concrete wall, with several large gaps cut into it. The secondary wall behind is a masonry construction with a crest height of +4mODM, with 4 pedestrian gaps cut through it to lead into a small park area between the two walls. During the February 2002 flood event the area around New Bridge, on the right bank was flooded considerably.

Three options have been considered for this length of the river, on the right bank. The first was to raise the existing secondary brick wall, continue it downstream until London Bridge and improve the condition of the existing masonry river wall. The second was to construct a new primary crest wall at the front of the existing quay wall and improves its condition. The third was to provide a new sheet pile quay with masonry clad concrete flood similar to option 2 for location 2.

As stated earlier the design 200 year tidally dominated water level in 2031 is 3.35mODM. Applying a suitable freeboard to this might result in a defence level of around 3.6 to 3.7mODM. However, it would not be prudent to lower the upstream defence levels relative to the existing new defence level along the right bank between London Bridge and Ringsend Bridge and as such it is recommended that the defences are maintained over this length as a level of 3.8mODM.

In addition to providing a new defence on the top of the existing quay, the condition of the quay is questionable and much vegetation and voids have been noted. It is therefore considered that as part of the overall option and to ensure the overall structure’s integrity it is necessary to undertake major refurbishment works to the existing wall. This could take the form of cleaning out existing vegetation, repointing of the masonry blocks and pressure grouting of the whole structure to improve its structural integrity and effectively turn it into a mass structure.

Access along the right bank in order to be able to undertake any of the options outlined below is going to be a major consideration. London Bridge has a number of central piers and a very low soffit level and so the use of a jack-up rig would not be possible unless one could be brought to site in pieces and erected on site. Even with access from the river side, there will also be considerable issues along the tennis club frontage as it is very likely that access onto their property will be required which might even result in potential disruption or damage to some of the courts. Further upstream the landscaped area fronting the new estate at New Bridge Ave would need to be completely removed to facilitate the construction works, however this area could easily be re-landscaped. Nevertheless the costs implications of these access and logistic issues including compensation are likely to be extensive and appropriate sums have been included within the estimated capital costs.

Option 1 – Raise or Construct a New Secondary Back Wall

At present there is a promenade walkway which runs between London Bridge and New Bridge. Towards New Bridge, at the back edge of the walkway is a low masonry wall with a metal grate fence. This is at a level of around 3.4mODM, and fronts a housing development. Unfortunately this arrangement is not continued at the perimeter of the tennis club, adjacent London Bridge. At present there is corrugated perimeter fence, which is not tied to the concrete promenade walkway. This presents a flood risk to the tennis court behind and the locations adjacent. In addition at the New Bridge end there existing a pedestrian access through the masonry wall.

This option considers the construction of a wall along this back edge of the walkway, along the same line as the existing masonry wall.

Therefore the fence adjacent to the tennis courts will be removed and a new brick wall with fence panels will be put in place, with the lowest crest level at any point being 3.8mODM. Likewise the existing brick wall further upstream will be partially demolished and raised so that the lowest crest level will be 3.8mODM. This will effectively make a continuous brick and fence secondary wall along the back of the entire walkway between London Bridge and New Bridge.

As the proposed option considers the use of a wall along the back edge of the existing footpath issues arise at either end in respect of providing a uniform defence will maintaining access at the same time. Flood gate option were considered and could be made to work, but it was considered better to minimise the risk as much as possible by providing a more permanent solution and ensuring all permanent defence levels were kept to a level not below the design water level. To achieve this the option to use ramped walkways at either end have been considered.

At London Bridge end an existing masonry wall extend from the bridge a short distance upstream. This wall would need to be strengthened and realigned slightly and a new wall and safety railing constructed in the small carpark adjacent to the entrance to the tennis club. A ramp would be constructed between these walls leading up to a top platform with a crest level of 3.4mODM. The pedestrian would need to turn through 90 degrees and then again to walk down and second ramp leading onto the existing walkway. The realigned wall on the downstream side of the ramp would be in line with the new secondary wall upstream, such that dam boards could be installed along grooves in the wall ends during an event in order to bring the freeboard up to the adjacent defence levels. This way the ramp reduces the risk as much as possible and the dam boards provide extra security. The alignment of the dam boards along the line of the wall is such to prevent undue forces on the boards from flowing water.

At the other end it is proposed that a new ramp be built at the location of the existing pedestrian access and that the access is rebuilt. The top of the new ramp and access platform would have a level of 3.4mODM and as with the London Bridge gap, provision would be made for the installation of dam boards between the new secondary wall at either side. In addition dam boards should be provided for across the crest of the ramp between the two walls either side of the footpath.

In addition to the construction of the wall it will be necessary to carry out substantial strengthening works, as mentioned earlier, to the existing quay wall in order to ensure the integrity of the overall structure in the long term.

The capital cost (Including VAT) for this option has been estimated at €2,365,000.

Option 2 – Raise or Construct a New Primary Front Wall

This option concentrates on creating a primary flood defence on the crest of the existing quay wall. The wall will be a masonry faced reinforced concrete construction, with a crest level of 3.8mODM. This will mean that no work will be undertaken on any of the secondary defences, and that no access issues will need to be addressed. The wall will tie between the London Bridge wall and the existing masonry wall at the public open area, just downstream of New Bridge. In addition to the construction of the wall it will be necessary to carry out substantial strengthening works to the existing quay wall in order to ensure the integrity of the overall structure in the long term.

The capital cost (Including VAT) for this option has been estimated at €2,220,000.

Option 3 - New Sheet Pile Wall with Concrete and Masonry wall at crest

This option is similar to the quay wall design recently completed on the right bank between Ringsend Bridge and London Bridge and also that proposed in option 2 for the left bank over that same reach of the river. This option requires the construction of a sheet pile wall with a concrete cap and a continuous reinforced concrete/masonry clad wall on top of the pile cap. The option has been included here in the event that following a detailed structural investigation of the existing wall it is considered that the wall is not suitable for founding a new flood defence in the long term.

The crest of the new wall will be at a level of +3.8mODM. The sheet piles will provide support to the existing river bank and the foundation for the new flood wall.

Services along this stretch of the river should be investigated prior to construction.

The capital cost (Including VAT) for this option would be €2,965,000.

No drawings for this option have been produced however the details would be similar to those shown on drawing number 9M2793 – 1013

Public Open Area Options

This area just downstream of New Bridge is an area that is bounded by two walls. The primary defence consists of a concrete river wall with crest levels between 2.5m and 3.5mODM. The secondary defence consists of a masonry wall with a crest level of around +4mODM. Between these two walls exists a vegetated area known locally as the “Jungle”. The concrete river wall has two long gaps and a gap with steps down to the waterline. The masonry perimeter wall has four pedestrian gaps, two directly adjacent to New Bridge, and a further two downstream. This option will focus on providing flood defence through the use of the masonry secondary wall thus maintaining the "Jungle" as a flood plain. It is intended to close the two narrowest gaps, one located near New Bridge and the other at the most downstream end. At the two remaining gaps localised flood protection measures will be provided by locally raising ground levels and using damboards. The costs for this are include in the two options above.

Details of the works required for Location 3, Option 2 are presented in the drawings 9M2793 – 1020 to 1022.

Location 4 – New Bridge to Railway Bridge (Chainage 554 to 751)

The right bank consists of a masonry block quay with a concrete walkway behind a crest wall. Behind that is a steel perimeter fence, which bounds the school grounds adjacent to the river. The crest part of the quay wall continues upstream for approximately 60m, and is then replaced by a steel fence and vegetated bank. The level of the walkway is around 3mODM, and the crest wall is between 3.6 and 3.8mODM.

The left bank is a heavily vegetated bank, which has a masonry wall adjacent to New Bridge and masonry revetment at the toe as you move upstream to the Railway Bridge. The approximate ground levels behind the wall and revetment are around 3mODM, although access to the site is difficult and these levels should be confirmed. The crest behind both the revetment and wall is heavily vegetated. There are a number of residential properties that back directly onto the riverbank, behind the wall/revetment and the vegetation. These residential properties could potentially be at risk during a tidal flood event, due to the low ground levels and indeed a potential flood path through them and across the road adjacent to the Landsdown Road stadium exists.

The extreme 200 year tidal flood risk level along this section of the river has been estimated at 3.4mODM. Therefore some raising of the defences will be required.

Left Bank Option

The existing masonry wall and revetment adjoining it are both in a poor condition, and will require some work as part of the options that are being considered along this section of the River Dodder. This will include the removal of the vegetation from and cleaning of the masonry structures. In the case of the wall, it will be repointed and grouting to ensure its integrity as part of the new flood defences. The crest level of the wall will remain unchanged.

At present there is a masonry revetment, which is in poor condition running between the existing masonry wall and the Railway Bridge. This option proposes removing this existing revetment and adding to and regrading the ground beneath its footprint, so that a new formal revetment can be installed with a 1 in 2 slope. The reveted slope will be brought up to a crest level of 3mODM. This new revetment will be constructed from both stockpiled material from the existing structure and new material. At present an estimate that approximately 60% of the existing masonry revetment material can be reused has been made. However, it is recommended that this be confirmed by carrying out a detailed site inspection prior to final design and construction.

At the crest of the new revetment and behind the existing wall, the slope will be continued up at 1 in 2 to form a new vegetated earth bank. The crest of this bank should be built to a level of 3.9mODM.

Right Bank Option

The wall, adjacent to New Bridge, which runs along the right bank has a considerable amount of vegetation growth that will need to be removed. As a result of this repointing and some grouting may be required. In addition, due to the poor condition in some places and also the possibility of scour, there is requirement to underpin the toe. For this underpinning work two methods have been considered. The first would use a new concrete buttress to the toe, and the second would use treated timber posts driven infront of the toe with a concrete infill. Further upstream of this point, the river bank consists of a natural earth bank with some stone pitching along the toe. On the crest of the bank is a footpath and behind this the grounds of a school. The levels along the footpath, while reasonably high, are not sufficiently so as to provide a safe margin of defence into the future.

The main flood protection option consideration along this length therefore is the construction of a raised earth flood mound placed behind the footpath and within the school grounds. The existing ground levels in this area are approximately 3mODM on both the concrete walkway with the levels into the school ground being slightly higher. The crest of this new flood mound will be 3.9mODM, and will be grass seeded and landscaped as part of the final finish.

Putting the flood mound in this location means that the access from New Bridge to the walkway will potentially become a floodpath that tidal floodwaters can flow through. To stop this a swing shut floodgate and walls will be installed adjacent to New Bridge, with the flood mound tying perpendicular into the landward floodgate post and wall. Under normal conditions the floodgate will remain open maintaining a 2m wide access gap along the footpath.

At the upstream end, just downstream from the Railway Bridge there is a two metre wide pedestrian access, which runs beneath the bridge. This access still needs to be maintained to allow pedestrian access along the existing walkway. For this to be achieved a new floodgate will be installed on the downstream side of the Railway Bridge, which will need to be closed prior to a extreme tidal flood event

The total capital cost (Including VAT) for the works considered along both banks of the river within this stretch is estimated at €3,195,000.

Details of the works required for Location 4, options are presented in the drawings 9M2793 – 1030

Location 5 – Railway Bridge to Ballsbridge (Chainage 78 to 537)

From the flood risk analysis it has been established that this section of river upstream of the Railway Bridge could be susceptible to a 1 in 200 year extreme tidal event of 3.45mODM.

The flood defences on the right bank of the river consists of a masonry and concrete crest wall at the top of a masonry and concrete quay wall. The wall itself has a crest level of between 3.5m and 4mODM and the footpath levels to the rear have levels varying from 2.9 to 3.5mODM. This wall has been sporadically repaired and is a combination of old and new construction. There are also a number of drainage holes through it to allow water from the footpath to flow back into the river.

The left bank is a mixture of high masonry walls near the Balls Bridge end, which are part of buildings that back onto the River Dodder. Further downstream there is a low vegetated bank, which is backed by a steel security fence and business units and residential properties.

Left Bank Option

Just downstream of Ballsbridge, on the left bank, there are a number of buildings whose perimeter walls back directly onto the quay side. These walls are all +6mODM, in good condition, but have extensive vegetation growth. This vegetation will need to be removed, and the walls cleaned for possible pointing of the walls.

Further downstream, the buildings do not back directly onto the river bank, instead they are set further back and there is a security fence on the quayside, fronted by a low vegetated bank. The crest level along this section is approximately 3 to 3.25mODM, which is some lower than required to protect from an extreme tidal flood event. Therefore a low reinforced concrete wall, with masonry cladding will be constructed on the quay side, approximately half a metre back, at the crest of the vegetated bank.

This new wall will be in place of the existing security fence and will have a crest level of 3.95mODM, and will have a new security fence fixed at the crest. The location of this option is such that it should not inhibit the continual use of both the river bank and the area behind which is used as a residential and business unit area.

Right Bank Option

The existing crest level of the wall is between 3.5m and 4mODM, so effectively would be sufficient to withstand a 1 in 200 year tidal flood event of 3.45mODM. The wall may need to be locally raised just upstream of the Railway Bridge as the crest level is approximately 3.5mODM, which does not give sufficient freeboard. In addition to this the drainage holes through the wall from the footpath will need to be blocked up or preferably flapped.

Otherwise the main issue is vegetation growth along the quay walls. There is a need as part of this option to have an extensive clearance of this vegetation and repointing of this quay wall. In some cases some grouting may also need to be undertaken to fill any voids that may have occurred as a result of the vegetation growth.

Inspection of the wall shows that the toe is showing signs of undermining as a result of river flow. This is most prevalent on the outer side of the meander, where scour is a major issue. About mid way along this section just downstream a bend on the inner side there is a large area of vegetated accretion, which is currently protecting the toe. However at the other locations along the right bank erosion of the toes is noted.

In these latter sections two options to underpin the toe have been considered. The first would use a new concrete buttress to the toe, and the second would use treated timber posts driven infront of the toe with a concrete infill.

The final part of this option considers the use of a swing shut floodgate at the upstream end of the walkway. In part this will be to stop pedestrians using this walkway during extreme tidal flood events and in part to maintain an acceptable level of freeboard from the footpath back along the river.

The capital cost (Including VAT) for this option would be €3,990,000.

Details of the works required for Location 5 options are presented in the drawings 9M2793 – 1040.

17.4.8 River Tolka Options

The options for this area extend upstream from the John McCormack Bridge to the Annesley Bridge (Chainage 43 to 485).

The options along this section of the river focus on refurbishing the existing quay walls and clearing vegetation. There will also be some local raising of the flood defences just upstream of the Railway Bridge on the right bank. The main issues addressed by these options are:

- Reduce flood water egress out onto the road behind the existing quay walls.
- To improve the overall condition of the river walls
- To provide an improved public health and safety around the footbridge.

Each location and its options will be described in more detail below.

