Flood protection and marine power in the Wash Estuary, United Kingdom

Technical and economical feasibility study

MSc Thesis

Delft University of Technology
Faculty of Civil Engineering and Geosciences
Department of Hydraulic Engineering
In association with
Royal Haskoning Rotterdam
Department Coastal & Rivers

Bram Hofschreuder
Delft, May 23th 2012

Courtesy cover photo: John Watson.

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ROYAL HASKONING

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PREFACE

Writing a MSc-thesis is the final part of the fifth year curriculum of my Master of Science study in Civil Engineering at Delft University of Technology. While attending the course water power engineering I found out how interesting this part of the work field is. When I was searching for a suitable subject for my thesis, combining both coastal engineering and water power engineering, I came in contact with Mr Mooyaart from Royal Haskoning Rotterdam. As it turned out Mr Mooyaart had a very interesting subject for my thesis: performing a technical and economical feasibility study regarding the combination of a storm surge barrier and a tidal power plant in the Wash estuary at the east coast of the United Kingdom. This report represents the end result of my research and work done over the past months, which has been a very interesting journey that was very challenging at times.

I would like to use this opportunity to thank the members of my graduation committee, Professor Vrijling, Mr Molenaar, Mr Labeur and Mr Mooyaart for their guidance, advice and much valued contributions. Also many thanks to Peter Dawe CEO of the Wash Tidal Barrier Corporation plc for sharing his documentation and Feddo Vollema from BMT ARGOSS for providing the wave and wind data. Last but not least I would like to thank my parents and friends for their great support during the past months.

Delft, May 15th 2012.

Bram Hofschreuder

SUMMARY

The Wash estuary is situated at the English east coast, making up the border between the counties Lincolnshire and Norfolk. Covering an area of approximately 615 km^2, the Wash estuary is among the largest estuaries in the United Kingdom. The estuary is characterized by deep channels alternating with large intertidal sandbanks and mudflats, while the coastline for a large part is fringed by extensive salt marshes. The relatively large tidal range results in a very dynamic environment, because of which the estuary is one of Europe's most valuable nature areas.

Adjacent to the Wash estuary the Fenlands are situated, a low lying area covering almost 3900 km^2. Centuries of large scale land reclamations created this largely engineered landscape. The Fenlands have a low population density and consist of mostly agricultural land. Both the intertidal sand and mud flats *and* the extensive salt marshes play an important role in the flood defence system as they dissipate much wave energy before it reaches the primary flood defences. As a result most primary flood defences along the Wash estuary's shoreline consist of relatively low earthen embankments, see the figure below.

On the left hand side of the embankment: the Fenlands; on the right hand side: salt marsh of the Wash estuary. (Courtesy: *snowgoostrust.org*)

Within the last hundred years several coastal flood disasters and near flood disasters have occurred in the area. According to the UK Government the flood protection level of the hinterland is adequate, however in view of the predicted climate changes part of the local population disagrees. The Wash Tidal Barrier Corporation has proposed to build a privately funded storm surge barrier across the Wash estuary. For the project to be profitable a tidal power plant is to be included.

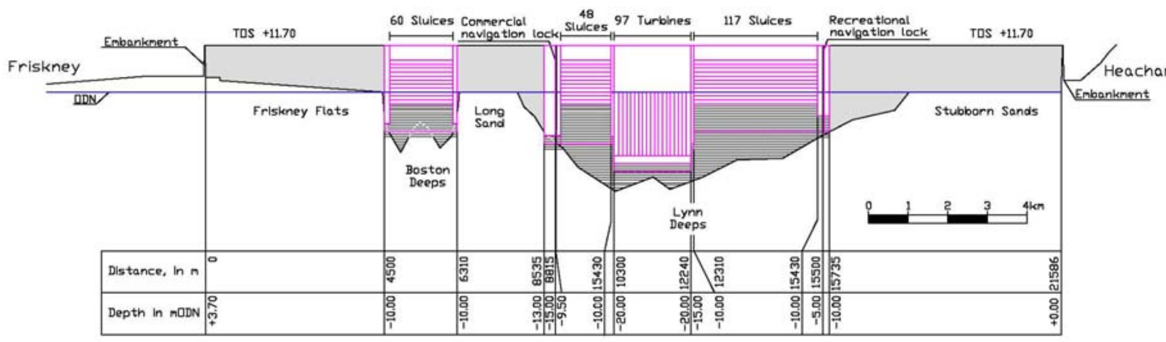
This thesis focuses on establishing the technical and economical feasibility of constructing the proposed combination of a storm surge barrier and a tidal power plant. The following research question is central to this study: *"To what extent is it possible and attractive to combine the closure of the Wash estuary with the generation of renewable energy from the tides?"* The following approach has been followed; by means of a desk study an analysis is made of the Wash estuary and adjacent Fenlands, thus identifying constraints and limitations within the study area. Next the hydrodynamic conditions are determined as they are of major importance for the design of the storm surge barrier and tidal power plant. An analysis of the UK's energy market is performed to establish in which price range the cost of the generated electricity have to lie in order to be competitive with conventional and other renewable energy sources.

At the same time a literature study is performed in order to gain insight into the different types of techniques available for the generation of energy from the tide, the expected energy yield and the available barrier, turbine and sluice types. The results of the performed analysis lead to conceptual designs of both components, that in turn are used to determine the technical

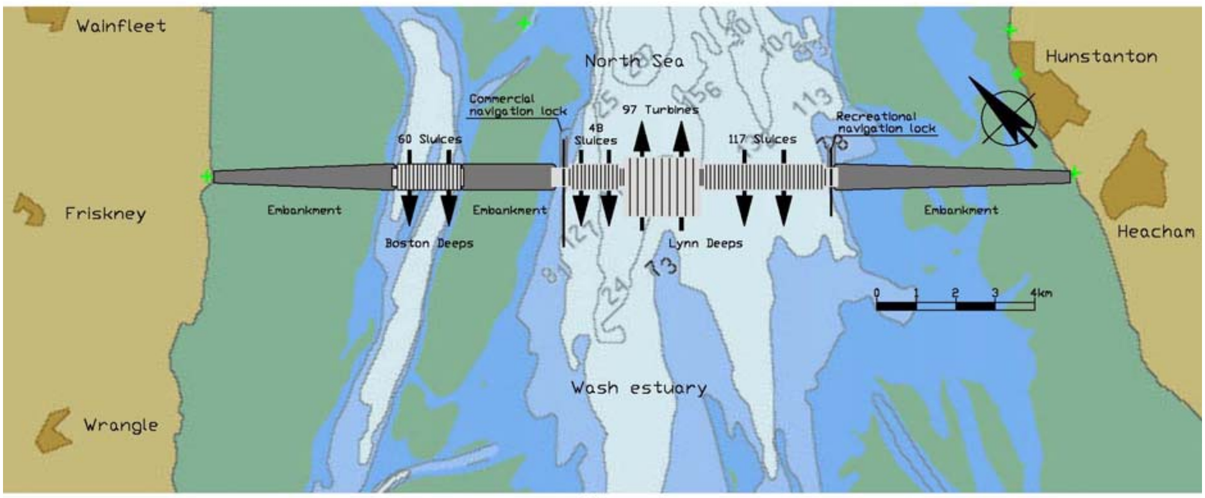
feasibility of both the tidal power plant and the storm surge barrier. After which both structures are integrated and the design is optimized, leading to the final conceptual design.

The performed analysis learned that the tidal currents present in the estuary are not strong enough for the generation of tidal stream power. Tidal range power is according to literature considered to be feasible in case the mean tidal range exceeds 5 m, with 4.70 m the mean tidal range in the Wash estuary is just below that limit. However due to the large area of intertidal flats it may be possible to design a economically feasible tidal power plant. The assessment of the UK's energy market learned that the costs of the electricity generated by the tidal power plant should lie within a price range of 8-11p/kWh in order to be competitive to other low-carbon energy resources.

After careful consideration, an ebb generation scheme, equipped with 97 bulb turbines and 225 sluices turned out to be the most optimal design, see the figure below. The tidal power plant has an installed power of 940 MW and an estimated annual energy yield of 2945 GWh. The turbines have a diameter of 8 m.