Flood alleviation options along the upstream sections of the River Tolka have been undertaken as part of the River Tolka Flood Alleviation Study by M C O'Sullivan. The DCFPP is concerned with alleviating the risk of coastal flooding that could occur within the River Tolka. The project boundary was limited to the Annesley Bridge, approximately 1.5km from the mouth of the river.

The river discharges into Dublin Bay through a series of culverts beneath the East Point Business Park Bridge. Moving upstream, between the East Point Business Park Bridge and John McCormack Bridge are currently the Dublin Port Tunnel construction works. Along this length of the river no options are considered due to its unfinished nature.

The right bank (looking downstream) of the section between John McCormack and Annesley Bridges comprises of a masonry and stonework quay wall. This wall is in a poor condition with extensive vegetation across much of its face. As part of the option for this bank, the vegetation will be cleared and considerable re-pointing and pressure grouting will be undertaken to fill voids and improve the overall integrity of the wall.

As part of the refurbishment of this wall, a low area along the walls crest will be renovated and raised to bring its crest level up to that of the adjacent crest levels of 3.9mODM. The existing crest height is at 3.2mODNM, and does not provide sufficient freeboard against potential flood events.

On the left bank the quay wall immediately downstream of Annesley Bridge, comprises of a concrete wall, which has had some remedial work undertaken on it. Further downstream the bank converts to a sheet pile wall with a concrete cap beneath both the footbridge and overhead railway bridge. Between the railway bridge and John McCormack Bridge is a concrete curved toe wall

The crest level of these quays varies between 3.5m and 4mODM. The option over this bank consists of extensive clearance of the vegetation along the concrete sections of the quays, with minor re-pointing and repair work as necessary. The crest levels are sufficient to cope with the potential tidal flooding that could occur, which has been estimated at approximately 3.3mODM, in 2031.

One area of concern along these quays is located adjacent to the steel footbridge on the left bank. Around this location the park ground levels are approximately the same level as the crest of the quays. Potentially this could be a health and safety hazard for pedestrians using the park and crossing the footbridge. As part of the general clean up and improvement of the left quay a new safety rail could be installed, and linked in, on either side of the footbridge. It will be continued downstream to under the railway bridge, and upstream until the wall crest levels are at least 0.5m above the park ground levels.

The capital cost (Including VAT) for this option has been estimated at €1,645,000.

There are no drawings for this section.

17.5 Summary of Options

In this section a summary of the options listed above is presented. The costs outlined above are estimates of the capital expenditure for each option including VAT. However, other significant costs are likely to be required prior to reaching the construction stage. These costs might include such things as,

- Site investigation 5%
- Planning, legal, and compensation issues 4%
- Engineering Fees 5%
- Site Supervision. 3%

At this stage in order to give a better overview of the total likely project costs to completion it is considered prudent to allow a percentage of the capital costs to cover these costs. The percentages above have been allowed at this outline stage and the summary table, Table 17.1, presented in this section includes an appropriate allowance based on these percentages to give a total likely cost per scheme.

Furthermore, at some locations a number of different options are available with varying costs. Firm recommendations as to which option should be carried forward at this stage has not been made, however the summary tables 18.1 and 19.1 provide a comprehensive summary of such things as costs, benefit cost ratios, environment issues, and priority in respect of location. In the summary table, Table 17.2, three total cost estimates have been carried forward and used to provide the benefit cost ratios presented in Table 19.1. These include the highest, medium and lowest cost of all the options developed for each location. From Table 17.2 it can be seen therefore that the total capital expenditure including site investigation and other issues lies within the range of €62,000,000 to €93,000,000.

18 ENVIRONMENTAL REVIEW

18.1 Introduction

This section presents a review of the potential environmental issues and impacts that may arise as a result of proposed flood defence options (or lack thereof) for the various flood areas within the study area.
This section contains 7 paragraphs, the first of which provides an introduction to the project and the results that are presented. Paragraph 2 briefly describes the range of defence options based on the Engineering Report. Paragraph 3 describes the methodology used for the review. Paragraph 4 presents a brief summary of the baseline environment for each flood "compartment". Paragraph 5 identifies the likely impact if no schemes are undertaken, whilst Paragraph 6 reviews and describes the potential issues and impacts associated with the various flood defence options. A summary and conclusions are presented in Paragraph 7.

18.2 Study Area

The extent of the project area is presented on Figure 1.1. The study area includes both the coastal boundaries and also river and canal boundaries over their tidal reach.

18.3 Proposed Options

The following provide a summary of the proposed options identified for the various flood compartments and areas.

18.3.1 Baldoyle Bay

Location 1 – North of Baldoyle Town Centre

Option 1 – Bermed earth bank.
Option 2 – Earth bank.
Option 3 – New sea wall.

Location 2 – South of Mayne River

Option 1 – Earth bank and new wall.
Option 2 – Earth bank and new wall replacing gabions.

Location 3 – North of Mayne River

Option 1 – Earth bank.

Location 4 – North Western End of Baldoyle Estuary

Option 1 – New earth banks, new flood water storage area and raise road levels.

Location 5 – Southern End of Portmarnock

Option 1 – New short walls and raise ground levels.
Option 2 – New earth banks.

18.3.2 Baldoyle Town

Option A – Dutch dam in sea wall gaps.
Option B – Sliding flood gates in sea wall gaps.

18.3.3 North Howth

No specific works are specified or currently required. However, options may need to be examined to address the front face erosion of the dune system in the longer term, including:

- Offshore breakwaters.
- Fishtail groynes.
- Beach renourishment.
- Sand dune management system.

18.3.4 Howth South West

For the complete frontage any of options 1, 2 or 3 can be combined with options 4 or 5 to provide a composite and uniform defence and reduce the risk of flooding considerably from less than a 1 in 10 year event to an event of greater magnitude than a 1 in 200 year return period:

Option 1 – New rock revetment and crest wall.
Option 2 – New rock revetment and flood wall landward of existing footpath.
Option 3 – New rock revetment with flood wall along Greenfield Road.
Option 4 – New concrete seawall and re-pointing of existing wall.
Option 5 – Repaired/new seawall with set back flood wall.

18.3.5 Clontarf

Option 1 – Raising the existing flood defences.
Option 2 – New set back re-curve seawall with new promenade.
Option 3 – Replacement of the secondary wall.
Option 4 – Offshore breakwater.

18.3.6 East Link Toll Bridge

Option 1 – Combined traffic barrier and flood defence.
Option 2 – New flood wall.

18.3.7 ESB Poolbeg Power Station Options

The options for this stretch use overlapping earth mounds, positioned on the seaward side of the road, at the crest of the revetment. These overlapping earth mounds are positioned across the frontage of any major access gaps that exist in the earth mound and which cannot be closed. The existing road level is above the still water level and these mounds are designed to deal with any wave action that could otherwise overtop onto the road. The mounds would have a crest level of 4.5mODM. On the seaward face of the raised earth mound a new rock revetment should be constructed. This will protect the new and existing earth mounds and provide a more stable flood protection barrier.

The options for closing any minor gaps in the earth mound, bounding the ESB Poolbeg Power Station, will be locally filled and like the overlapping gaps will be vegetated to blend in with the surrounding area.

18.3.8 Sandymount Strand, North of Existing Promenade

Option 1 – Raise existing masonry and stonework wall.
Option 2 – Raise existing wall to 4.2mODM and build new rock revetment at toe.
Option 3 – Extend promenade north to Sean Moore Park and raise existing wall to 4.2mODM.

Access Gap Sean Moore Park
Four access options.

18.3.9 Sandymount Strand, Existing Promenade Options

Pedestrian gaps 1, 2, 6, 11 and 14.
Pedestrian gap 5.
Pedestrian gap 7.
Vehicular gaps 3 and 4, vehicular gaps 12 and 13 options.
Vehicular gap 8 options.
Pedestrian Gaps 9 and 10 options.

18.3.10 Merrion Gates

Option 1 – Flood wall & revetment
Option 2 – Flood wall & rock berm

18.3.11 River Liffey

Upper River Liffey – Island Bridge Weir to Sean Heuston Bridge.
Mid River Liffey – Sean Heuston Bridge to Matt Talbot Memorial Bridge.
Lower River Liffey – Matt Talbot Memorial Bridge to East Link Toll Bridge.

Option 1 – Demountable flood defences.
Option 1.1 – Board barrier.
Option 1.2 - Mobile dam.
Option 1.3 - Dutch dam.
Option 2.1 – New concrete floodwall.
Option 2.2 – New masonry floodwall.
Option 3 – Glass flood barrier.

18.3.12 River Dodder

Location 1 – River Liffey Confluence to Ringsend Bridge

No options identified.

Location 2 – Ringsend Bridge to London Bridge

Option 1 – Low new sheet pile wall with concrete cap and flood barriers behind.
Option 2 - New sheet pile wall with concrete and masonry wall at crest.
Option 3.1 – New reinforced concrete wall.
Option 3.2 – New pre cast reinforced concrete wall.
Option 4.1 – New masonry gravity wall.
Option 4.2 – New masonry block wall.

Location 3 – London Bridge to New Bridge

Option 1 – Raise or construct a new secondary back wall.
Option 2 – Raise or construct a new primary front wall.
Public open area options.

Location 4 – New Bridge to Railway Bridge

Left bank option.
Right bank option.

Location 5 – Railway Bridge to Ballsbridge

Left bank option.
Right bank option.

18.3.13 River Tolka

The options for this area extend upstream from the John McCormack Bridge to the Annesley Bridge. The option along this section of the river focuses on refurbishing the existing quay walls and clearing vegetation. There will also be some local raising of the flood defences just upstream of the Railway Bridge on the right bank.

18.4 Methodology

18.4.1 Introduction

This section describes the method used for review of the potential impacts for the various schemes. This methodology does not extend into the detail of determining the impacts in a quantifiable manner, only identifying potential positive or negative effects, and potential issues.

The potential environmental impacts associated with both the options are identified through:

- Observations on site;
- A review of the existing data;
- Knowledge of and impact matrices related to other flood protection schemes.

An impact is determined based on the existing baseline environment and the alteration of any physical, chemical, biological or perceived characteristics of that environment.

18.4.2 Impact Appraisal

Where possible, beneficial and adverse impacts have been appraised based on their potential scale/magnitude, longevity and significance. Where potential adverse impacts were identified, possible mitigation measures have been defined. Where impacts were identified as irreversible these have been differentiated.

A subjective scale has been used to classify the potential significance of the impacts, using a seven point scale (from major adverse to major benefit), as shown on Table 18.1. The magnitude of each proposed impact is compared with the importance of the individual assets. The magnitude of impact is characterised as high, medium or low for both adverse and beneficial impacts, and its determination is based on the description below. The value of the features to proposed impacts is characterised on a five-point scale from international to low site-specific.

Table 18.1 - Derivation of Significance Criteria from Magnitude/Value Comparisons

Magnitude	Value of Feature				
	International/ National	Regional/ County	District	Local	Site-Specific
High	Major	Major	Major	Moderate	Minor
Medium	Major	Moderate	Moderate	Minor	Minor
Low	Moderate	Minor	Minor	Minor	Minor

Magnitude

The magnitude of the effect is the degree of change that it causes or is considered to cause compared to the baseline. In order to determine the degree (or magnitude) of change created by a certain effect, compared to baseline conditions, an indication of the existing baseline level and its variations (temporal and spatial) are determined. In addition, information relating to other anthropogenic effects that could occur on the resource in question from a necessary part of the determination of magnitude. However, it is the overall sensitivity of the feature to change, and how the feature is changed that predominantly factors in the identification of the magnitude of the affect.
The sensitivity of a feature relates to the level of intolerance of the receptor to the effect being considered, or the degree to which the specific aspects that give the feature its value are altered. Table 18.2 provides a description of the 3 levels of quantification of magnitude with a general description of the meaning of each ‘level’ of magnitude as well as a description of its definition in terms of feature sensitivity.

Table 18.2 - Derivation of Magnitude of the Effect

Magnitude	Description	Sensitivity
High	A significant change.	In ecological terms, the species/population is likely to be killed/destroyed by the effect under consideration.
Medium	Change that is noticeable.	In ecological terms, some individuals of a species/population may be killed/destroyed by the effect under consideration and the viability of a species population will be affected.
Low	Change which may only just be noticeable.	In ecological terms, some individuals of a species/population may be killed/destroyed by the effect under consideration but the viability of a species population will not be affected.

18.4.3 Impact Characteristics

Based on the determined level of magnitude and the importance or value of the feature, the significance of the impact is then determined using the characterisation identified in Table 18.1. The basic definitions of significance (major, moderate and minor) are defined in Table 18.3.

Following the objective description of the impact, the impact can then be characterised in terms of its nature and magnitude or physical extent. The nature of predicted impacts would be described, as appropriate, using the following terms:

- Beneficial or adverse;
- Direct or indirect;
- Secondary;
- Short-, medium- or long-term;
- Permanent or temporary;
- Reversible or irreversible; and
- Cumulative.

Table 18.3 - Terminology for Classifying and Defining Impacts

Impact	Definition
Major beneficial	The impact provides a significant positive gain
Moderate beneficial	The impact provides some gain to the environment
Minor beneficial	The impact is of minor significance but has some environmental benefit
Negligible	The impact is not of concern
Minor adverse	The impact is undesirable but of limited concern
Moderate adverse	The impact gives rise to some concern but it is likely to be tolerable (depending on its scale and duration)
Major adverse	The impact gives rise to serious concern; it should be considered as unacceptable

In general terms, it will be assumed, unless otherwise stated, that impacts are:

- Short-term during the construction phase (i.e. 18 months);
- Long-term during the operational phase;
- Local rather than regional; and
- Potentially reversible rather than irreversible.

18.5 Baseline Environment

18.5.1 Introduction

This section provides a brief summary of the baseline environment for each "compartment" or "scheme area". Details used for this are based on a site visit, previous data collection, and other literature sources. Additional detailed data is presented in Appendix R1. Where available designated site synopses are presented in Appendix R2, however, due to an error on the Duchas website the NHA synopses cannot be obtained.

18.5.2 Baldoyle Estuary

Location 1 – North of Baldoyle Town

This area is located along the southern half of the Baldoyle Estuary, as shown in Figure 18.1. The shoreline is fronted by hard defences, behind which is a coast road linking Portmarnock to Baldoyle and Howth, backed by residential properties. There is a narrow promenade between two walls with a number of pedestrian access points to the beach. The inter-tidal beach is predominantly sand and gravel and there is no vegetation fronting the defences or on the defences themselves. As well as views across Balydoyle Bay, there are wide sea views of Ireland's Eye and Howth. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

Figure 18.1 - Location 1 Looking North and South Respectively

Location 2 – South of Mayne River

This location contains an area of high inter-tidal of Baldoyle Bay with a straight faced wall providing protection to the coast road immediately behind, as shown on Figure 18.2. A couple of properties are located behind the road, and the remaining land use is agricultural. There are no access points down onto the inter-tidal. The inter-tidal area is sandy and gravel. Saltmarsh vegetation and occasional trees and shrubs are located in the inter-tidal and immediate landward area. There are relatively extensive views across Baldoyle Bay and to Howth in the south. Views from the lower floor of properties are generally obstructed by coast protection walls. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

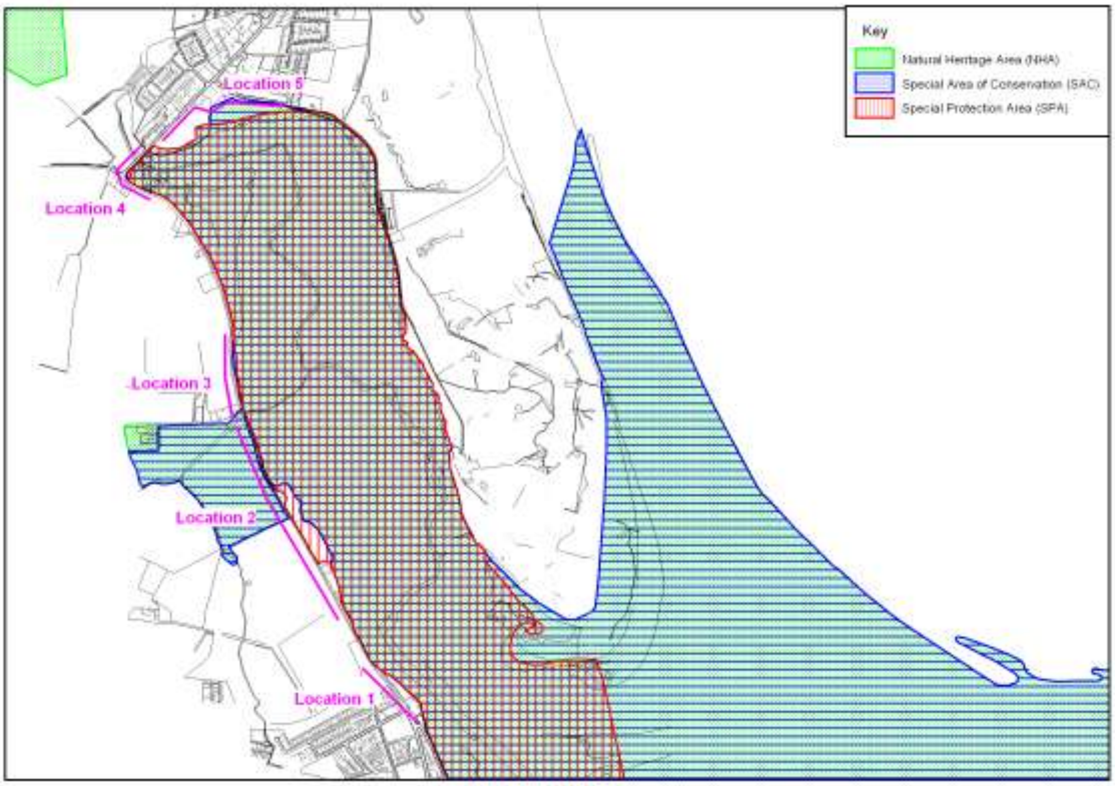


Figure 18.2 - Baldoyle Bay Area

Figure 18.3 - Location 2 from the Coast Road Looking Eastward

Location 3 – North of Mayne River

Similar in character to Location 2, this area also contains the outfall for the Mayne River, and is shown on Figure 18.2. This area supports a greater amount of saltmarsh vegetation as well as terrestrial vegetation. The Mayne River is a small watercourse that outfalls via a one way flap valve. The watercourse supports a low density and diversity of aquatic vegetation. There are relatively extensive views across Baldoyle Bay as well as Howth to the south. Views from the lower floor of properties are generally obstructed by coast protection walls, and also trees and shrubs. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

Location 4 – Portmarnock Roundabout

This area lies in the far north west corner of Baldoyle Bay as shown on Figure 18.2. There are large areas of saltmarsh in the upper inter-tidal, which front a straight-faced wall behind which is situated the roundabout leading the coast road into Portmarnock. The roundabout is backed by trees and agricultural land, with occasional residential properties. A small watercourse outfalls via a one way flap valve. Behind the road it also contains a small storage area. Views are generally constrained by trees to the north and west, and the shrubs behind the inter-tidal saltmarsh, with more open views east toward Portmarnock and the upper areas of Baldoyle Bay. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

Figure 18.4 - Location 4 Portmarnock Roundabout

Location 5 Southern Portmarnock

This area lies at the northern end of Baldoyle Bay as shown on Figure 18.2. It is fronted by a large area of saltmarsh covering the upper inter-tidal. This is backed by a large area of open land used for informal recreation, particularly by residents, with stands of trees and shrubs. Behind the open land is a wall behind which lies the coast road into Portmarnock and then residential properties. Residences are situated in front of the road. Views from the west look southward over the open land toward Baldoyle Bay, with more constrained views to the east due to established tree and shrub growth. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

Figure 18.5 - Location 5 Portmarnock Looking South East

Figure 18.6 - Location 5 Portmarnock Looking West Across Public Open Space

18.5.3 Baldoyle Town

This area is located at the south western edge of the Baldoyle Estuary, as shown on Figure 18.8. The shoreline is fronted by hard defences, behind which is a coast road linking Portmarnock to Baldoyle and Howth, backed by residential properties. There is a narrow promenade between two walls with a number of pedestrian access points to the beach as well as a narrow slipway. The inter-tidal beach is predominantly sand and gravel and there is no vegetation fronting the defences or on the defences themselves. As well as views across Baldoyle Bay and there are wide sea views of Ireland's Eye and Howth. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). Site synopses are present in Appendix R2.

Figure 18.7 - Looking South toward Howth from the North End of Baldoyle

18.5.4 North Howth

This northerly facing area contains a shallow sloping sandy beach backed by eroding sand dunes. The frontage and area is shown on Figure 18.8. This relatively thin strand of dunes vegetated by salt resistant plants is backed by a row of large residential properties each bounded by a wall. There are a small number of public access through the properties onto the beach, and access for most of the residences. These access points have resulted in eroded paths through the dunes. Views are generally north up the estuary with sea views to the north east. The area of Baldoyle Bay fronting this area is a designated Natural Heritage Area (NHA) and a Special Area of Conservation (SAC). Site synopses are present in Appendix R2.

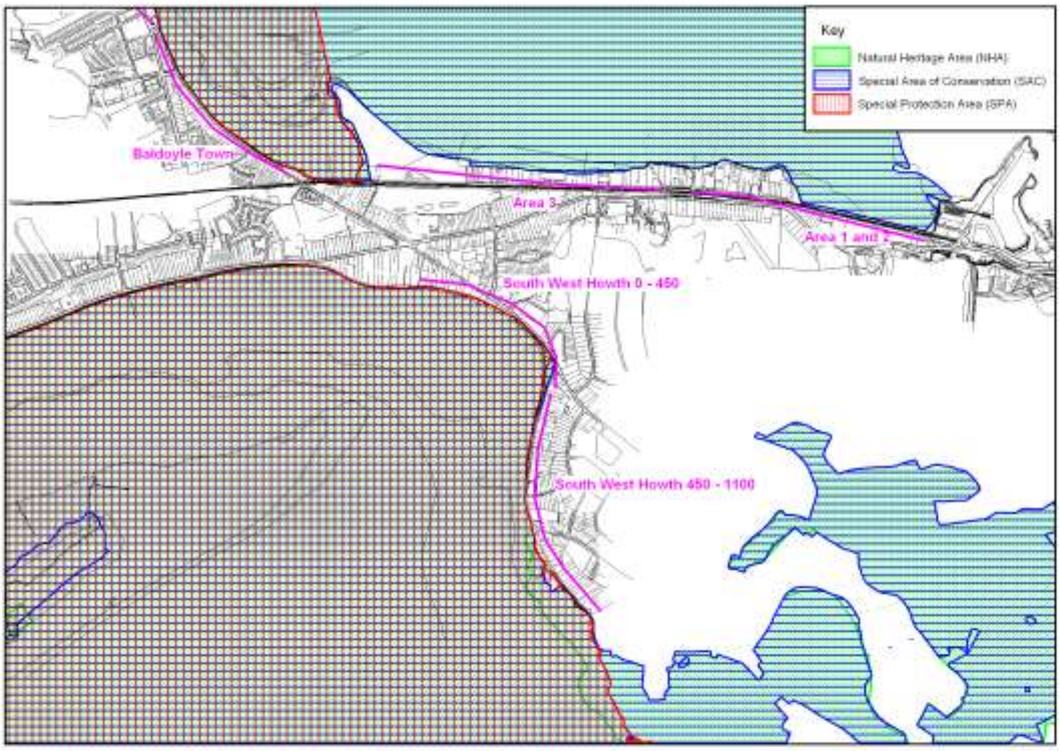


Figure 18.8 - Baldoyle Town and North and South Howth Area


Figure 18.9 - North Howth looking East Along Dune System (2004 and 2005)

18.5.5 Howth South West

This area of coastline faces to the south west into Dublin Bay as shown on Figure 18.8. The shoreline consists of a sand and gravel inter-tidal area, backed by a seawall with revetment along the northern edge. There is a promenade and parking area behind the seawall, behind which lies a road and residential properties behind this. The inter-tidal area supports growth of seaweed. Views are across Dublin Bay, encompassing Bull Island and Dublin in the background. The inter-tidal and sub-tidal habitat fronting this area is part of the Bull Island Natural Heritage Area (NHA), Special Area of Conservation (SAC) and Special Protection Area (SPA). Site synopses are present in Appendix R2.

Figure 18.10 - South West Howth Looking South East along the Shoreline

Figure 18.11 - South West Howth Looking East toward Figure 18.10 Location

18.5.6 Clontarf

The Clontarf frontage is dominated by the sandy and mudflats of the Tolka Estuary to the west and North Bull Island to the east. The shoreline is characterised by a promenade behind which is the main road between Howth and Dublin, and behind this is a predominantly residential land use, with occasional commercial premises. As well as the many informal recreational participants along this stretch of shoreline, there are also formal recreational activities that occur along or off this area, with the Clontarf Yacht and Boast Club being at the forefront. Tolka Estuary and Bull Island are both designated as Natural Heritage Area (NHA), Special Area of Conservation (SAC), and Special Protection Area (SPA). Site synopses are presented in Appendix R2. Views along the promenade and roadside properties are dominated by the port and commercial aspect of south Dublin, as well as views along Bull Island and out across Dublin Bay. The frontage is presented on Figure 18.13, which also shows the designated site boundaries.

Figure 18.12 - Clontarf Looking South toward Dublin Port

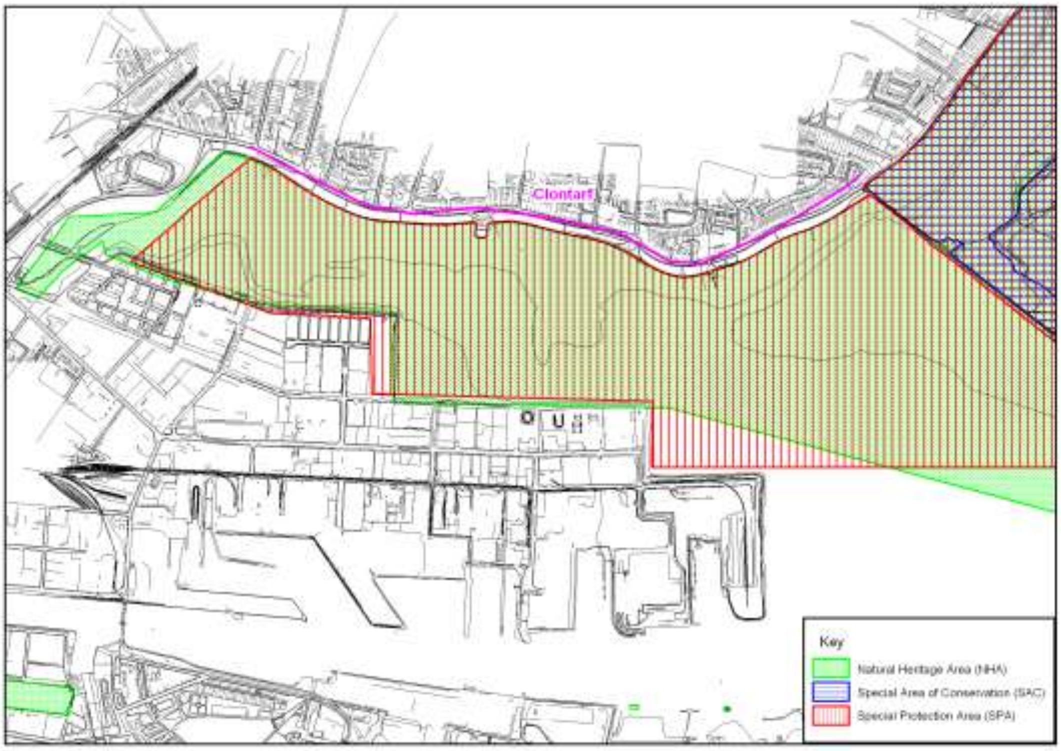


Figure 18.13 - Clontarf and Tolka Estuary

18.5.7 East Link Toll Bridge

The East Link Toll is situated on the south side of the River Liffey as shown on Figure 18.15. It is located immediately adjacent to the River Liffey, bounded by a quay wall.

The area of the East Link Toll is dominated by commercial or port related premises, forming little or no amenity for the residents of Dublin.

18.5.8 ESB Poolbeg Power Station

The Poolbeg Power Station is situated near the eastern end of Dublin Port, along the southern side of the River Liffey, as shown on Figure 18.15. The Power Station lies along the port frontage of the Liffey, but also backs onto Sandymount Strand, with a small road in between. The land use of the terrestrial area is commercial and industrial, but many residents participate in informal recreational facilities along this area of coastal access. Views are blocked to the north by the built up nature of the commercial activities, and also to the west by the city of Dublin. However, there are expansive and open views across the sea to the east, and Sandymount Strand to the south. The inter-tidal and sub-tidal area of Sandymount Strand is a designated as a Natural Heritage Area (NHA), Special Area of Conservation (SAC) and a Special Protection Area (SPA). A site synopsis of the SPA is presented in Appendix R2.