Elevation



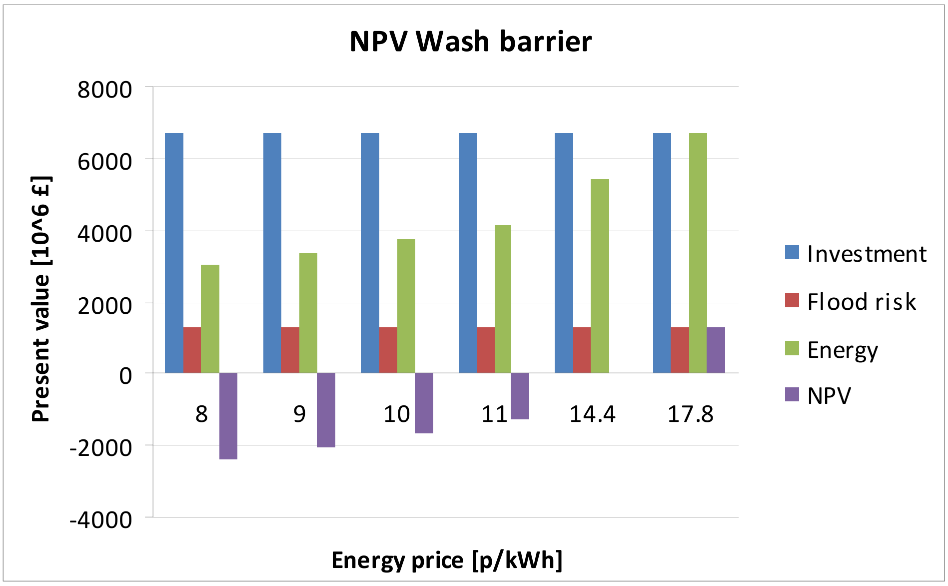
Plan view

Final conceptual design of the Wash estuary storm surge barrier and tidal power plant.

The barrier line running roughly from Friskney to Heacham is found to be the most suited alternative, as the deepest parts of the Lynn Deeps are avoided. As a result of which both the construction costs and structures footprint are smallest. The barrier section consists of an embankment dam crossing the salt marshes and intertidal flats, while the Boston and Lynn Deeps are crossed using caissons. A permanent barrier was preferred over a combination of a permanent and movable barrier, because the latter turned out to be very costly and also due to the fact that a movable storm surge barrier is intended for low frequency use, while the operational requirements of a tidal power plant prescribe a high frequency use.

The total investment costs before taxes of the combined structure are estimated to be £ 6.88 billion. The economic appraisal of the combined storm surge barrier and tidal power plant is based on the comparison of the discounted values of all expenses and revenues, resulting in the Net Present Value of the investment. The expenses consist of the total investment costs, the costs of the required refurbishments and the maintenance and operation costs during the design lifetime of 120 years. The revenues resulting from the generation of energy and enhancing the current flood protection level from 1:50/1:200 too 1:500. Based on a Net Present Value of zero, meaning that expenses and revenues are exactly balanced, the break-even energy price is computed.

Including the revenues from reducing the flood risk of the hinterland (this assumption is only valid in case the UK Government participates in the project) the break-even energy price amounts to 14.4 p/kWh. Regarding the project as a solely privately funded investment, the break-even energy price is computed to be 17.8 p/kWh.



Net Present value Wash barrier including revenues from flood risk reduction.

The main conclusions are that technically it is feasible to construct a tidal power plant within the Wash estuary. Also the construction of a storm surge barrier across the mouth of the estuary is technically possible. However raising the existing embankments results in the same flood protection level for less costs, therefore the construction of a storm surge barrier is not necessary to guarantee the safety against flooding. Since the computed break-even energy prices are both larger than 11 p/kwh, the project is considered to be not economical feasible.

In the performed analysis possible financial incentives from both the UK Government and European Committee, such as: carbon pricing, the buy-out price and Feed-in-Tariffs are not included. The reason for this is the fact that the revenues largely depend on market operation, which is a complex and continuously changing system. The mapping and quantifying of all the influences is recommended, as to include these effects in a future study. It is also recommended to study the impact of the structure on the morphology within the estuary and the adjacent Norfolk and Lincolnshire shorelines. Furthermore it is recommended to perform an environmental impact study, a geotechnical survey and to develop a hydraulic model to determine the relation between water level and wetted area *and* to check the design criteria. Finally it is recommended to execute both the structural design of all civil works and the design of the electromechanical equipment.

TABLE OF CONTENTS

LIST OF FIGURES

Page

LIST OF TABLES

LIST OF SYMBOLS AND ABBREVIATIONS

Symbol		Description	Unit
Greek symbols			
α	:	shape parameter Weibull distribution	[-]
α_n	:	phase angle of constituent number n	[rad]
β	:	location parameter Weibull distribution	[-]
δ	:	declination	[°]
ε	:	relative compression	[-]
		fraction of tidal cycle during which TPP is operational	[-]
γ	:	scaling parameter Weibull distribution	[-]
γ_b	:	influence factor for a berm	[-]
γ_β	:	influence factor for oblique wave attack	[-]
γ_f	:	influence factor for surface roughness on a slope	[-]
γ_v	:	influence factor for a vertical wall on top of a levee	[-]
γ_w	:	volumetric weight of sea water	[kN/m^3]
η	:	plant efficiency	[-]
$\eta(t)$	:	measured tidal level with reference to ordnance level	[m]
ξ	:	breaker parameter	[-]
π	:	number; representing a value of 3.14	[-]
ρ	:	volumetric density of water	[kg/m^3]
σ	:	new effective stress	[-]
σ_I	:	initial effective stress	[-]
τ	:	relaxation time	[s]
χ	:	loss coefficient	[-]
ω_n	:	angular velocity of constituent number n	[rad/s]

		Description	Unit

Roman symbols

Lower case

a	:	slope of the regression line	[-]
a_0	:	mean water level	[m]
a_n	:	amplitude of constituent number n	[m]
$a_{E\text{-}M}$	:	gravitational acceleration of the centre of the earth in the earth-moon system	[m/s^2]
$\Delta a_{E\text{-}M}$	:	differential pull of the moon in 1 kg of mass on the near side of earth	[m/s^2]
$a_{E\text{-}S}$	:	gravitational acceleration of the centre of the earth in the earth-sun system	[m/s^2]
$\Delta a_{E\text{-}S}$	:	differential pull of the sun in 1 kg of mass on the near side of earth	[m/s^2]
b	:	y-intercept regression line	[-]
c	:	wave celerity	[m/s]
c_f	:	friction coefficient	[-]
d	:	water depth	[m]
		average water depth	[m]
		layer thickness	[m]
$\tilde{d}$	:	dimensionless water depth	[-]
d_b	:	vertical difference between middle of berm and SWL	[m]
g	:	gravitational acceleration	[m/s^2]
h	:	water level	[m]
		water depth	[m]
h_b	:	basin water level	[m]
		average water level in the basin	[m + sluice floor]
h_{new}	:	retaining height of the new barrier	[m]
h_{NS}	:	water level on North Sea	[m]
		average water level on the North Sea	[m + sluice floor]
Δh	:	increase in water level	[m]
m	:	mass	[kg]
		discharge coefficient	[-]

		Description	Unit
n	:	economic lifetime of the power plant	[yr]
		number of years (from investment year)	[-]
		operating speed	[rpm]
n_i	:	number of observations in a directional bin	[-]
n_q	:	specific speed	[rpm]
n_{sl}	:	number of sluices	[-]
n_t	:	number of turbines	[-]
q	:	average overtopping discharge	[m^3/s/m]
r	:	distance between masses	[m]
		discount rate	[-]
		real interest rate	[-]
r_b	:	normalized width of the berm	[-]
r_{db}	:	normalized difference between SWL and the middle of the berm	[-]
r_m	:	distance between centres of earth and moon	[m]
r_s	:	distance between centres of earth and sun	[m]
t	:	Time	[s]
		time period considered	[day]
t_1	:	reference time period	[day]
$t_{closure}$	:	duration of the closure	[s]
t_{storm}	:	storm duration	[hr]
$\tan\alpha$	:	slope	[-]
u_{10}	:	wind speed at 10 m	[m/s]
u_d	:	flow velocity	[m/s]
x	:	distance	[m]

Capitals

$\overline{A}$	:	average basin area	[m^2]
A_b	:	basin area	[m^2]
A_{gate}	:	area new gate	[m^2]