18.5.9 Sandymount Strand North of Existing Promenade

This area of shoreline consists of the inter-tidal sand flats of Sandymount Strand, backed by Beach Road and Strand Road, behind which is a large area of residential housing, as shown on Figure 18.16. At the northern end of this section is the open space of Sean Moore Park. Views from the residential properties fronting the shore extend out across Sandymount Strand and Dublin Bay, with some constraint by the port and commercial enterprises located along the Liffey and extending out into the Bay. Extensive informal recreation occurs along this area, utilising both the park and the beach. The whole of the inter-tidal area lies within the Sandymount Strand (South Dublin Bay) Natural Heritage Area (NHA), Special Area of Conservation (SAC) and Special Protection Area (SPA). A site synopsis of the SPA is presented in Appendix R2.

Figure 18.14 - Sandymount Strand Near Sean Moore Park and Beach Access looking toward ESB Poolbeg

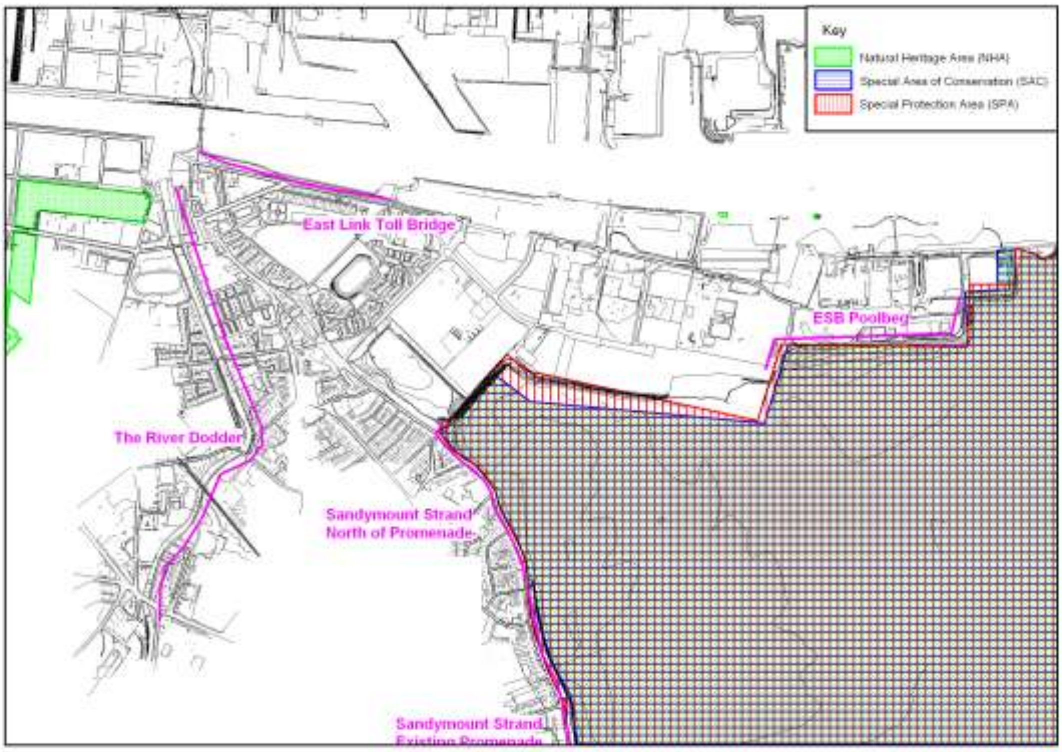


Figure 18.15 - East Side of Dublin (River Dodder and Sandymount Strand)

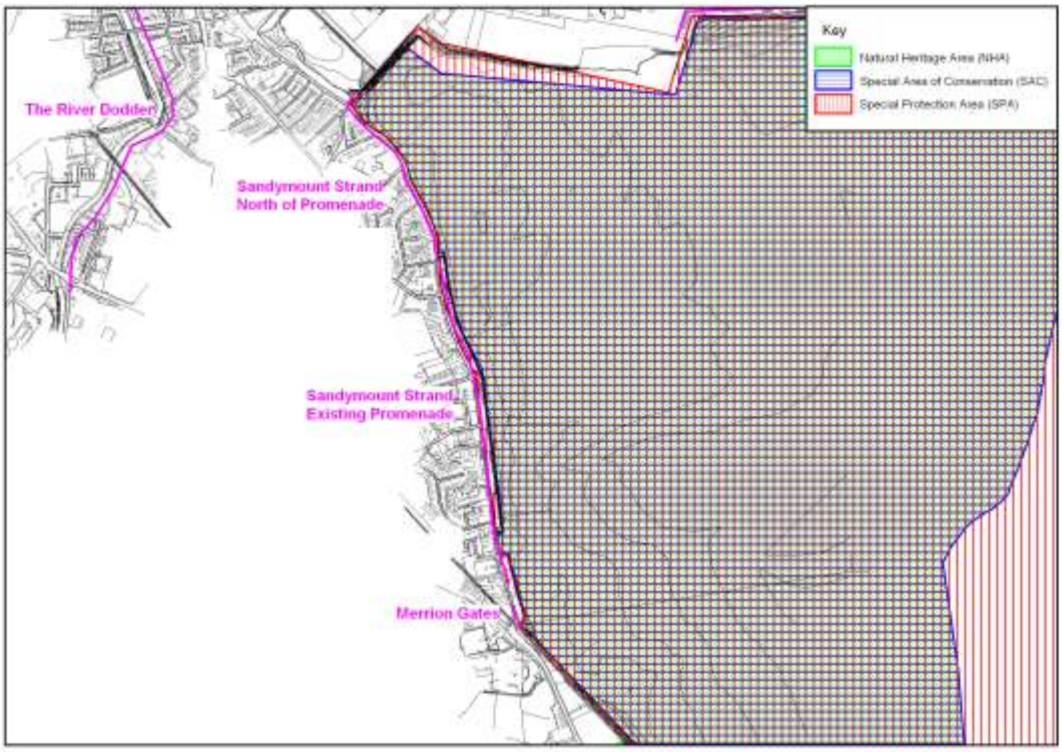


Figure 18.16 - Sandymount Strand Frontage

Figure 18.17 - Sandymount Strand Beach Access at Sean Moore Park looking South

18.5.10 Sandymount Strand Existing Promenade

This area of shoreline consists of the inter-tidal sand flats of Sandymount Strand, backed by a promenade with a number of parking areas, behind which is Beach Road, and then further residential housing, as shown on Figure 18.16. Views from the residential properties fronting the shore and from the promenade extend out across

Sandymount Strand and Dublin Bay. Extensive informal recreation occurs along this area, utilising both the promenade and the beach. The whole of the inter-tidal area lies within the Sandymount Strand (South Dublin Bay) Natural Heritage Area (NHA), Special Area of Conservation (SAC) and Special Protection Area (SPA). A site synopsis of the SPA is presented in Appendix R2.

Figure 18.18 - Sandymount Strand Promenade Looking North to ESB Poolbeg

Figure 18.19 - Sandymount Strand Promenade Looking South

18.5.11 Merrion Gates

This area of shoreline consists of the inter-tidal sand flats of Sandymount Strand, backed by a small number of residential properties, and then Strand Road and the dart line, as shown on Figure 18.16. Views from the residential properties fronting the shore and from the promenade extend out across Sandymount Strand and Dublin Bay. Informal recreation takes place on the beach but is limited by the lack of parking, with the focus placed on the promenade further north. The whole of the inter-tidal area lies within the Sandymount Strand (South Dublin Bay) Natural Heritage Area (NHA), Special Area of Conservation (SAC) and Special Protection Area (SPA). A site synopsis of the SPA is presented in Appendix R2.

Figure 18.20 - Merrion Gates

18.5.12 River Dodder

The River Dodder outfalls into the River Liffey immediately upstream of the East Link Bridge, as shown on Figure 18.15. The river is situated on the south side of the Liffey, and passes through residential and commercial areas. There are no designated nature conservation sites along the tidal length of the River Dodder. Views along or of the Dodder are variable, some lengths have ready access to pedestrians whilst elsewhere the river backs onto residential or commercial premises, minimising the visual amenity derived from the river. However, where access is available to the public extensive informal recreational activity does occur.

Figure 18.21 - Typical River Dodder Sections

18.5.13 River Liffey

The River Liffey is channelised over much of its length between the port and Island Bridge Weir. Through the centre of Dublin the channel sides consist of high stone walls, forming the quayside. For the length of the river examined for this study the river runs through the commercial heart of the City, bounded on all sides by offices, shops and leisure facilities. Furthermore, key roads run parallel to the river, whilst much pedestrian and cyclist use is made of the quaysides and footpaths in the area. Views along the river are constrained by the densely built up nature of its surroundings, though long views up and down the river give provide a visual amenity that is both immensely eventful and permanently engaging. The river is not designated for nature conservation interest, though it does support marine and estuarine species of fauna (invertebrates and fish) and is the migratory route for salmon, sea trout and common eels.

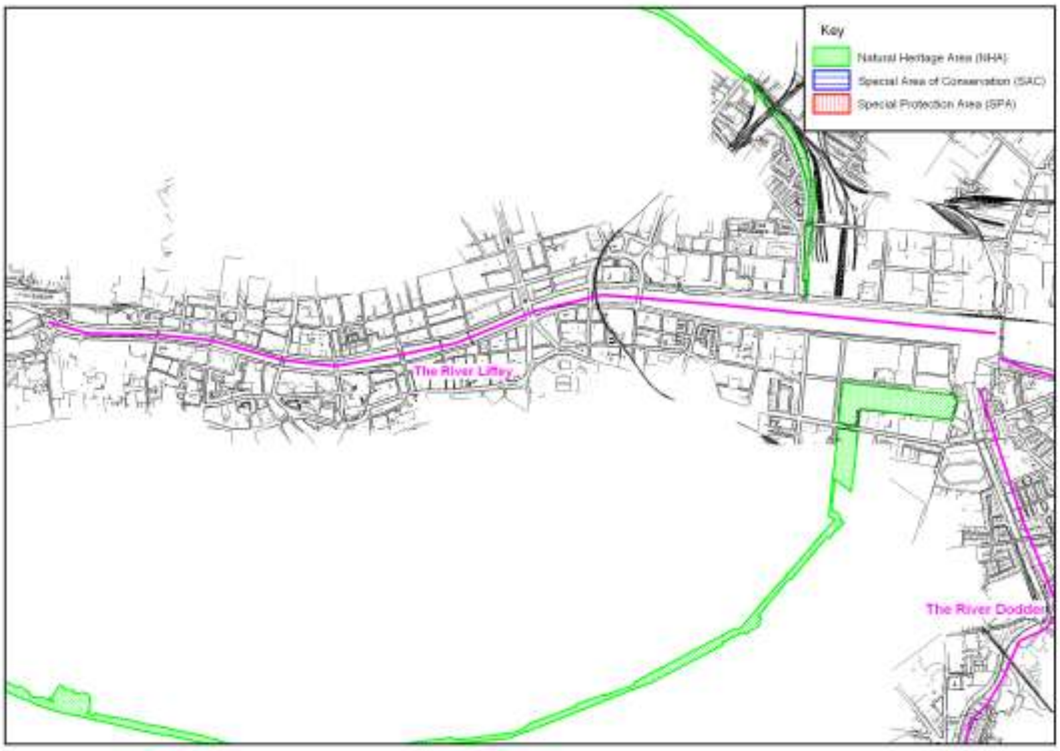


Figure 18.22 - River Liffey Frontage

18.6 DO NOTHING SCENARIO

18.6.1 Introduction

In this section, we identify whether any effects are anticipated in the future upon the existing environment and its assets. The major concern which underlines the purpose of the proposed coastal protection project is the issue of coastal flooding, and its various impacts upon the economic, social and environmental assets of the study area. The potential impacts associated with the various key areas are briefly identified in the following sub-sections.

18.6.2 Baldoyle Bay

The surrounding area of Baldoyle Bay is likely to be effected by flooding in the long term. In particular, a small number of residential properties would continue to be affected, and are likely to be increasingly affected (moderate adverse impact). Furthermore, the coastal road would flood during extreme events causing significant disruption to local traffic, inhibiting both residential and emergency access (major adverse impact).

No significant changes are anticipated in relation to the habitats within the study area, although there is a potential that the watercourse near the Portmarnock roundabout could over a long period of time become brackish in nature, however, it is the likelihood of the ingress of tidal waters that could affect the agricultural land behind the existing coastline, resulting in a loss to the agricultural asset of the area, but potentially increasing the area of saltmarsh within Baldoyle Bay.

18.6.3 Baldoyle Town

This area and frontage is protected by coastal defences, and consequently, the potential long term impacts would be dominated by flooding caused by overtopping. However, in the long term possible deterioration of the defences could result in potentially significant flooding events. Overall, impacts would mainly be related to the flooding of residential and commercial property (moderate adverse impact) and roads (moderate adverse impact), though the informal recreation asset of the promenade could deteriorate.

No significant long term changes are anticipated in relation to the nature conservation interests, or other aspects such as water quality.

18.6.4 North Howth

The area likely to flood is access points to the beach and roads (moderate adverse impact), consequently, this would continue in the long term. However, this area of shoreline is believed to be eroding, which is fairy clear on the dune system, and potentially further erosion could result in far greater flooding, with resulting impacts on properties and roads. Continued erosion would also be disastrous for this small dune system, but the causes are not known, e.g. whether it is due to recreational pressure, a change in coastal processes or sediment movements or a combination of those or other factors.

18.6.5 Howth South West

In the long term, the shoreline in this area would continue to experience flooding of the roads (major adverse impact) and subsequent impact on traffic and access. Potentially, long term deterioration in the coastal defences could result in far greater flooding with more serious impacts on residential properties (major adverse impact).

No significant long term changes are anticipated in relation to the nature conservation interests, or other aspects such as water quality within this area.

18.6.6 Clontarf

In the long term flooding would continue during extreme events, affecting roads (major adverse impact), residential and commercial properties (moderate adverse impact), and recreational access (minor adverse impact) with the associated impacts of economic loss, access closure, stress, etc. There are no significant long term changes likely within this study area, other than the continued rate of sediment accretion within the Tolka Estuary.

18.6.7 East Link Toll Bridge

Due to the built up nature of this area, the likely affects of the do nothing scenario would be those caused by flooding as a result of extreme events, which affected traffic and access (major adverse impact). No significant long term changes are anticipated in relation to nature conservation interests, or other aspects such as water quality.

18.6.8 ESB Poolbeg Power Station

The southern side of the power station and the access road to the South Bull Wall were flooded due to overtopping, which caused disruption to access (minor adverse impact). The impact on access would continue in the long term, but no other significant changes are anticipated on the nature conservation or other interests.

18.6.9 Sandymount Strand North of Existing Promenade

In the long term flooding would continue during extreme events, affecting roads (major adverse impact), residential properties (major adverse impact), and recreational access (minor adverse impact), with the associated impacts of economic cost, access and traffic disruption, stress, etc. In addition, areas of Sean Moore Park would also be flooded affecting the recreational interest. There are no significant long term changes likely on other interests in this area.

18.6.10 Sandymount Strand Existing Promenade

In the long term flooding would continue during extreme events, affecting roads (major adverse impact), residential properties (major adverse impact), and recreational access (minor adverse impact) with the associated impacts of economic cost, access and traffic disruption, stress, etc. In addition, the promenade areas would also be flooded affecting the recreational interest. There are no significant long term changes likely on other interests in this area.

18.6.11 Merrion Gates

In the long term flooding would continue during extreme events, affecting the road (major adverse impact), dart line, and a small number of residential properties (moderate adverse impact), with the associated impacts of economic cost, access and traffic disruption, stress, etc. There are no significant long term changes likely on other interests in this area.

18.6.12 River Dodder

In the long term flooding would continue during extreme events, and could potentially occur as a result of combined freshwater and tidal events though these are outside the remit of this study. A number of roads (major adverse impact), a large number of residential properties and some commercial properties (major adverse impact), and recreational amenity (minor adverse impact) would be affected, with the associated impacts of economic cost, access and traffic disruption, stress, etc. There are no significant long term changes likely on other interests in this area, however, flood events could potentially disturb the riverine habitats, causing erosion of bed deposits and other potential long term impacts.

18.6.13 River Liffey

In the long term flooding would continue during extreme events affecting the roads (major adverse impact) immediately adjacent to the river, as well as potentially affecting a number of commercial properties (moderate adverse impact) adjacent to the river, as well as recreational amenity (minor adverse impact). In addition, pedestrian access alongside the river would also be disrupted. There are no significant long term changes likely on other interests in this area.

18.7 Option Appraisal

18.7.1 Introduction

This section identifies the potentially significant impacts associated with the identified options available for coastal and flood defence. The detail of each option is not available at this stage, however, a general indication of the key areas that may be affected can be presented based on the site visit, knowledge of the area, and other data obtained for this study. Where possible a determination has been made of whether an Environmental Impact Assessment (EIA) would be required for specific options.

Where a type of asset or physical function or long term, cumulative or in-combination effects are not described, this infers that no significant or noticeable impact is predicted. The current level of assessment is to ascertain the "key" issues likely for each scheme option, rather than a substantive and quantifiable assessment of each option.

Where relevant some options have been aggregated together, due in part because of the need for success in providing flood and coast protection for a flood compartment.

18.7.2 Baldoyle Estuary

Location 1 – North of Baldoyle Town Centre

All of the options would provide a sufficient standard of protection such that flooding would be prevented for extreme events up to a 1 in 200 year event. Consequently, flooding of the road and properties along this frontage would be prevented, thereby resulting in a moderate beneficial impact with respect to traffic and access, as well as to the local community and residents.

Due to the location of the works adjacent or within the Baldoyle Bay NHA, cSAC and SPA, it is likely that an EIA may be required for all options.

Option 1 – Bermed Earth Bank

Although this is a soft form of coast protection, the footprint of the bank would extend into the Baldoyle Bay NHA, cSAC and SPA. A loss of saltmarsh habitat would occur, albeit a very small percentage of that within the Bay as a whole. The loss of saltmarsh could potentially affect bird species for which the site is internationally designated. However, due to the proximity of the area to residential properties, footpath and road, the existing level of disturbance (assumed) may minimise the significance of this area for birds. Overall, it is anticipated that a minor adverse impact would occur, which would be slightly offset by the beneficial habitat created by the embankment.

Views from residential properties and the road could be obstructed by the defence due to a higher level than the existing situation. However, the obstruction would not extend far above the existing road level, and the soft landscaping would soften the character of the obstruction, consequently a minor adverse impact is anticipated.

Construction works would be minor in scale, with some traffic disruption, noise and other related disturbance, albeit for the short term. However, timing of construction could result in increased impact, particularly if it results in the disturbance to wintering wildfowl within the SPA.

Option 2 – Earth Bank

Similar impacts would be expected as those for Option 1. However, the footprint of the embankment is slightly less, resulting in a lesser, but similar in scale, impact on the saltmarsh habitat and designated site. This embankment would need to be higher so the obstruction to views would be slightly greater, but again similar in scale to the impact identified in Option 1.

Option 3 – New Sea Wall

The footprint of the wall would not extend noticeably within the Baldoyle Bay NHA, cSAC and SPA. Consequently, little or no loss of saltmarsh habitat would be expected, and no loss of feeding habitat for bird species and the designated interests, therefore, no impact is anticipated.

The raised wall would result in a visual obstruction of a similar magnitude to Options 1 and 2, albeit the height of the wall is slightly lower than for Option 2, however, the man made appearance of the wall is likely to result in a slight reduction in the visual amenity from the road and properties. Overall, a minor adverse impact is anticipated.

As with the other options, construction works would be minor in scale. Again, timing of construction could result in an increased impact, particularly if it results in the disturbance to wintering wildfowl within the SPA.

Location 2 – South of Mayne River

Both options would provide a sufficient standard of protection such that flooding would be prevented for extreme events up to a 1 in 200 year event. Consequently, flooding of the road and residential properties along this frontage would be prevented, thereby resulting in a <u>moderate beneficial impact</u> with respect to traffic and access, as well as to the residents.

Due to the location of the works adjacent or within the Baldoyle Bay NHA, cSAC and SPA, it is likely that an EIA may be required for both options.

Option 1 – Earth Bank and New Wall

Similar impacts would be expected as for Option 1 at Location 1. The existing interests are very similar, although the level of the embankment could be high in relation to the residences, though only from a ground floor outlook.

Option 2 – Earth Bank and New Wall replacing Gabions

Similar impacts would be expected as those for Option 1. However, the footprint taken for this option would be slightly less, due to the use of gabions along a stretch, resulting in a lesser, but similar in scale, impact on the saltmarsh habitat and designated site.

This embankment would need to be higher so the obstruction to views would be slightly greater, but again similar in scale to the impact identified in Option 1.

Location 3 – North of Mayne River

Option 1 – Earth Bank

This option would provide a sufficient standard of protection such that flooding would be prevented for extreme events up to a 1 in 200 year event. Consequently, flooding of the road along this frontage would be prevented, thereby resulting in a moderate beneficial impact with respect to traffic and access.

Due to the location of the works adjacent or within the Baldoyle Bay NHA, cSAC and SPA, it is likely that an EIA may be required.

Similar impacts would be expected as for Option 1 at Location 1. The existing interests are very similar, although there are no residential properties that would be affected by the visual obstruction caused by the embankment.

Location 4 – North Western End of Baldoyle Estuary

Option 1 – New Earth Banks, New Flood Water Storage Area and Raise Road Levels

This option would provide a sufficient standard of protection such that flooding would be prevented for extreme events up to a 1 in 200 year event. Consequently, flooding of the road and properties along this frontage would be prevented, thereby resulting in a moderate beneficial impact with respect to traffic and access, as well as to the local community and residents.

This option would also prevent the tidal inundation of the agricultural land behind the road. However, there is insufficient information to ascertain whether the effects of this would be beneficial or adverse.

Due to the location of the works adjacent or within the Baldoyle Bay NHA, cSAC and SPA, it is likely that an EIA may be required for this option.

Similar impacts would be expected as for Option 1 at Location 1, as the existing interests are similar.

Location 5 – Southern End of Portmarnock

These two options would provide a sufficient standard of protection such that flooding would be prevented for extreme events up to a 1 in 200 year event. Consequently, flooding of the road along this frontage would be prevented, thereby resulting in a moderate beneficial impact with respect to traffic and access. In addition, the public house car park would also be protected from flooding.

Due to the location of the works adjacent or within the Baldoyle Bay NHA, cSAC and SPA, an EIA may be required for these options.

Option 1 – New Short Walls and Raise Ground Levels

The proposed footprint for this option would be on terrestrial land and is not anticipated to extend into the designated site area, consequently, no impact is anticipated.

Due to the low height of the walls and land raising, no alteration to the visual amenity or character would occur, and no impact is anticipated.

Construction works would be minor in scale, with some noise and recreational access being the notable issues albeit for the short term only. Timing of construction works could result in an impact due to disturbance to wintering wildfowl within the SPA, though this is considered to be unlikely.

Option 2 – New Earth Banks

Similar impacts and scale of impacts for Option 1 at this location are anticipated. However, due to the addition of the flood protection to the public open space, a minor beneficial impact would be expected on this amenity asset.

18.7.3 Baldoyle Town

These two options would close of the gaps in the existing defences thus preventing flooding occurring through the existing gaps. Consequently, flooding of the road and residential properties along this frontage would be prevented, thereby resulting in a moderate beneficial impact with respect to traffic and access, as well as to the local community and residents.

As none of the works are anticipated to take place within the designated site, it is unlikely that an EIA would be required.

Option A

The scheme would be undertaken within the footprint of the existing promenade, so no loss to habitats would occur, therefore, no impact is expected on the designated site and its interests.

Gaps for access would be provided, consequently no impedance would occur to recreational access and no impact expected.

The defences would be demountable, and therefore no visual obstruction or reduction in amenity would occur, except during extreme storm events, so no impact would be expected.

Construction works would be minor in scale, with some traffic disruption, noise and other related disturbance, albeit for the short term. However, timing of construction could result in increased impact, particularly if it results in the disturbance to wintering wildfowl within the SPA, however, this is unlikely.

Option B

Similar impacts and scale of impacts for Option 1 at this location are anticipated.

18.7.4 North Howth

No specific works identified or required at this stage. However, as part of any geomorphological studies in the future it is recommended that a detailed evaluation of environmental issues is undertaken in parallel.

18.7.5 Howth South West

Potentially some of the options are likely to require an EIA as they may extend into the Bull Island NHA, cSAC and SPA. Those options that remain alongside the designated site boundary without intruding into it are not likely to require an EIA.

All options would provide protection to the amenity, road, and residential properties along the Greenfield Road and Strand Road frontage, thus preventing the economic and social losses associated with flooding, as well as the impacts on access, traffic disruption, stress and other suffering within the local community and residents.

Option 1 – New Rock Revetment and Crest Wall

This option involves minor encroachment of the toe of the revetment and slipway into the designated site area. Most of this would then be covered by existing beach material. However, the incursion into the site boundary alone raises potential issues, however, the area of encroachment compared to the area of the designated site is extremely small, though could result in a limited loss of interest habitat, and subsequent loss of feeding area for wildfowl and waders. No detailed information is available regarding wildfowl and wader usage of the area that may be affected. However, due to the small scale of the loss and proximity to a recreational footpath, it is anticipated that a minor adverse impact may arise.

The existing frontage contains a footpath, which would be protected from flooding during extreme storm events. As the footpath is of local value and infrequently affected by flooding, this is considered to result in a minor beneficial impact on recreational amenity. This also includes the benefit of retaining slipway access for both recreation and maintenance/emergency access.

Due to the localised and limited extent of works (predominantly raising the revetment height), no noticeable change is anticipated to the shoreline or the hydrodynamic regime in the area, consequently, no impact is expected.

Due to the raising of the revetment by nearly 1.5m, views from residential properties set back from the defences may be obstructed, though view from the footpath would be obstructed unless some land raising was undertaken along the length of the footpath. However, the distance from the revetment and limited extent of the obstruction is anticipated to result in a minor adverse impact. However, further work regarding property levels would need to be undertaken to suitably quantify this potential impact.

Construction works could be kept to a minimal area of disturbance, affecting only the recreational asset of the grassy area and immediate area of the coastal defence line, though some potential traffic disruption may occur at times. Construction noise and disturbance (dust, visual amenity) would also be expected but not significant in scale or magnitude. However, if the works were carried out in winter there is a potential that associated works (noise, machinery, personnel) could disturb wintering wildfowl and waders within the Bull Island SPA. This could have a potentially significant adverse effect on the wildfowl interest of the designated site, resulting in a potential major adverse impact on the designated site and its context. However, timing the works outside the wintering period would almost certainly avoid this impact.

Option 2 – New Rock Revetment and Flood Wall Landward of Existing Footpath

This option is fairly similar to Option 1, however, there are some minor differences which are described below.

As the revetment is set back, there would be less or no incursion into the designated site area, consequently, no impact is anticipated on the designated site interest or habitat.

The footpath would not be protected, consequently, the same impact of the do nothing scenario would occur.

The height of the new revetment would be 0.25m lower than Option 1, consequently, that and the fact that views from the footpath would not be obstructed by the defence indicate that no impact is anticipated in respect of landscape and visual amenity.

Option 3 – New Rock revetment with Flood Wall along Greenfield Road

Due to the similarity with Option 2, the same impacts are anticipated.

Option 4 – New Concrete Seawall and Re-pointing of Existing Wall

This option would entail raising of the sea wall along a distance of around 400m. This would not result in encroachment into the designated site, or may entail the placement of rock below the existing level of sand that could therefore be reinstated. Consequently, no impact is anticipated on the designated site or its nature conservation interest.

Due to the large increase in height above the existing ground level (greater than 1.5m in places) localised raising would be undertaken to maintain pedestrian views. However, a large number of properties set back from the road would experience an obstruction to views from ground floor levels. Overall, this is anticipated to result in a minor to moderate adverse impact on visual amenity.

Construction works would be extensive and would result in the loss of width of the road and also recreational parking area, consequently, there would be traffic and access disruption during the works. Construction noise and disturbance (dust, visual amenity) would also be expected and could be locally significant in magnitude, however, this could be mitigated to some extent. The works would entail disturbance to the immediate shoreline adjacent to the sea wall, and if the works were carried out in winter there is a potential that associated works (noise, machinery, personnel) could potentially disturb wintering wildfowl and waders within the Bull Island SPA. These could have a potentially significant adverse effect on the wildfowl interest of the designated site, resulting in a potential moderate adverse impact on the designated site and its context. However, timing the works outside the wintering period would almost certainly reduce this impact, whilst the disturbance of beach during construction would cease as soon as works were completed.

Option 5 – Repaired/New Seawall with Set Back Flood Wall

Although this option predominantly entails the construction of a set back flood wall, there is a stretch where the sea wall would need to be built sea ward of the existing wall, consequently extending into the designated site. Because there is no detailed information of wildfowl and wader usage of the area, or other details relating to designated site interest, it is assumed that this would result in a potential moderate adverse impact.

This option would result in the creation of a 6m wide promenade, which would provide safe and enhanced informal recreational amenity and improved views across Bull Island and Dublin Bay. Overall, this is considered to result in a minor beneficial impact on recreational amenity, due to the relatively low level of use of the area for informal recreation.

Due to the localised and limited extent of works (predominantly raising the revetment height and only limited incursion into the sea), no noticeable change is anticipated to the shoreline or the hydrodynamic regime in the area, consequently, no impact is expected.