		Description	Unit
A_i	:	area of fill levee section i	[m^2]
A_{ref}	:	area existing gate	[m^2]
A_s	:	current-carrying cross-section	[m^2]
A_t	:	throat area	[m^2]
B	:	storage width	[m]
		berm width	[m]
		width of sluice opening	[m]
		beam of a ship	[m]
$B_{chamber}$	:	width of chamber	[m]
BEP	:	energy price corresponding to the break-even point	[£/kWh]
C	:	Celsius	[-]
C_2	:	Constant, approximately $4{\cdot}10^{-6}$	[-]
C_{2006}	:	price level in 2006	[£/m^3]
C_{2012}	:	price level in 2012	[£/m^3]
C_E	:	monetary value of the generated energy in year n	[£]
C_{ex}	:	monetary value of the expenses in year *n*	[£]
C_{gate}	:	cost sluice or turbine gate	[£]
$C_{gate,t}$	:	cost vertical lift gate	[£]
$C_{gate,sl}$	:	cost tainter gate	[£]
C_{gates}	:	total cost tainter and vertical lift gates	[£]
$C_{land\ based}$	:	cost land based section barrier line	[£]
C_{new}	:	cost new barrier design	[£]
C_p	:	primary compression coefficient	[-]
C_{ref}	:	cost existing gate	[£]
C_s	:	secondary compression coefficient	[-]
C_s	:	cost electromechanical equipment	[£]
C_{TPP}	:	factor expressing the mode of operation of a TPP	[-]

		Description	Unit
D	:	turbine diameter	[m]
		draught of a ship	[m]
		total damage	[£]
$D_{chamber}$	:	depth of chamber	[m]
E_{annum}	:	annual energy yield	[GWh]
E_P	:	annual energy potential of the basin	[kWh]
E_t	:	net electricity generation in year t	[MWh]
F	:	force	[N]
		form factor	[-]
		fetch	[m]
$\widetilde{F}$	:	dimensionless fetch	[-]
$F_{E\text{-}M}$	:	gravitational pull of the moon on 1kg of mass on earth	[N]
F_{index}	:	factor representing indexation of cost	[-]
F_t	:	variable operating and maintenance costs in year t	[£]
G	:	gravitational constant	[Nm^2/kg^2]
$\widetilde{H}$	:	dimensionless significant wave height	[-]
H_{av}	:	average head per tidal cycle	[m]
$H_{max,\,new}$	:	maximum head over the new barrier	[m]
H_{m0}	:	significant wave height	[m]
H_r	:	rated head	[m]
H_s	:	significant wave height	[m]
$H_{s,mouth}$	:	significant wave height at the mouth of the estuary	[m]
$H_{s,saltmarsh}$	:	significant wave height above the salt marshes	[m]
$H_{s,storm}$	:	average significant wave height during a storm event	[m]
$I_{building}$	:	characteristic value for adapting adjacent buildings	[£/m^2]
I_{fill}	:	characteristic value fill material	[£/m^3]
I_{ref}	:	characteristic value reference design	[£/m^3]
$I_{sheet\ pile}$	:	characteristic value for lengthening a sheet pile wall	[£/m^2]

		Description	Unit
I_0	:	initial investment costs	[£]
I_t	:	capital cost in year t	[£]
$I_{quay\ wall}$	:	characteristic value for raising a quay wall	[£/m^2]
K_1	:	principal lunar-solar diurnal tidal constituent	[-]
		amplitude of principal lunar-solar diurnal tidal constituent	[m]
L	:	wave length	[m]
		length	[m]
L_{berm}	:	characteristic berm length	[m]
$L_{chamber}$	:	length of chamber	[m]
L_i	:	length of levee section i	[m]
L_{new}	:	length of the new barrier	[m]
LOA	:	length overall of a ship	[m]
M_E	:	mass on earth	[kg]
M_M	:	mass of the moon	[kg]
M_S	:	mass of the sun	[kg]
M_t	:	fixed operation and maintenance costs in year t	[£]
M_2	:	principal lunar semidiurnal tidal constituent	[-]
		amplitude of principal lunar semidiurnal tidal constituent	[m]
M_4	:	wave speed induced higher harmonic of the M_2-tidal constituent	[-]
M_6	:	bottom friction induced higher harmonic of the M_2-tidal constituent	[-]
N	:	Northern Hemisphere	[-]
		total number of observations	[-]
		design life time storm surge barrier	[-]
N_{hr}	:	number of hours per year	[hr]
N_{M2}	:	number of half tidal cycle per day	[-]
N_s	:	number of seconds per day	[s]
		number of storms	[-]
N_{tide}	:	number of energy generating tides per year	[-]
NPV	:	net present value of the investment	[£]

		Description	Unit
O_1	:	principal lunar diurnal tidal constituent	[-]
		amplitude of principal lunar diurnal tidal constituent	[m]
P	:	tidal prism	[m^3]
		power generated per tidal cycle	[W] / [GW]
P_{annum}	:	power generated per year	[GW]
P_d	:	power per turbine	[MW]
P_{fu}	:	future failure probability	[1/yr]
P_{new}	:	installed power new tidal power plant	[kW]
P_{pd}	:	present day failure probability	[1/yr]
P_{rated}	:	rated power per turbine	[MW]
PV_{ex}	:	summation of the discounted values of the expenses	[£]
$PV_{refv,\ E,\ 1p/kWh}$	:	summation of the discounted values of the energy revenues for a energy price of 1p/kWh	[£]
$PV_{rev,\ SoP}$	:	summation of the discounted values of the revenues from raising the SoP	[£]
Q	:	discharge	[m^3/s]
		probability of exceedance of a single event	[-]
Q_{av}	:	average discharge per tidal cycle	[m^3/s]
Q_d	:	design discharge	[m^3/s]
$Q_{d,idle}$	:	average idle discharge through turbine	[m^3/s]
$Q_{d,sl}$	:	average discharge through sluice	[m^3/s]
Q_p	:	combined peak discharge from all rivers	[m^3/s]
Q_s	:	rated discharge	[m^3/s]
Q_s	:	probability of exceedance of H_s in a storm per year	[-]
R	:	hydraulic radius	[m]
$\overline{R}$	:	mean tidal range	[m]
R_E	:	radius of the earth	[m]
R_C	:	crest height above design SWL	[m]
R_{mean}	:	mean tidal range	[m]

		Description	Unit
S	:	Southern Hemisphere	[-]
		total wind set-up	[m]
S_2	:	principal solar semidiurnal tidal constituent	[-]
		amplitude of principal solar semidiurnal tidal constituent	[m]
T	:	wave period	[s]
$\widetilde{T}$	:	dimensionless peak wave period	[-]
T_p	:	peak wave period	[s]
T_{tide}	:	duration of the tidal cycle	[s]
U	:	wind velocity at 10 m	[m/s]
		degree of consolidation	[-]
U_s	:	extreme wind speed	[m/s]
V_{av}	:	average volume of water per tidal cycle	[m^3]
W	:	Weibull reduced variable	[-]
ΔZ	:	settlement during lifetime	[m]

Other symbols

°	:	degrees
€	:	Euro
£	:	Pound Sterling
$	:	US dolar
%	:	percentage

Abbreviations

AD	:	Anno Domini
AAD	:	Estimated Annual Average Damages
AONB	:	Area of Outstanding Natural Beauty
BEP	:	Break Even Point
BC	:	Before Christ
CD	:	chart datum; water level corresponding to the lowest astronomical tide
CCGT	:	Combined Cycle Gas Turbine
CCS	:	Carbon Capture and Storage
CFMP	:	Catchment Flood Management Plan
CO_2	:	carbon dioxide
DECC	:	Department of Energy and Climate Change
DEFRA	:	Department of Environment, Food and Rural Affairs
EMS	:	European Marine Site
EU	:	European Union
EU ETS	:	European Union Emissions Trading System
FIT	:	Feed-in-Tariff
GT	:	Conventional Gas Turbine
GW	:	Giga Watt (10^9)
GWh	:	Giga Watt hour (10^9)
HM	:	Her Majesty
NPV	:	Net Present Value
kW	:	kilo Watt (10^3)
kWh	:	kilo Watt hour (10^3)
LOA	:	length overall
LEC	:	levelised electricity generation costs
LNR	:	Local Nature Reserve
MSL	:	Mean Sea Level
MW	:	Mega Watt (10^6)

MWh	:	Mega Watt hour (10^6)
NATO	:	North Atlantic Treaty Organisation
NNR	:	National Nature Reserves
NPV	:	Net Present Value
ODN	:	Ordnance Level Newlyn
Ofgem	:	Office for Gas and Electricity Markets
PV	:	Present Value
RAF	:	Royal Air Force
RSPB	:	Royal Society for the Protection of Birds
SMP	:	Shoreline Management Plan
SMP2	:	The Wash Shoreline Management Plan 2
R&D	:	Research and Development
RO	:	Renewables Obligation
ROC's	:	Renewables Obligation Certificates
SAC	:	Special Areas of Conservation
SAR	:	Synthetic Aperture Radar
SLR	:	Sea Level Rise
SoP	:	Standard of Protection
SPA	:	Special Area of Interest
SSSI	:	Sites of Special Scientific Interest
TPP	:	Tidal Power Plant
UK	:	United Kingdom
UKCP09	:	UK Climate Projections 2009
US	:	United States
USACE	:	US Army Corps of Engineers
V.A.T	:	Value Added Tax
WOII	:	2nd World War

INTRODUCTION

The Wash estuary is situated at the English east coast, making up the border between the counties Lincolnshire and Norfolk. Covering an area of approximately 615 km^2, the Wash estuary is among the largest estuaries in the United Kingdom. The estuary is for the most part very shallow and is characterized by deep channels alternating with large intertidal sandbanks and mudflats, while the coastline for a large part is fringed by extensive salt marshes. Several rivers discharge into the estuary, together with a large tidal range and much wave exposure near the mouth of the estuary this results in a very dynamic environment. Due to this large range of habitats present within the Wash estuary it is one of Europe's most valuable nature areas, comparable to the Wadden Sea coast along the Dutch, German and Danish coastlines.

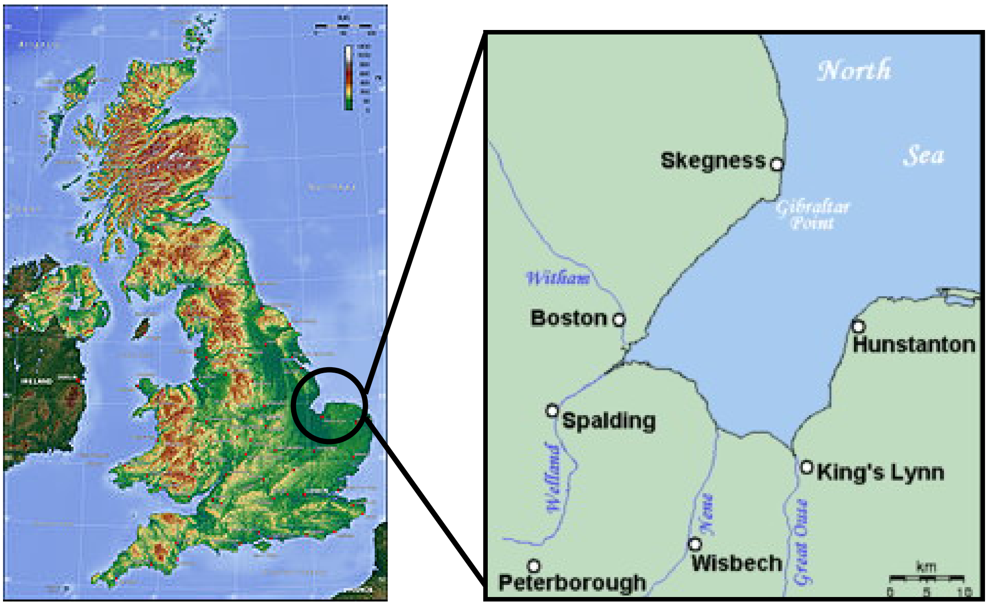


Figure 1: location of the Wash estuary at the English coast.

Adjacent to the Wash estuary the Fenlands are situated, a low lying area covering almost 3900 km^2. Centuries of large scale land reclamations, for agricultural purposes mainly, created this largely engineered landscape, resembling a Dutch polder landscape. Nowadays the Fenlands still are a region of mainly agricultural importance. The region contains approximately half of the grade 1 agricultural land in the United Kingdom.

Within the last hundred years several coastal flood disasters and near flood disasters have occurred in the area. Both the intertidal sand and mud flats *and* the extensive saltmarshes play an important role in the flood defence system as they dissipate much wave energy before it reaches the primary flood defences. The saltmarshes are responsible for the largest dissipation of wave energy. As a result most primary flood defences along the Wash estuary shoreline consist of relatively low earthen embankments covered with grass. Revetments are protecting stretches of shoreline near Skegness and between Hunstanton and Heacham. While the only natural flood defences, apart from the saltmarshes obviously, are restricted to the dunes and sand spits forming the shoreline near Gibraltar point, the cliffs near Hunstanton and a shingle[1]

[1] Very coarse gravel.

ridge between Wolferton Creek and Hunstanton. However the latter is backed by stretches of grass covered embankments and revetments.

According to the UK Government the current standard of protection (SoP) is sufficiently guaranteed, within the boundaries of the performed economic assessment. The policy is aimed at sustaining the present level of flood defence, including the effects of climate change. This is primarily done by means of managed realignment, which simply means that the primary flood defence is set back and land is left undefended. The thought behind this measure is that saltmarshes will develop on the land, thus increasing the flood defence level. This policy is possible because the land bordering the Wash estuary has a low population density and consists mainly of agricultural land. In build-up areas managed realignment is of course not an option, so here the current defence line is to be held on its existing alignment.

Since the Government policy, according to at least part of the local population, does not provide an adequate level of flood protection, the Wash Tidal Barrier Corporation plc was founded by Peter Dawe. This corporation has proposed to build a privately funded storm surge barrier across the Wash estuary that is also used for the generation of marine energy, thus combining flood defence with the generation of electricity using the tide as a renewable energy source. However the political response to date has been principally one of disinterest. The key to this disinterest is the statutory protections for various habitats in the Wash estuary and Fenland area. Hence the Royal Society for the Protection of Birds (RSPB) and other nature protection organizations condemned the building of a barrier as soon as the project was launched. Politicians, it appears, are reluctant to challenge these organisations[2].

Aim of the feasibility study

This thesis focuses on establishing the technical and economical feasibility of constructing the proposed combination of a storm surge barrier and a tidal power plant. The following research question is central to this study:

"To what extent is it possible and attractive to combine the closure of the Wash estuary with the generation of renewable energy from the tides?"

In order to be able to conclude whether a storm surge barrier in combination with a tidal power plant is feasible or not several other questions need answering, these questions are arranged according to the main subjects of this thesis.

General aspects:

- What constraints and limitations are present in the Wash and Fenland area?
- What stakeholders can be identified?
- What are the hydrodynamic boundary conditions?
- What is the current Standard of Protection and what is the Government's policy on flood protection in the Wash and Fenland area?
- What is the most suitable location for a storm surge barrier/tidal barrage?

[2] Source: The politics of Adaptation, Peter Dawe, July 2010.

Tidal power plant:

- What techniques are available for generating energy using the tide and what form of marine energy production has the most potential within the Wash estuary?
- What turbines are most suitable and what is the optimum number?
- What sluices are most suitable and what is the optimum number?
- What are the general dimensions of the storm surge barrier?
- What are the costs per kWh for tidal power and what are the costs per kWh for conventional energy in the UK?

Storm surge barrier:

- Is closure of the Wash estuary necessary to ensure the flood safety of the hinterland?
- What types of closure measures are possible and what type of closure is most suitable for the Wash estuary?
- What are the general dimensions of the storm surge barrier?

The purpose of this thesis is, after careful consideration of the different aspects, to achieve a comprehensive and broad-based final conceptual design of the storm surge barrier in combination with a tidal power plant.

Approach and methodology

By means of a desk study an analysis is made of the Wash estuary and adjacent Fenlands, thus identifying constraints and limitations within the study area. Also the stakeholders and their interests are mapped. Next the hydrodynamic conditions are determined as they are of major importance for the design of the storm surge barrier and tidal power plant.