Due to the wall being set back, the wall height would only increase by a maximum of around 1m. This would result in a reduction in the scale of obstruction from residential properties set back from the road, however, not sufficiently to reduce any potential impact. Therefore, a minor adverse impact is expected to remain as identified for Option 4.

Similar impacts during construction would be expected to those identified for Option 4.

18.7.6 Clontarf

Overall, except for the breakwater option, an EIA is not likely to be required for the coast defence options at Clontarf.

Most of the options would provide protection to the amenity, road, residential and commercial property along the Clontarf frontage, thus preventing the economic and social losses associated with flooding, as well as the impacts on access, traffic disruption, stress and other suffering within the local community and residents. Some options may not protect the recreational amenity and these will be noted.

Option 1 – Raising the Existing Flood Defences

The footprint of this option would not impinge on the Tolka Estuary cSAC, SPA and Ramsar site that covers the inter-tidal area of Tolka Estuary. Consequently, no impact is foreseen on the designated site and its interest.

The existing frontage contains a promenade area that would be protected from flooding during extreme storm events, though the protection would only maintain the access for a short period of additional time, resulting in a minor beneficial impact on recreational amenity.

No change would be expected to the shoreline or the hydrodynamic components of the Tolka Estuary as a result of the raising of the sea walls, consequently, no impact would occur.

Due to the raising of the sea wall obstruction and therefore change in the visual amenity of the shoreline from the road and properties immediately adjacent to the length of frontage altered would occur. Raising of the promenade would be undertaken to maintain the visual amenity of for informal recreation and pedestrians. In addition, the use of planting and landscaping to soften the character of the change would minimise the potential impact such that a limited degree of obstruction would occur. Overall, this is anticipated to result in a minor adverse impact. However, further work regarding property levels and levels of the defence would need to be undertaken to suitably quantify this potential impact.

Construction works could be kept to a minimal area of disturbance, affecting only the recreational asset of the promenade (or lengths of it) during works, with some minor traffic disruption at times. Construction noise and disturbance (dust, visual amenity) would also be expected but not significant in scale or magnitude. However, if the works were carried out in winter there is a potential that associated works (noise, machinery, personnel) could disturb wintering wildfowl and waders within the Tolka Estuary SPA. This could have a potentially significant adverse effect on the wildfowl interest of the designated site, resulting in a potential major adverse impact on the designated site and its context. However, timing the works outside the wintering period would almost certainly avoid this impact.

Option 2 – New Set Back Re-curve Seawall with New Promenade

The impacts associated with this option are similar to those for Option 1, except for the following aspects.

The existing promenade would not be protected from flooding during extreme storm events, consequently, a minor adverse on recreational amenity is expected.

Although a slightly lower wall height would be required, due to it being set back closer to residential properties, it is not anticipated to result in a significant reduction in obstruction of views compared to Option 1. Consequently, a minor adverse impact is also expected for this option.

Option 3 – Replacement of the Secondary Wall

This option is similar to Option 2 except for the following aspect.

The set back wall would only be raised by a small level, and consequently, limited change to the existing views is anticipated, however, the promenade would flood during extreme storm events. Overall, no impact is anticipated.

Option 4 – Offshore Breakwater

The breakwater would be placed within the Tolka Estuary, thus resulting in the loss of an area of the designated site (Tolka Estuary cSAC and SPA). This loss of habitat, albeit not significant in terms of the area of the estuary in total, could affect the population of the wildfowl and waders for which the site is designated. Further work would be required to ascertain the scale of the impact, which could be determined using existing bird counts within the estuary and surrounding area, though this is outside the remit of this report. Overall, a minor to moderate impact is expected due to the direct loss of habitat, however, the impact could increase in scale to moderate to major adverse impact depending on the effect of the breakwater on hydrodynamics.

The presence of additional structures within the estuary could potentially effect the movement of recreational vessels. The Clontarf Yacht and Boat Club use the estuary for sailing, particularly for beginners, whilst more experienced sailors use the estuary to access to the River Liffey and Dublin Bay beyond. Overall, the breakwaters could result in additional obstructions to recreational navigation, and overall a moderate to major adverse impact is predicted.

Without the results of a hydrodynamic model and assessment, the effect of the breakwaters on sediment erosion and accretion patterns cannot be ascertained. Consequently, the indirect effects of these upon the designated site interest, or even water quality, cannot be determined.

The presence of additional man made structures within the open waters and inter-tidal mudflat of the Tolka Estuary would result in a change in character and also some limited obstruction of parts of the estuary. In terms of character change the man made character of the port in the background and the North Bull Wall and South Bull Wall all minimise the additional effect of the breakwaters. Nonetheless, their presence and obstruction of views within the open and unbroken vista of Tolka estuary is expected to result in a localised moderate adverse impact.

There are historic records of a number of ship losses within Tolka Estuary and along the Clontarf frontage, as well as within the River Liffey and area of the port. Although there is no indication of exact wreck sites within the study area, there is a potential for the footprint of the breakwaters to overlie possible archaeological and historical artefacts present within any preserved site. In order to ensure that this does not occur, further investigations would be necessary (such as geophysical survey) to ensure the absence of any potential wrecks.

Construction works within the Tolka Estuary SPA could result in significant disturbance to wildfowl and waders, particularly so if the works were carried out in the wintering period. At present as the exact scale of the works is not known a precautionary approach is used, consequently, a potential moderate to major adverse impact could occur. Other than some disturbance to visual amenity and some limited noise from the work areas, no other significant impacts are anticipated.

18.7.7 East Link Toll Bridge

It is unlikely that the works alongside the toll road would require an EIA as the two options are limited to the existing infrastructure.

The two options would prevent the further flooding of the road and immediate area during extreme storm events.

Option 1 – Combined Traffic Barrier and Flood Defence

There are no designated sites of conservation interest in or adjacent to the scheme area, consequently, no direct or indirect impacts are anticipated.

There are no recreational assets or immediate residential properties, consequently, no impacts are expected to arise.

There are no changes to the physical characteristic of the River Liffey alongside the toll road, and no additional inputs to water, therefore no impact is expected.

The use of the existing crash barrier feature as a means of integrating any flood defences into the surrounding character of the area ensures that no noticeable change is anticipated on the landscape, and no impact would occur.

It is unlikely that the works would disturb preserved archaeological remains. Although the site lies in an area of the city that would have been a focus of the earlier activities associated with the port, earlier construction works associated with the toll road would have removed any surviving remains. Therefore, no impact is anticipated on the archaeological resource.

Construction works are unlikely to significantly affect any residential properties due to their relative distance from the proposed works. However, due to the proximity to an extremely busy road construction works could potentially result in significant traffic disruption.

Option 2 – New Flood Wall

This option is very similar in character and extent to Option 1, in terms of the potential environmental impacts.

18.7.8 ESB Poolbeg Power Station Options

The proposed works at ESB Poolbeg are not likely to require an EIA, though the construction works could potentially result in a significant adverse impact on the SPA interest, if the works occurred during a sensitive time of the year.

The proposed option would provide protection to the industrial property along this frontage from flooding associated with extreme storm events.

The embankments are situated behind the access road to South Bull Wall, and are not located within the Sandymount SPA, consequently, no direct impact is anticipated on the designated site or its interest. As the embankment is present to prevent splash flooding, no hydrodynamic changes would be expected within the designated site, consequently, no indirect impacts are predicted on the designated site and its interest.

The embankments would be landscaped and planted. The immediate surroundings consist of grassy areas and vegetated scrub habitat, therefore, the embankment would be of similar character, consequently, no impact is expected on the landscape.

Construction works adjacent to the Sandymount SPA could potentially result in disturbance to wildfowl and waders, particularly so if the works were carried out in the wintering period. Due to the scale of the potential impact and the location of the works the effect is anticipated to result in a short-term moderate adverse impact on the designated site interest at worst. However, due to the existing disturbance of vehicles and people passing along the access road the magnitude of the disturbance could potentially be of negligible scale. Further work would be required to accurately determine the scale of the impact. No other significant impact is anticipated during construction works at this location.

18.7.9 Sandymount Strand, North of Existing Promenade

Potentially a number of options are likely to require an EIA as some options may extend into the Sandymount SPA. Those options that remain alongside the designated site without intruding into it are not likely to require an EIA.

All options would provide protection to the amenity, road, and residential properties along the Sandymount Strand frontage, thus preventing the economic and social losses associated with flooding, as well as the impacts on access, traffic disruption, stress and other suffering within the local community and residents.

Option 1 – Raise Existing Masonry and Stonework Wall

This option would entail raising of the sea wall along, and may entail some small encroachment into the designated site (up to a meter) to maintain stability of the wall. No details are available regarding wildfowl and wader use along this immediate frontage, so limited assessment can be undertaken, however, there is relatively constant use of the upper beach along this frontage, which already results in disturbance. Consequently, a minor adverse impact is anticipated on the designated site and its nature conservation interest. In the event that rock groynes are required along this frontage this impact is likely to increase but would not exceed being moderately adverse.

Due to the large increase in height above the existing ground level a large number of properties set back from the road would experience an obstruction to views from ground floor levels, as well as for pedestrians. Overall, this is anticipated to result in a potential moderate adverse impact on visual amenity. However, comparison of property levels with wall heights would need to be undertaken to suitably quantify this impact.

Construction works would be extensive and would result in the loss of width of the road and resulting in traffic and access disruption during the duration of the works. Construction noise and disturbance (dust, visual amenity) would also be expected and could be locally significant in magnitude, however, this could be mitigated to some extent. The works would entail disturbance to the immediate shoreline adjacent to the sea wall, and if the works were carried out in winter there is a potential that associated works (noise, machinery, personnel) could potentially disturb wintering wildfowl and waders within the Sandymount Strand SPA. These could have a potentially significant adverse effect on the wildfowl interest of the designated site, resulting in a potential minor to moderate adverse impact on the designated site and its interest species. However, timing the works outside the wintering period would almost certainly reduce this impact, whilst the disturbance of beach during construction would cease as soon as works were completed.

Option 2 – Raise Existing Wall to 4.2mODM and Build New Rock Revetment at Toe

The effects of this option are fairly similar to those for Option 1, except for the following aspects.

The option would extend the footprint of the defences (as a result of the new revetment) into the Sandymount Strand NHA, cSAC and SPA. Currently, there is insufficient information regarding the density and use of the area of the shore that would be affected to quantify the potential effect of the loss upon the interest species of the designated site. Consequently, it is assumed that a moderate adverse impact would occur.

Without the results of a hydrodynamic model and subsequent assessment of results, the effect of the incursion into the Bay cannot be ascertained. Consequently, the indirect effects of these upon the designated site interest, or even water quality, cannot be determined.

The increase in defence height would be less than for Option 1. However, because extensive detail regarding property levels and wall heights is not available, it is assumed that this would still result in obstruction of views from properties and for pedestrians on the path adjacent to the road. Consequently, this is anticipated to result in a minor adverse impact on visual amenity.

Option 3 – Extend Promenade North to Sean Moore Park and Raise Existing Wall to 4.2mODM

The option would extend the footprint of the promenade into the Sandymount Strand NHA, cSAC and SPA. Currently, there is insufficient information regarding the density and use of the area of the shore that would be affected to quantify the potential effect of the loss upon the interest species of the designated site. Consequently, it is assumed that a major adverse impact would occur.

The creation of additional frontage and promenade would extend the recreational resource that this provides along the majority of the Sandymount frontage, extending it all the way to the north to Sean Moore Park. Because of the informal recreational resource that this would enhance, this is anticipated to result in a moderate beneficial impact.

The extension of the defended shoreline and loss of sandy shore habitat is negligible in scale in comparison to the Bay as a feature, and the continued man made defences would not significantly alter the character of the frontage. Furthermore, views toward the frontage would not be perceptibly altered due to distance and scale of viewpoints, therefore, no impact is anticipated on landscape.

Access Gap Sean Moore Park

These options provide pedestrian and disabled access at Sean Moore Park, whilst preventing flood water passing into Sandymount area behind the defences. The options are all fairly similar in scale and location, and consequently, would result in similar impacts. However, due to the extremely localised nature of the options, no significant issues are likely to arise as they all provide access into Sean Moore Park.

18.7.10 Sandymount Strand, Existing Promenade Options

As the options are specifically related to the prevention of splash flooding with no incursion into the Sandymount Strand NHA, cSAC and SPA, it is unlikely that an EIA would be required.

All options are presented in relation to particular lengths of the promenade and frontage rather than particular defensive types, all would need to be undertaken as one scheme to provide a coherent and frontage wide defence. With all lengths constructed, the scheme would provide protection to the amenity, road, and residential properties along the entire frontage, thus preventing the economic and social losses associated with flooding, as well as the impacts on access, traffic disruption, stress and other suffering within the local community and residents.

The options would not extend into the Sandymount Strand NHA, cSAC and SPA, consequently, there would be no impact on the designated site or its interest features and species.

The scheme would provide protection to the promenade but would not prevent flooding of the whole recreational asset during extreme storm events. However, overall, due to the limited frequency and duration of extreme storm events this is not expected to result in any noticeable effects, and no impact is expected.

As there is no alteration to the shoreline, no discernible change is expected in relation to hydrodynamic processes along the frontage. Consequently, there would be no impact on water related aspects.

The scheme options do not result in any alteration in the visual character of the frontage, though minor changes would occur, though not resulting in any significant obstruction of views. Consequently, no impact is anticipated on landscape along the frontage.

Construction works adjacent to the Sandymount Strand NHA, cSAC and SPA could potentially result in disturbance to wildfowl and waders, particularly if the works were carried out in the wintering period. At present the usage of the immediate area is not known (for summer or winter), so a precautionary approach is used, and the worst case effect is predicted to be a potential moderate adverse impact. Other construction impacts include disturbance to visual amenity (from residences and roadside), construction noise, and possible disturbance to traffic due to work adjacent to the Strand Road. In addition, there would be disruption to access to the promenade and beach during construction, which would be of greater significance in summer months. Overall, however, these are not likely to be significant in scale or magnitude, and could be mitigated to a great degree by sensitive planning and construction methods.

18.7.11 Merrion Gates

The options here are very minor in scale, however, where incursion into the Sandymount Strand NHA, cSAC and SPA occurs as a result of an extended footprint of the two options, an EIA may be required.

These two options would provide protection to the amenity, road, Dart line, and residential properties along the entire frontage from splash flooding. This would prevent the economic and social losses associated with flooding, as well as the impacts on access, traffic disruption, stress and other suffering within the local community and residents.

Option 1

Although minor in scale, expansion of man made features into the Sandymount SPA would result in loss of habitat and a potential effect on the designated site interest features and species. Using the precautionary principle, due to the very minor scale of the likely footprint and relatively remote location of the area a minor adverse impact is anticipated.

The existing access to the beach would be altered, however, access would still remain, therefore, no impact is anticipated.