An analysis of the UK's energy market is performed to establish whether the electricity costs per kWh for the tidal power plant are competitive compared to the costs per kWh of conventional electricity. Because the UK Government, together with many other European Governments, has decided to promote wind energy as the renewable energy source with the most potential, the energy yield and estimated costs per kWh are compared to those of tidal energy.

At the same time a literature study is performed in order to gain insight into the different types of techniques available for the generation of energy from the tide, the expected energy yield based on rules of thumb and the available barrier, turbine and sluice types. The main goal is to identify the potential, the weaknesses and strengths of each solution. The same is done for the storm surge barrier.

The results of the performed analysis lead to the statement of the Terms of Reference that is used to establish the conceptual designs that in turn are used to determine the technical feasibility of both the tidal power plant and the storm surge barrier. After which both structures are integrated and the design is optimized, leading to the final conceptual design. The several optimization rounds form the basis for the establishment of the economic feasibility of the proposed combination of a storm surge barrier and a tidal power plant.

The final conceptual design will be the result of a balanced and weighted choice regarding the barrier type, sluices and turbines needed to provide accurate flood defence and at the same time result in an efficient tidal power plant. The methodology followed is clarified in the flow chart depicted in figure 2.

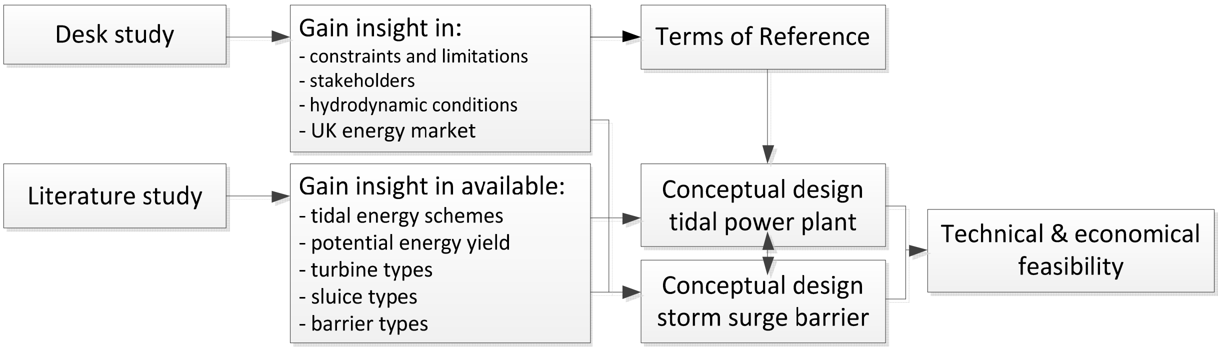


Figure 2: methodology.

Overview of contents

In the first chapter the results of the performed analysis are presented. Starting with a short description of the historical developments in the Wash basin, which is followed by a concise description of the present day land use, the current coastal defences and flood management policies. This information is then used to identify restraints and limitations within the study area, as are the different stakeholders and their interests. The second chapter describes the hydrodynamic conditions present within the Wash estuary and the intertidal part of the rivers discharging into the estuary. Within the third chapter the results of the performed assessment regarding the UK's energy market are shown. Followed in chapter four by the Terms of Reference for the project.

In chapter five the technical feasibility of a storm surge barrier is determined, while in chapter six the same is done with respect to the tidal power plant. Chapter seven will deal with both the integration of both structures and the optimization of the combined structure. Leading in chapter eight to the economical feasibility assessment of this combined structure. The final chapter will state the conclusions and recommendations.

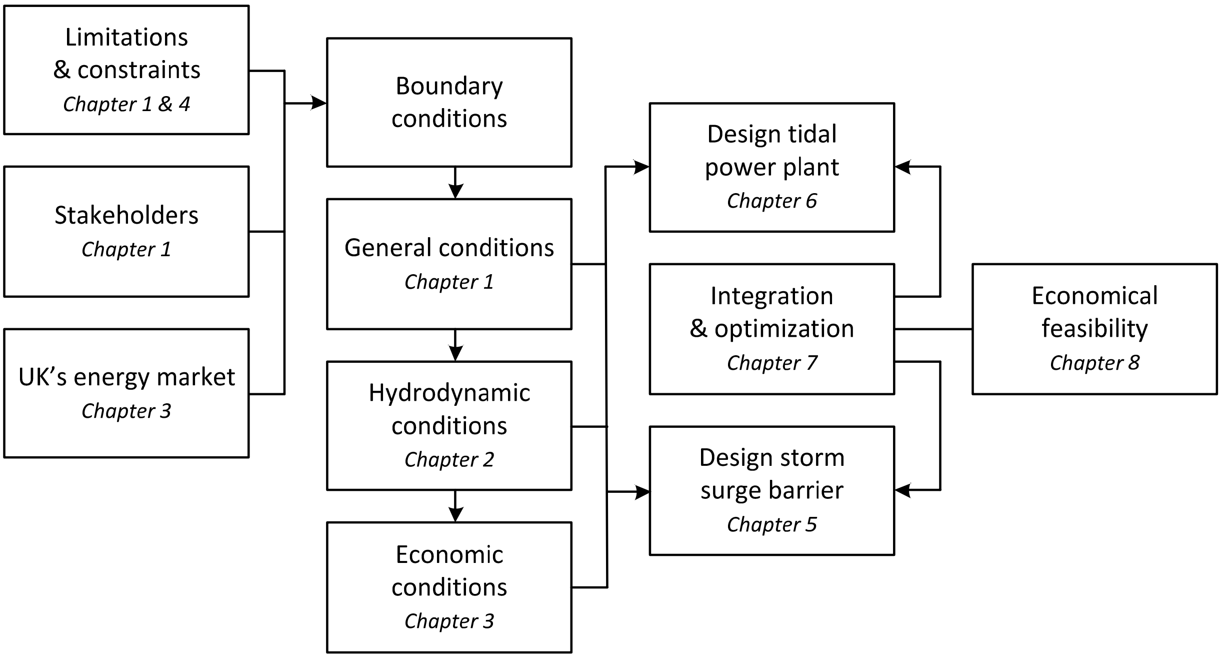
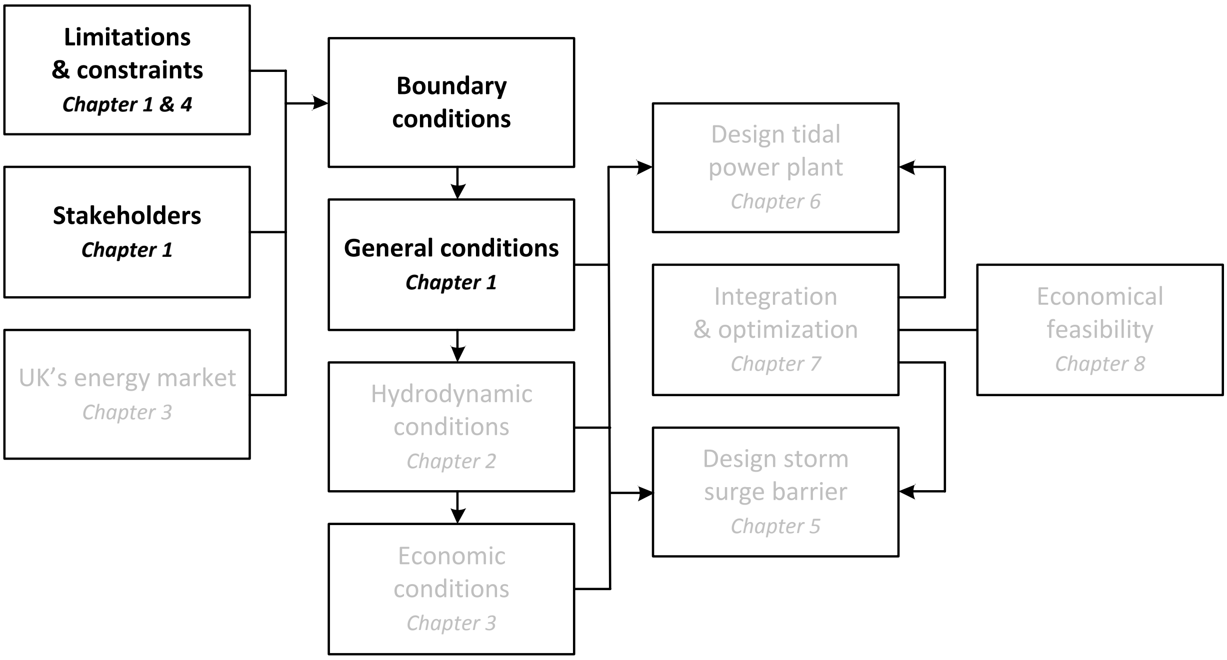


Figure 3: overview of contents.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

1 THE WASH ESTUARY AND FENLAND AREA

In this chapter a short introduction to the Wash estuary is given. First the geological, geomorhological and manmade changes over the centuries are described to get some insight in the origin of the present day situation and corresponding challenges. Next the present day land use is described in order to map the stakeholders present in the region and their interests. Followed by a description of the present day coastline, the current flood defences bordering the Wash estuary and the current UK flood defence policy.

1.1 Short history

The coastline of the Wash estuary has altered considerably in time. On the one hand due to sediment deposits from the rivers and North Sea and on the other hand due to centuries of land reclamations which resulted in the Fenlands[3]. The Fenlands are depicted as the yellowish green parts in figure 4. In this section a short description is given concerning the geological development of the study area, followed by the history of land reclamation and drainage works in the Wash and Fenland area. Finally the development of the settlements in the area is reviewed.

Figure 4: Wash estuary and Fenlands.

1.1.1 Geology and geomorphology

During the Jurassic period mud stones were formed in the Wash and Fenland region, on which in a later stage Cretaceous chalk was deposited. Erosion of the softer mud stone deposits resulted in a large clay valley stretching form the Humber valley to Cambridgeshire [The Wash SMP2, appendix C, 2010].

The current coastline of the Wash estuary[4] was formed during the Pleistocene and Holocene epochs. The ice flowing southwards up the slope of the modern day coastline during the Anglian[5], Wolstonian[6] and Devensian[7] glacial periods carved out tunnel valleys like the deep

[3] A fen is a local name for an individual area of marshland or former marshland. It is fed by mineral-rich surface water or groundwater. Fens are characterised by their water chemistry, which is alkaline with relatively high dissolved mineral levels but with few other plant nutrients. Source: Wikipedia.
[4] This section is based on information found on Wikipedia.
[5] The name used on the British Isles for the 2[rd] major glacial period that in Northern Europe is called Elsterian.

narrow valleys of the Silver pit (north-south directed long valley in the sea bed) and Wash estuary, see figure 5, and also widened and deepened the estuary's embayment. During these ice ages large quantities of sands and gravels were deposited.

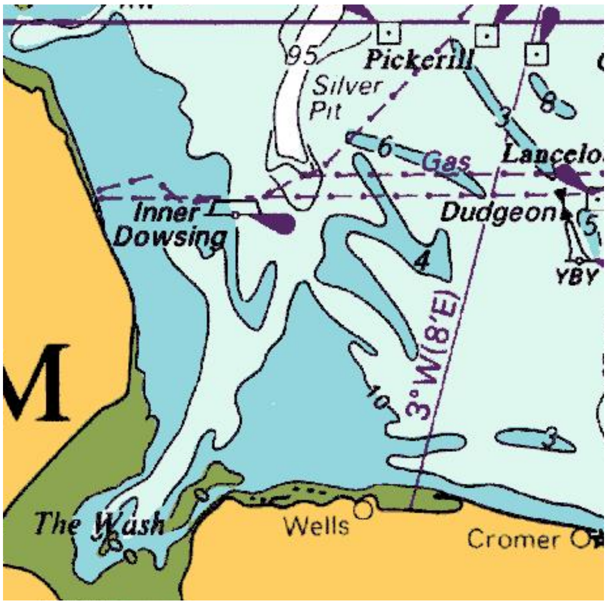


Figure 5: Wash and Silver pit tunnel valleys.

In the Ipswichian[8] interglacial stage and at the end of the Devensian glacial stage the rivers Welland, Witham, Nene and Great Ouse were tributaries of a large river called the Wash River (At these stages the sea level remained lower than it is today). The Wash river kept the Silver pit tunnel valley free of periglacial sediment. During this period large volumes of gravel were deposited in the catchment areas of the Wash river's tributaries.

During the Holocene epoch the sea level gradually rose and flooded the tunnel valley and seems to have it kept open by tidal action. However the Wash River basin was gradually filled with sediments (mainly sand and gravel) from the rivers and the North Sea, thus forming the Wash estuary. Hence in the present day situation the Wash River does not exist anymore physically.

Nowadays the Wash estuary is a tide dominated estuary with a prograding coast. Within the estuary the river outfalls have formed tide dominated deltas. These inter tidal river deltas resemble small estuaries as a result of their embayed setting fringed by salt marshes and mud flats, wile the different channels are separated by intertidal sand flats. The sediment within the estuary consist mainly of sand and gravel, see appendix 1.

1.1.2 Land reclamation during the ages

Over the past centuries parts of the Wash estuary bordering the main land and rivers were reclaimed on a large scale. This process started in the Roman period with the construction of an earthen sea wall protecting the hinterland against flooding. These Roman embankments

[6] The name used on the British Isles for the 3rd major glacial period that in Northern Europe is called Saalien.
[7] The name used on the British Isles for the 4th major glacial period (Weichselian in Northern Europe).
[8] The name used on the British Isles for the 3rd interglacial period (Eemian in Northern Europe).

connected the higher grounds, furthermore they constructed the first drainage canals being the *Car dyke* and the *Foss dyke*. Tough there were some medieval drainage works, large scale land reclamation and drainage works started again around the English Civil War (1642-1651). The drainage of the marshes in the Fenlands was organized by levels, each of which including several parishes and the corresponding parts of the Fenlands, see figure 6.

The Great Level or Bedford Level is the largest level and covers approximately 1300 km^2. The fourth Earl of Bedford, Francis Russell, was one of the leaders of the reclamation syndicates in the Fenland region, commissioned by King Charles I. The investors were financing the reclamation and drainage works and were rewarded large parts of the resulting farmland. The first phase started in the 1630s, but during the civil war the works were destroyed. After the civil war the second phase started in the 1650s under leadership of the Dutch engineer Cornelius Vermuyden[9], who again reclaimed the area.

Figure 6: Fenlands bordering the Wash estuary.

Lord Lindsey and his partner Sir William Killigrew reclaimed and drained the Lindsey Level by 1638. However the works were destroyed during the civil war and remained marshland until in 1765 the reclamation started again.

The first part of the 120 km^2 large Deepening fen was reclaimed and drained in 1637 by the fourth Earl of Bedford. The draining of the fen was addressed again in 1664 under the Earl of Manchester, but they did not succeed. Not until 1730 a new attempt was made that was completed in 1774, however the drainage system would not be efficient until1827.

The Witham Commission Fens, also known as east and west Wildmore Fens, are located north of Boston, extending as far as Skegness. Reclamation and drainage works started in the 11^{th} century, but were not very successful, until the 17^{th} century. The main reasons were lack of maintenance, disasters and destruction. However in 1642 the sluices were destroyed and the land was flooded again. Repairs were made and drainage of the land started again.

Besides reclaiming land by draining marshes in the Fenlands also land was claimed directly from the sea by building dikes on the salt marshes fringing the Wash estuary. Although some attempts were made in the 6^{th} century, large scale land reclamation started in the 13^{th} century

9 Sint-Maartsensdijk, 1590 – London, October 1677.

and continued on a smaller scale the late 1970's / early 1980's. In appendix 2 an overview is given of the progress of the land reclamations directly bordering the Wash estuary during the past centuries.

However the success of the drainage works in the 17th and 18th century was short lived as the drainage of water resulted in shrinkage and oxidation of the peat, thus causing settlements of the reclaimed land. The more effective the drainage works the larger the problems became as the land subsided to a level lower than the surrounding rivers and the land close to the estuary, thus preventing the discharge of the drainage water. In the 18th century one tried to solve this problem by using windmills, later replaced by powerful steam engines. These were in turn replaced by diesel powered pumping stations, after WOII followed by electric powered pumping stations. Only in 1962 some drainage works were constructed, according to the original plans drawn up by Vermuyden (these were not build by the clients due to financial reasons), that improved the drainage of the area.

Figure 7: windmill at Wicken Fen.

Figure 8: steam engine at Stretham.

Figure 9: pumping station at Prickwillow.