Construction work would be negligible in scale, resulting in localised disturbance to the local residence and possible disruption to traffic. Depending on the footprint of the works area and whether it extends into the designated site, there is a potential for disturbance to wildfowl and waders using the designated site, which could be potentially significant during the wintering period. Sensitive timing of the works would avoid or significantly minimise any potential construction impacts.

Option 2

Due to the similar scale and location of this option compared to Option 1, no additional change in the significance of any impact is identified.

18.7.12 River Liffey

The River Liffey is not designated for its conservation interest, and due to the nature of the options, it is unlikely that an EIA would be required.

As with the River Dodder all these "options" are combined as one option for the length of the Liffey. With all sections of proposed work undertaken, residential and commercial properties and a large number of roads would be protected from tidal flooding during extreme storm events. This would result in significant benefit in relation to economic costs, traffic and access disruption, and the community.

However, these hold the line options entail a variety of “types” of holding the line. The following impacts are those associated with holding the line generally, whilst for each method of doing so any potential differences to the following impacts are identified.

The work proposed along all sections is limited to work along the existing line of the river defences, consequently, there would be no loss of habitat, and no impact would occur.

As the works would retain the existing line of defence without alteration of the watercourse, no changes are likely on hydrology and hydrodynamic processes, and no impact expected.

There are many view points along the length of the river, however, the changes proposed are on the whole in character with the existing defences, and any alteration to the height of the defences is minor. Overall, therefore, a minor adverse impact is anticipated.

Although the quayside along the Liffey is of historic interest, development over the last few centuries is likely to have removed any potential archaeological or archaeological remains. Consequently, no impact is anticipated on the historical resource as a result of the proposed scheme.

The construction impacts of the proposed works are anticipated to be of greater significance than the long term "operational" effects. In particular, if works are to take place in the river there is a greater potential for works to affect in-river habitat. However, due to the scale of the likely works, and greater likelihood of works to occur on the landward side, no impact is currently envisaged.

Although all options have a potential for accidental pollution (from construction materials, effluents, etc.), the works within and immediately adjacent to the river would be of particular concern and should be mitigated within the construction methodology. This is particularly important within the Liffey because it supports a migratory salmon population.

Other construction impacts that would inevitably occur include aspects such as visual disturbance, potential traffic disruption, construction noise, and disruption to access. However, all these aspects should be readily minimised using standard approaches to mitigation, and would only occur throughout the duration of the works.

Flood Defence Types

The use of demountable defences would reduce the permanence of the visual intrusion of the defences, as they would only be present during extreme storm events. However, some aspects of the infrastructure for the demountable defences would be present. In comparison to hard defences, the demountable defences are in a minimum height size, which is generally greater than that actually required, so the permanent defences would be lesser in scale (height).

The sensitivity of the area in relation to landscape character change as a result of defences is difficult to adequately quantify for a number of reasons, the most significant is the suitability and sensitivity of the design of the permanent defences. In places permanent walls of suitable construction that fit into the surrounding character would have a lesser impact on both character and obstruction to the demountable defences.

Overall, it is the perceived view that successful and sensitive design of the permanent defences would not result in a noticeable greater impact than demountable defences.

18.7.13 River Dodder

Because the River Dodder is not designated for its conservation interest, and its relatively urban setting, it is unlikely that an EIA would be required. However, due to the potential for archaeological interest alongside the river and its visual amenity throughout the south of Dublin, there may be a requirement for more detailed investigation into the complexity of the schemes and the potential impacts through the formal EIA process.

The options proposed are specific to reaches of the river, rather than to types of option, except where alternative options to maintaining the existing line of defence arise. These will be considered separately. Consequently, all sections should be considered together as they form the "flood compartment" and the benefit of one section of works alone would be far less than the combined (synergistic) benefit of all sections of work. With all sections of proposed work undertaken residential and commercial properties, a large number of roads, as well recreational paths would be protected from tidal flooding during extreme storm events. This would result in significant benefit in relation to economic costs, traffic and access disruption, and the community.

However, these hold the line options entail a variety of “types” of holding the line. The following impacts are those associated wit hold the line generally, whilst for each method of doing so any potential differences to the following impacts are identified.

The work proposed along all sections is limited to work along the existing line of the river defences, consequently, loss of habitat within the river is avoided, and habitat immediately adjacent to the river would predominantly be affected during construction works. Overall, therefore, no significant loss of habitat and subsequent loss of diversity or density of flora and are expected, and no impact would occur in the long term.

As the works would retain the existing line of defence without alteration of the watercourse, no changes are likely on hydrology and hydrodynamic processes, and no impact expected.

There are many view points along the length of the river that would experience the changes resulting from the proposed works. These include residential and commercial properties, and footpaths and roads. The works will entail extension in terms of height and character of the existing defences. In all, the general character of the views would not change significantly, though due to raised defences the potential for obstruction would increase. However, in terms of the scale of the work, the overall effect is not anticipated to be significant in scale and would occur along specific reaches and stretches as opposed to significant reaches of the river. Therefore, a minor adverse impact is anticipated.

Due to the historic interest nearer the confluence with the River Liffey, and potential archaeological interest further upstream, potential adverse effects could occur as a result of the works. As the works would be undertaken along the existing defence line it is likely that limited disturbance would occur to preserved remains, consequently, a minor adverse impact is anticipated.

The construction impacts of the proposed works are anticipated to be of greater significance than the long term "operational" effects. In particular, disturbance to vegetation and habitat as a result of the works has the potential to disturb a wide variety of species, such as breeding birds. However, as insufficient information on the presence or absence of fauna is available, this remains a potentially significant impact, if mitigation measures are not incorporated into the construction methodology. Furthermore, the likely methodology of works would also influence the potential effects that could arise, for example, if works take place within the river itself further impacts could arise to species particularly fish. Overall, as a precautionary approach, a potential moderate adverse impact could occur. Mitigation measures could easily be implemented to minimise any potential impacts, though these can only be defined based on sufficient survey data.

Although all options have a potential for accidental pollution (from construction materials, effluents, etc.), the works within and immediately adjacent to the river would be of particular concern and should be mitigated within the construction methodology.
Other construction impacts that would inevitably occur include aspects such as visual disturbance, potential traffic disruption, construction noise, disruption to access, and possibly vibration. However, all these aspects should be readily minimised using standard approaches to mitigation, and would only occur throughout the duration of the works.

Location 3 – London Bridge to New Bridge

Option 1 – Raise or Construct a New Secondary Back Wall

This option would set back the primary defence, and overall the key differences in impact are the continued flooding of the footpath/promenade, which would result in a minor adverse impact. In addition, possible access issues may arise, though at present there is insufficient information to ascertain the impact.

18.7.14 River Tolka

This option entails works to the existing walls and as such is anticipated to result in predominantly construction oriented impacts, including noise, vibration, visual disturbance, and possible access and traffic disruption, albeit for the duration of the works only. An EIA is unlikely to be required. The benefit of undertaking these works is the protection from flooding of a large number of commercial and residential properties, as well as local access roads, resulting in significant beneficial impacts.

18.8 Summary of Potential Impacts

Table 18.4 to Table 18.8 present a visual summary of the potential impacts and key issues associated with each of the options across the study area, with regard to the Dublin Coastal Flood Protection Project.

The tables summarise the impacts described in Section 6. These impacts are based on general study area information (and knowledge of the area held by project personnel). Where possible the impacts have been quantified using a qualitative method as opposed to a quantitative method, therefore, much of the assessment presented in this document is based on assumptions regarding the proposed works and the potential impacts. Detailed information and assessment could alter the key impacts likely to be associated with each option, as well as the scale of the impact. Consequently, it should be borne in mind that this 'assessment' presents an indication of the key issues.

One aspect of determining the significance of an impact and the effects of an option are determining whether it is likely that a formal Environmental Impact Assessment would be required to carry out each specific option. As legal advice cannot be given by ourselves, we have indicated in Section 6 which options may be likely to require an EIA and which may not. In order to formally determine this requirement a screening opinion should be sent to the appropriate authority (e.g. the planning departments of Fingal County Council or Dublin City Council). However, in the summary tables the likelihood of an EIA being required is most likely when a moderate adverse impact is expected as a result of an option in its operational phase, or if there is a potential impact upon designated nature conservation sites (e.g. NHA, cSAC and SPA). Construction phase impacts are not likely to influence this requirement, except where a designated site may be effected, and whereby this effect cannot be avoided for example by undertaking works outside of the winter period.

18.9 Conclusions

On the whole, most options identified as possible methods of protecting areas within Dublin and Fingal County from coastal flooding are generally acceptable in environmental terms, especially when balanced against the social (and economic) costs of flooding and the extensive disruption it can cause. There are impacts associated with the options, particularly landscape, which are likely to be affected by increased heights and scale of coastal defence options. However, mitigation measures can be identified and expanded on at more detailed stages to minimise the scale of these impacts. Sensitive design having the most influence upon this!

Although many options appear to affect or potentially affect designated sites of national and international importance, this in itself does not indicate that these options are wholly unacceptable. As well as the necessity for flood protection some options are identified with health and safety aspects in mind, which can make such impacts on designated nature conservation sites acceptable. In addition, the assessment used to determine the effects on designated sites has been precautionary due to the lack of area specific information as well as information regarding changes to coastal processes, and could therefore reduce or be avoided through further assessment and identification of possible mitigation measures.

Table 18.4 - Summary of Potential Impacts on Fingal County Sites

Impact	Location and Option									
	Baldoyle - Do Nothing	Baldoyle 1 - 1	Baldoyle 1 - 2	Baldoyle 1 - 3	Baldoyle 2 - 1	Baldoyle 2 - 2	Baldoyle 3	Baldoyle 4	Baldoyle 5 - 1	Baldoyle 5 - 2
Operational Phase										
Flooding of road(s)	---	++	++	++	++	++	++	++	++	++
Flooding of residential properties	--	++	++	++	++	++		++		
Flooding of commercial premises	-								+	+
Flooding of agricultural land	-							?+		
Flooding of recreational amenity/access										+
Designated habitat (NHA, SAC, & SPA)		?-	?-		?-	?-	?-	?-		
Landscape		-	-	-	-	-		-		
Cultural heritage										
Construction Phase										
Designated habitat interest		?-	?-	?-	?-	?-	?-	?-	?-	?-
Traffic		-	-	-	-	-	-	-	-	-
Recreational amenity access									-	-
Noise		-	-	-	-	-	-	-	-	-
Visual disturbance		-	-	-	-	-	-	-	-	-

Key

+++	Major beneficial impact			-	Minor adverse impact		
++	Moderate beneficial impact		No/neutral impact	--	Moderate adverse impact	?	Potential (avoidable) impact
+	Minor beneficial impact			---	Major adverse impact		

Table 18.5 - Summary of Potential Impacts on Fingal County Sites

Impact	Location and Option								
	Baldoyle Town - Do Nothing	Baldoyle Town – Option A	Baldoyle Town – Option B	Howth South - Do Nothing	Howth South West – 1	Howth South West - 2	Howth South West - 3	Howth South West - 4	Howth South West - 5
Operational Phase									
Flooding of road(s)	--	++	++	---	+++	+++	+++	+++	+++
Flooding of residential properties	--	++	++	---	+++	+++	+++	+++	+++
Flooding of commercial premises	--	++	++						
Flooding of recreational amenity/access				-	+	-	-		+
Designated habitat (NHA, SAC, & SPA)					?-				?--
Landscape					-			- to --	-
Cultural heritage									
Construction Phase									
Designated habitat interest					?---	?--	?--	?--	?--
Traffic	?	?	?		-	-	-	-	-
Recreational amenity access		-	-		-	-	-	-	-
Noise					-	-	-	-	-
Visual disturbance		-	-		-	-	-	-	-

Key

+++	Major beneficial impact		-	Minor adverse impact		
++	Moderate beneficial impact	No/neutral impact	--	Moderate adverse impact	?	Potential (avoidable) impact
+	Minor beneficial impact		---	Major adverse impact		

Table 18.6 - Summary of Potential Impacts on Dublin City Sites

Impact	Location and Option									
	Clontarf - Do Nothing	Clontarf - 1	Clontarf - 2	Clontarf - 3	Clontarf - 4	East Link Toll - Do Nothing	East Lin Toll - Option 1	East Lin Toll - Option 2	ESB Poolbeg - Do Nothing	ESB Poolbeg
Operational Phase										
Flooding of road(s)	---	+++	+++	+++	+++	---	+++	+++	-	+
Flooding of residential properties	--	++	++	++	++					
Flooding of commercial premises	--	++	++	++	++					
Flooding of agricultural land										
Flooding of recreational amenity/access	-	+	-	-	-- to ---				-	
Designated habitat (NHA, SAC, & SPA)					?-- to ---					
Landscape		-	-		--					
Cultural heritage					?					
Construction Phase										
Designated habitat interest		?---	?---	?---	?-- to ---					?--
Traffic		-	-	-			--	--		-
Recreational amenity access		-	-	-						-
Noise		-	-	-	-					
Visual disturbance		-	-	-	--					

Key

+++	Major beneficial impact			-	Minor adverse impact		
++	Moderate beneficial impact		No/neutral impact	--	Moderate adverse impact	?	Potential (avoidable) impact
+	Minor beneficial impact			---	Major adverse impact		

Table 18.7 - Summary of Potential Impacts on Dublin City Sites

Impact	Location and Option								
	Sandymount N - Do Nothing	Sandymount N - Option 1	Sandymount N - Option 2	Sandymount N - Option 3	Sandymount S - Do Nothing	Sandymount S	Merrion Gates - Do Nothing	Merrion Gates - Option 1	Merrion Gates - Option 2
Operational Phase									
Flooding of road(s)	---	+++	+++	+++	---	+++	---	+++	+++
Flooding of residential properties	---	+++	+++	+++	---	+++	--	++	++
Flooding of commercial premises									
Flooding of agricultural land									
Flooding of recreational amenity/access	-	+	+	++	-	+	-	+	+
Designated habitat (NHA, SAC, & SPA)		?-	?--	?---			?-		
Landscape		--	-	-					
Cultural heritage									
Construction Phase									
Designated habitat interest		?- to --	?--	?---		?--		?-	?-
Traffic		-	-	-		-		-	-
Recreational amenity access		-	-	-		--			
Noise		-	-	-		-		-	-
Visual disturbance		-	-	-		-		-	-

Key

+++	Major beneficial impact		-	Minor adverse impact			
++	Moderate beneficial impact		No/neutral impact	--	Moderate adverse impact	?	Potential (avoidable) impact
+	Minor beneficial impact			---	Major adverse impact		

Table 18.8 - Summary of Potential Impacts on Dublin City River Sites

Impact	Location and Option						
	River Liffey - Do Nothing	River Liffey	River Dodder - Do Nothing	River Dodder - All Options	River Dodder - Location 3 - 1	River Tolka - Do Nothing	River Tolka
Operational Phase							
Flooding of road(s)	---	+++	---	+++		---	+++
Flooding of residential properties	--	++	---	+++		---	+++
Flooding of commercial premises			---	+++		---	+++
Flooding of agricultural land							
Flooding of recreational amenity/access	-	+	-	+	?-		
Designated habitat (NHA, SAC, & SPA)							
Landscape		-		-	-		
Cultural heritage				?-	?-		
Construction Phase							
Riverine habitat and species				--	--		
Traffic		-		-	-		-
Recreational amenity access		-		-	-		
Noise		-		-	-		-
Visual disturbance		-		-	-		-

Key

+++	Major beneficial impact			-	Minor adverse impact		
++	Moderate beneficial impact		No/neutral impact	--	Moderate adverse impact	?	Potential (avoidable) impact
+	Minor beneficial impact			---	Major adverse impact		

19 CONCLUSIONS & RECOMMENDATIONS

Conclusions

The following main conclusions can be drawn from the DCFPP:

1. Dublin City Council and Fingal County Council have commissioned the Dublin Coastal Flooding Protection Project (DCFPP) to Royal Haskoning. The work that has been carried out between May 2003 and April 2005 has resulted in this Final Report.