Due to modern drainage in the 19th and 20th century the Fenlands transformed into very fertile farmland. A negative side effect of the continuous pumping is that at present day many parts of the Fenlands lie below mean sea level and will continue to subside. This explains why the land slopes down from the coastline into Cambridge, were the oldest polders are situated.

1.1.3 Settlement development

In the area evidence was found of Neolithic, Bronze age and Iron Age settlements. These settlements were scattered over the region and many of them are nowadays covered with sediments as the sea level rose the past centuries. During Roman time, there were some small villages located in the area, mainly cattle farmers and salt winning industry. However the main settlements were situated in the vicinity of the military route north to Brancaster and Holme.

Historically the early post-Roman settlements were located on the Fen islands[10], Fen edges[11] and the Townlands[12]. An example of a Fen island is the Isle of Ely, on which the highest point is 39 m above mean sea level. On this "island" the cathedral city of Ely was built, which is located roughly halfway between King's Lynn and Cambridge. Regarding the Townlands the situation was remarkably the other way around. Until the peat began to shrink as a result of the drainage works these lands were lower than the surrounding Fenlands. However the more stable and fertile silt soils were reclaimed by medieval farmers and embanked to protect them against flooding from both the sea and the Fenlands. All the settlements on the Townlands were laid out as elongated strips in order to provide access to the Fenlands, salt marches and sea. The same holds for the settlements on the Fen edges, thus providing access to both Fenland and upland. Examples of Townlands settlements are Swineshead, Wisbech, Spalding and Boston. With the drainage and reclamation small farmsteads started to appear outside of the Townlands. Most buildings in the open, inland Fenlands are post 1750.

The land in northwest Norfolk has always been above mean sea level and thus higher than the Fenlands. Hence the settlement pattern is different compared to the pattern in the Fenlands. Here the building pattern is defined by vast estates and a few larger settlements.

1.2 Present day land use

In the following sections a short description is given of the different types of land uses that are present in the study area. The purpose is to map the different interests in the study area in order to assess in a later stage the impact of the proposed measures.

1.2.1 Agriculture

The major land use in the area surrounding the Wash estuary is agriculture, the Fenlands area contains approximately 50% of the grade 1 agricultural land and circa 10% of the grade 2 agricultural land in England [The Wash SMP2, 2010]. According to the Agricultural Land Classification system in England and Wales, grade 1 refers to excellent quality agricultural lands and grade 2 to very good quality agricultural lands.

The area is therefore very important for the food supply in the United Kingdom. In appendix D of the Wash Shoreline Management Plan it is stated that the farming sector and related businesses have an annual turnover of £2.5 billion and employ 45,000 people. The main crops are vegetables, potatoes, sugar beets and grains, but also cash crops as bulbs, flowers and rapeseed are grown.

Figure 10: tulips on grade 1agricultural land.
(Courtesy: *WESG / Alan Lambert*)

[10] Fen islands are areas of higher land which were never covered by the growing peat and remained dry when the Fenlands around them were flooded.
[11] Fen edges are the uplands surrounding the Fenlands.
[12] The Townlands are an arch like broad bank of silt around the Wash estuary and form the remains of the river embankments that formed naturally during the Bronze and Iron ages.

1.2.2 Fishing industry

Another important economic resource within the Wash estuary is the commercial fishing industry, consisting mainly of shellfisheries. The shell fisheries are located on the intertidal sand and mud flats in the estuary. Besides the commercial fishing industry there is also a large tourism oriented sports fishing industry.

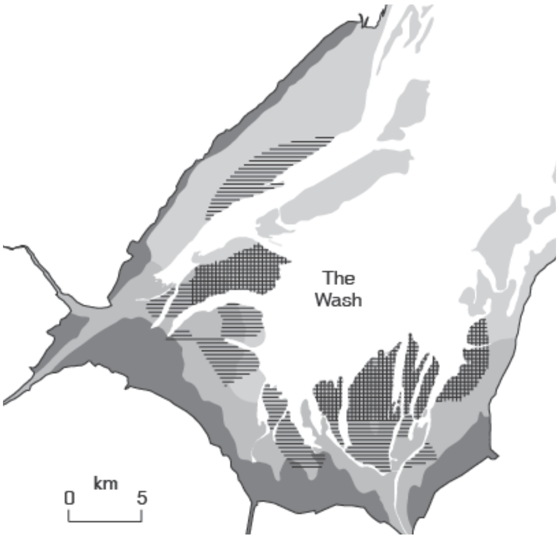


Figure 11: vulnerability of shellfish beds.
The cross-hatched areas are most at risk to destruction as a result of winter storms.
(Courtesy: *Dare, 2004*)

The mussel and cockle beds within the Wash estuary are vulnerable to periodic damage as a result of strong tidal currents during storm surges and wave action during severe northerly storms [Dare, 2004]. Appendix 3 shows the location of the important mussel and cockle beds through the years. Most mussel and cockle beds are located in the most landward half of the estuary and have retreated in landward direction during the years. Besides cultivating shellfish the estuary is an important fishing ground for brown and pink shrimp. Brown shrimp are caught in the channels between the inner channels, while pink shrimp is caught in deeper water (>10 m). Since the brown shrimp are caught in greater quantities, they are of greater commercial importance [Pawson, 2002].

1.2.3 Port activities

Historic ports are situated in Boston, King's Lynn, Wisbech, Fosdyke and Spalding, the latter port is nowadays known as Port Sutton Bridge. These ports form an important part of the local and regional economy. The commercial attractiveness of the ports is determined by the strategic position with respect to the easy access of the Midlands industrial areas and the sea, the fact that both coastal and river class vessels can be handled and the importance of the Wash estuary as a fishery resource. Besides the commercial port activities there is also much shipping traffic related to tourism, especially in spring and summer time. No ferries sail from the ports in the Wash estuary. Besides fishing and recreational vessels approximately 1800 commercial shipping movements take place within the estuary per annum [Wash Estuary Strategy Group, 2004]. In the table below an overview is given of the activities per port.

Port	Port activities		
	Commercial	Fishing	Leisure
Boston	X	X	
King's Lynn	X	X	
Wisbech	X		X
Sutton Bridge	X		
Fosdyke			X

Source: *www.ports.org.uk*

Table 1: port activities in the Wash estuary.

The commercial ports of Boston, Sutton Bridge, Wisbech and King's Lynn see much shipping traffic of container goods, dry bulk, liquid bulk, steel, stone, timber and the like. Boston and King's Lynn also still support fishing fleets that harvest shellfish and shrimp. The port of Fosdyke used to be a commercial port, but nowadays it is yacht harbour.

Figure 12: Wisbech yacht harbour. (Courtesy: *Fenland district Council*)

Figure 13: Port of Boston (Courtesy: *Port of Boston*).

1.2.4 Tourist industry

The tourism industry adjacent to the Wash estuary consists mainly of large coastal resorts and holiday parks, situated along the eastern shoreline of the estuary, from Wolferton Creek to Hunstanton, and along the Lincolnshire coastline north of Skegness. The beaches along these stretches of the shoreline are of European significance according to the EU bathing water directive.

The main attractions for the tourists are the beaches, sport fishing and water sports activities. Other attractions are the fantasy island theme park near Ingoldmells, the Hunstanton sea life sanctuary, Seahenge at Holme-next-the-Sea, several golf courses and off course the seaside promenades and historical city centres. Last but not least the area facilitates wildlife-related tourism, such as hiking and wildfowling. The tourist industry is also a very important source of income for the region.

1.2.5 Military activities

According to the Wash Shoreline Management plan the Royal Air Force (RAF) uses parts of the Wash estuary and the Fenlands as weapons training ranges. There used to be a range at Wainfleet, just off the coast of Gibraltar point. This range was closed in 2010 and the site was cleared by the Explosive Ordnance Disposal from the RAF, however unrecovered ordnance and unexploded ordnance will remain for many years.

The second weapons training range is located at Holbeach at the southern coastline of the Wash estuary, this target range is still in commission. Part of the target range is located on the Fenlands in the Gedney marshes, including the airbase, the control tower and the observation towers. The main part of the target range is situated on the mudflats and salt marshes of the estuary. The range provides several old vessels that are beached and are used as targets for bombing raids by the RAF and other NATO air forces.