2. The DCFPP has been initiated in direct response to the extreme tide and flood event that was experienced across Dublin City and Fingal County during the 1st February 2002 and forms a major constituent of Dublin City's work on the SAFER project as well as Dublin Flooding Initiative.

3. The primary sponsoring authority for the DCFPP is Dublin City Council. Furthermore Fingal County Council (FCC), the Department of Communications, Marine and Natural Resources (DCMNR) and the Office of Public Works (OPW).

4. The main objectives of the study were:
   - undertake a strategic examination of the risk to Dublin from coastal flooding;
   - identify appropriate strategies and policies to combat and manage risk;
   - identify short term urgent works on experience gained from the February 2002 event;
   - identify medium to long term options to reduce and / or manage risk;
   - learn from the past.

5. On 1st February 2002 an exceptionally high tide occurred in Dublin, which resulted in significant flooding throughout parts of the city and in Fingal County. It resulted in the highest water level ever recorded to date of + 2.95 m ODM. The main factors contributing to this event were a high spring tide as well as a significant surge. Rainfall and river flows were not significant contributing factors. A thorough analysis of the event was carried out.

6. An extensive data collection exercise was undertaken as part of the study and the data collected catalogued for future use. Data collection among others has been aimed at information on existing coastal defences, water levels, wave and meteorological data, topographical and bathymetrical surveys and historical reports of relevance. The data collection process continued throughout the project and is currently on going.

7. Regarding the existing coastal defences an asset condition survey was carried out collecting relevant data through site inspections, classifying the areas around the coastline in discrete defence units, entering and storage of recorded data into a database and preparation of a manual to facilitate use of the database. This database has been delivered to both DCC and FCC for their use.

8. To engage with the population of Dublin City and Fingal County in a positive and productive manner a public information campaign was carried out. The main elements included a leaflet survey, a web-site and a public information campaign at

various locations within the project area. With the completion of much of the project, significant steps in this respect can be further taken and a number of public information days are currently in planing. Furthermore workshops, stakeholder and focus group meetings were carried out.

9. A detailed analysis of mean sea level was carried out using actual historic data as well as a review of the latest international best practice. Based on this it was recommended that an annual average sea level rise in all designs of 4.15 mm/year be adopted to the end of this century. This includes an allowance of 0.3 mm/year for land subsidence.

10. To understand the significance of the February 2002 event and to provide a basis for preparing designs for flood defence, an analysis of tide records from Dublin Port has been carried out. Factors that were investigated were historical development of astronomical tide and of mean sea level, the occurrence of seiches and the main factors that result in surges. Part of the analyses focussed on the joint probability of extreme tides and surges. Furthermore the effect of river discharges on water levels (and joint probability) has also been investigated.
    It was concluded that the February 2002 event was an extreme event having a return period in excess of approximately 60 years.

    It also concluded that an additional unpredictable parameter requiring further investigation is the mechanism of seiches, which are noted to be a regular feature in Dublin Bay.

11. The DCFPP considered the interaction between fluvial and tidal components to assess the level of flooding and to arrive at design conditions. For this purpose various numerical models have been developed. These models are:
    - SWAN: modelling of wave conditions, representing wave propagation in time and space;
    - FINEL2D: tidal model, reproducing tidal conditions in two dimension across the project area;
    - ZWENDL: river model, 1D hydraulic model for river calculations;
    - AMAZON: overtopping model, 1D model which calculates overtopping discharges and volumes.

    Furthermore use is made of the UK Met Office Forecast Models for the Irish Sea. This model provides 36 hour forecasts of water levels twice a day (at 2 am and at 2 pm) and provides the main boundary condition on the sea side. The other models have been converted to simplified models (matrices), which transfer these offshore conditions to nearshore conditions and eventually to conditions along the coastline.

12. A major deliverable of DCFPP is the tidal flooding forecasting system which will allow advance warning of potential flooding. The system comprises several elements, each of which contributes to the final forecast along the coastline. These are:
    - Input from the UK Met Office Shelf Seas and Storm Surge models;
    - Wave transformation matrices;
    - Overtopping matrices;
    - Water level prediction matrices;
    - Tidal water levels from Alexander Pier Lighthouse tide gauge.

    The output is given at 27 warning points along the coastline.

13. Within the flood forecast system, it is proposed to have a four-tier warning approach, with each tier getting progressively more serious: flood watch, Flood Warning A, Flood Warning B and Severe Flood Warning. Each of these levels is triggered using criteria for water levels and overtopping (rates and volumes). Initially default values will be used for all warning points. Through on site experience during future events and through detailed analysis these values will then be fine tuned.

14. The Flood Forecasting System will be installed on a server at the offices of Met Éireann. A second server will be provided at Dublin City Council to also allow access to the warning system by DCC. Apart from providing a user's manual, the staff of Met Éireann and DCC involved with the early warning system will be trained to work with it.

15. The flood hazard along the coastline has been investigated within the DCFPP. This included an assessment of a 200 year event and an indication of the standard of protection offered by the existing defences. In follow on to this, indicative extents of flooded areas for an event with a return period of 200 years in the present day as well as in 2051 were also determined. These calculations have been carried out to provide basic estimates of flood damage for economic assessment and to justify risk reduction works proposed. Further elaboration of this aspect of flood risk will take place within the SAFER project. The flood risk assessment concluded that much of the Dublin and Fingal coastline falls below an acceptable level of protection which the project has concluded should be a 200 year event or greater. Should an event of this order of magnitude occur today significant areas of Dublin and Fingal would be as risk and considerable economic damage would result.

16. An overview of relevant policies applicable to Dublin has been provided. This overview has been given, benchmarking these against international practices. The various aspects of flood management policy have been categorised:
    - Category 0: where there is no defined policy or documentation;
    - Category 1: where there is some form of policy, but not documented as such;
    - Category 2: where there is some form of documented policy;
    - Category 3: where there is comprehensive policy that is fully documented.

    Based on this, subsequently recommendations have been given regarding the various policy items (e.g. on climate change, flood warning systems, flood defence measures, operation and maintenance), indicating the advised scope, the advised "owner" and the advised "vehicle" for progressing matters.

17. Based on the current standard of protection, the current condition of the defences (visual inspection only), the flood hazard analysis and the design conditions (i.e. event with a 200 year return period), design options were given for the various stretches along the coastline and tidal reaches of the three rivers. The designs have furthermore been based on a design life up to 2031. A distinction has been made between urgent works as well as works for the medium to long term. For all options cost estimates have been given. The total cost of works to bring the defences up to an acceptable 200 year standard is estimated at €12m for FCC and €50m for DCC, or a total of €62m (low scenario). The maximum scenario is €93m. Furthermore the total costs of implementing a full Early Warning System is estimated at €2m. These costs do not include the cost of design fee, further site

investigation, structural inspection or environmental works, land acquisition, way leaves, but does include VAT.

18. An environmental review has been undertaken within the DCFPP. The review provides a summary of the baseline environment for all relevant areas and describes the potential issues and impacts associated with the various flood defence options. Furthermore recommendations have been made whether or not an EIA will be required. It is anticipated that overall the proposed options will have minor adverse impacts, however some options are likely to require a full EIA.

19. In Table 19.1 an overview is given of the project area, the current standards of protection, the current condition of the defences, the potential benefits that may be derived from implementing flood defence options, the options themselves, the costs involved, the main environmental impacts and a recommended priority for implementing the options.

20. Regarding the options, the following can be concluded:
    - the options proposed for Fingal can generally be characterised as having a medium rated priority for undertaking the proposed works; the main consideration behind these are the reasonable height and condition of the existing defences as well as the limited benefits that can be achieved from implementing the works;
    - the options for Dublin City Council can, with the exception of a few sections, generally be characterised as having a high priority; the main considerations behind these are the high benefits that can be derived from implementing the works as well as the beneficial benefit – cost ratios;
    - Conspicuous sections in this respect are: Merion Gates (high risk, extremely beneficial benefit – cost ratio), Clontarf (high risk / major road, beneficial benefit – cost ratio) and the River Dodder and mid and lower sections of the River Liffey (relatively high risk, reasonably beneficial benefit – cost ratio).

Recommendations

The DCFPP has resulted in the following main recommendations:

1. Regarding the models used:
    - Investigation into the mechanics of seiches should be carried out, to improve prediction of seiche effects.
    - The accuracy of the surge prediction of the UK Met Office Shelf Seas model with respect to mean sea level should be investigated further.
    - River modelling was undertaken only for the lower reaches of the Tolka, Liffey and Dodder. Furthermore calibration was difficult for extreme fluvial events and initial findings suggest that significant risk could exist on both the Dodder and the Liffey for such extreme events. It is therefore recommended to model the complete systems (Liffey and Dodder in particular) in order to obtain a better understanding of the influence of the rivers on (fluvial) flood situations.

2. Regarding the Flood Forecasting System:
    - The forecasting system initially will be installed with default values for the triggers used to issue warnings. It is recommended that these are fine-tuned

over a period of approximately 1 year to allow appropriate actions as a result of the forecasts.

- The flood forecasting system currently focuses on coastal flooding. An integrated approach is suggested, combining fluvial as well as tidal forecasting and warning. In order to optimize the development of such an integrated warning system, a separate proposal has been made to provide (i) a framework for a fluvial warning system (main elements, criteria related to accuracy and reliability), (ii) required steps for the development of such a system and (iii) a pilot case study.
- Furthermore integration of contingency planning and the forecasting system is required, such that it is clear who is responsible, what phases can be distinguished, which actions need to be taken and when, and how can these actions be monitored effectively, thus providing a full Early Warning and Response System.

3. Regarding the design options for the various flood defences:
   - The options have been detailed to a preliminary design level. Further detailing is required in order to allow for design optimisation. It is however important to support this with additional site investigations, these include: geotechnical, structural and environmental investigations.
   - The designs proposed are generally straight forward. Although attention has been given to aesthetics, it is suggested to give considerable attention to this. Not only with respect to the individual stretches, but also to the combined plans that will be undertaken.
   - Design life to 2031 in terms of flood defence has been considered. As a consequence economic justification has also been considered to this time scale. It has been seen that significant additional economic benefit is available if the design life were extended to say 2051. This should be considered against the minimal extra capital costs which would be required to bring the defence up to this design life.

4. Regarding Flood Risk

   Whilst every effort has been made to obtain reliable level and flood path information for the flood risk assessment process, a number of locations require further data collection and assessment due to access restrictions and other issues. The areas include:
   - Small stream at northern end of Baldoyle Estuary flowing under Portmarnock Bridge presents a potential flood path. Level information relevant to this stream was not available and further investigation is required to substantiate the risk and options proposed for this area.
   - North Howth defences contain and depend on the existing dune system along that frontage. The flood risk has been evaluated based on the current level and extent of the dunes. It does not attempt to investigate or quantify the geomorphological aspects of the frontage and the effect of the potential reduction or loss of these dunes to flood protection, except to indicate that it would be of significant importance. It is therefore recommended that a detailed coastal evolution and geomorphologic study be undertaken to evaluate these issues and recommend appropriate long term coastal protection and hence flood defence options.
   - ESB Poolbeg Power Station defence should be confirmed. Access restrictions at the time of the survey prevented detailed level information being obtained

along the Liffey Frontage. The levels should therefore be checked and the flood risk reviewed. Furthermore issues with respect to the water cooling system and its potential restriction during extreme events could result in the need to shut down the power station. This should be investigated further.

- Access restrictions on the left bank of the River Dodder prevented detailed defence levels being obtained in places upstream of New Bridge. These levels should be confirmed.
- Access restriction on the left and right banks of the River Liffey upstream of Sean Hueston Bridge prevent detailed defence levels being obtained in some locations. These should be checked.

5. Regarding the Liffey and Dodder

   The River Dodder and to a lesser extent the River Liffey require substantial measures to deal with coastal dominated flooding. Fluvial flooding however is expected to be a major consideration. In defining measures it is therefore suggested to have an integrated approach, addressing coastal as well as fluvial flood risks. Furthermore it is suggested to investigate all measures dealing with flood alleviation in a phased approach: from a spatial planning point of view (what are the most important restrictions, what are opportunities), from an operational point of view (which operational measures can (or could) alleviate the situation) and finally from an engineering point of view (types of defences).

   Regarding the first step: a tool that may prove valuable in this phase is the so-called water-opportunity-mapping, an approach developed in the Netherlands that has been successfully used internationally.

6. Regarding North Howth

   Whilst North Howth along Burrow Road is not significantly at risk from flooding at present, this is mainly due to the presence of the existing dunes. These dunes are however showing signs of considerable erosion, particularly after storms. It is therefore recommended that a detailed coastal evolution and geomorphological study be undertaken to evaluate the coastal erosion issues in this area and consideration given to appropriate long term coast protection and flood defence options. In the short term a programme of sand dune management works should be implemented and managed over a period of a number of years to minimise current erosion impacts. These measures, through monitoring and management, would help justify or not the need for a more detailed investigation depending on the benefit they bring to the area.

7. Coastal database

   DCC currently has the DCFPP Coastal Database. It is strongly recommended the most vulnerable locations are inspected and that the database be used on a regular basis. Not only will this allow timely taking of measures, but only through monitoring in time will thorough insight be obtained in the condition of the coastal defences.

8. Tide and River Guages

   It is recommended that new tide and river gauges are installed around the coastline and within the rivers to aid future modelling and analysis work and for the benefit of future EWS developments.

9. Remaining activities of the DCFPP

With the completion of this report most of the work has been completed and delivered to DCC. The remaining activities concern holding public information sessions, final upgrade of the website and carry out the study tour.

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Chapter 19

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