Figure 14: F-15E aircraft over the range.
(Courtesy: *Ian Simons*)

Figure 15: beached vessels on the range.
(Courtesy: *Ian Simons*)

1.2.6 Protected nature areas

The Wash estuary includes extensive mudflats and sand areas, fringed by salt marshes. These habitats range from estuarine to fully marine conditions and from sheltered conditions in the inner parts of the estuary to wave exposed conditions at the North Sea coast. Therefore the Wash estuary is one of Europe's most valuable estuaries for wildlife, comparable to the Wadden Sea coast along the Dutch, German and Danish coastline, and home to one of the largest colonies of Common Seals in Europe. Furthermore the area is also important as a rare example of mature salt marshland. This type of marshland used to be common in Europe, but was mostly destroyed by reclamation schemes[13]. The importance of the estuaries nature is reflected in the long list of areas with a protected status. This list includes Ramsar sites, Natural Nature Reserves (NNR), Sites of Special Scientific Interest (SSSI's) and Natura 2000 sites that are subdivided into Special Protection Areas (SPA's) and Special Areas of Conservation (SAC's).

Ramsar sites are wetland sites of international importance that are protected by the governments that have ratified the Ramsar Convention treaty. The SSSI's are designated sites of nationally importance protected under national legislation[14]. SAC's protect habitats and species listed under the EU Habitats Directive and SPA's protect wild birds as set out under the EU Birds Directive. Both Directives also contain requirements regarding the protection of listed species (European Protected Species). In order to make matters more complicated intertidal and sub tidal SPA's and SAC's are collectively referred to as European Marine Sites (EMS's).

In the following table an overview is given of the areas protected under national and international legislation. From the table can be concluded that the complete area is protected and that the areas frequently overlap each other. It is therefore important to realise that every activity in the area will involve mitigating measures and that it is also very important to gain public support for a possible project, especially from the conservation organisations. In appendix 4 maps indicating the areas are included.

[13] Source: New Scientist, edition 16 April 1981, article "Marsh save – but at what price" by an unknown author.
[14] Source: Natural England.

Area	Status					
	Ramsar	NNR	SSSI	SPA	SAC	EMS
Wash estuary	X	X	X	X	X	X
North Norfolk Coast	X		X		X	X
Gibraltar Point	X	X	X	X	X	X
Hunstanton cliffs			X			
Holme dunes		X				

Source: *The Wash & North Norfolk Coast site plan, November 2010.*
NNR = National Nature reserve
SSSI = Sites of Special Scientific interest
SPA = Special Protection Area
SAC = Special Areas of Conservation
EMS = European Marine Site

Table 2: nature protection areas.

In the overview given above Local Nature Reserves (LNR's) are omitted, also omitted is the fact that part of the Wash estuary is designated within the national landscape as an Area of Outstanding Natural Beauty (AONB), see appendix 4. An AONB is a unique landscape of natural significance. The North Norfolk Coast has a beach barrier system that is unique in the UK, while within the Wash estuary the area comprises of a shingle ridge, mud and sand flats, old river arms and a series of inland saline lagoons.

On Seal Sand in the southeast corner of the Wash estuary lives the biggest single colony of Common Seals in Europe. The seals use the estuary as a breeding and haul out area, while they hunt in the North Sea. A conservation target set out by the Wash Estuary Strategy Group is to maintain and enhance the Common Seal population in the estuary, this policy is supported by the UK Government.

1.2.7 Historic environment

All past traces of human presence, such as remnants of the historic manmade landscape still visible today, historic buildings, archaeological sites and evidence of past environments are encompassed by the term historic environment.

Offshore there are submerged land surfaces bearing the evidence of early human habitation. Examples are the recently uncovered early Bronze Age monuments near Holme-next-the-Sea. The structures were originally constructed on a salt marsh that was covered by sediment as a result of sea level rise. Due to marine erosion processes the sites were uncovered again in 1998.

Across and within the sediments of the Fenlands lies a record of human history going back until Neolithic times. In addition to the buried remains, there is a variety of historic buildings that reflect the past activities and landscape. Also it is likely that many undiscovered historic wrecks are still preserved beneath the seabed of the Wash estuary [The Wash SMP2, 2010].

The surviving Roman, medieval and post-medieval sea and river flood defences present along the Lincolnshire coastline of the Wash estuary, are part of the few principal areas in England with surviving historic flood protection field monuments.

According to English Heritage numerous listed monumental buildings are present along the entire Wash estuary coastline, the same holds for WWII military structures and the remnants of ancient monuments such as medieval villages and surrounding fields, medieval and Roman salterns and the remnants of roman villa's, towers and forts present in the landscape[15]. But the largest concentrations are found on the historically higher grounds along the eastern shoreline of the Wash Estuary and just south of Skegness along the western shoreline.

Figure 16: outlines of saltern mounds at Marshchapel. (Courtesy: *English Heritage 2009*)

Figure 17: Seahenge I near Holme-next-the-Sea. (Courtesy: *circulostres.blogspot.com*)

1.2.8 Industry

There is no large scale industry present in the area. The industry present consists of agricultural related activities, such as food preparations plants and fruit and vegetables packaging industry.

1.2.9 Critical infrastructure

In the Fenlands surrounding the Wash estuary some primary road connections are present that may be partly flooded in case a breach occurs in the Wash primary flood defences. These roads are listed in the table below.

Main road	From	Via	To
A16	Spalding	Boston	Spilsby
A17	King's Lynn		Sleaford
A47	King's Lynn	Wisbech	Peterborough
A52	Grantham	Boston	Skegness
A149	A17 (King's Lynn)		Hunstanton
A151	Spalding		A17
A152	A52 (Donnington)	Gosberton	A16
A1101	Wisbech		A17
B1165	Sutton St James	Tydd St Giles	A1101
B1359	Sutton Bridge	Long Sutton	Chapelgate
B1390	Long Sutton		Sutton St James
B1397	Sutterton	Kirton	Boston

Table 3: primary roads surrounding the Wash estuary.

[15] Source: National Heritage List for England.

The same holds for part of the railroad stretches from Ely to King's Lynn and Sleaford via Boston to Skegness, see figure 18. The dotted lines represent bus connections.

Figure 18: rail road network surrounding the Wash estuary (Courtesy: *National Rail*).

Because this part of the UK has a low population density there are no airfields present, except for the Fenland Aero club near Spalding. Parts of both the national high pressure gas grid and the national electricity grid are situated near the southern end of the Wash basin, roughly following the A17 motorway route.

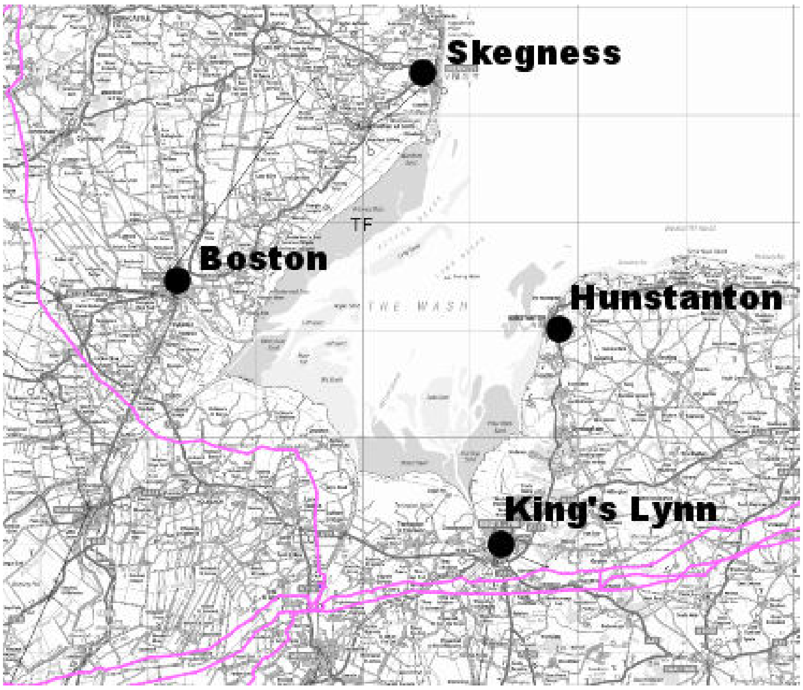


Figure 19: national gas transmission system (Courtesy: *National Grid*).

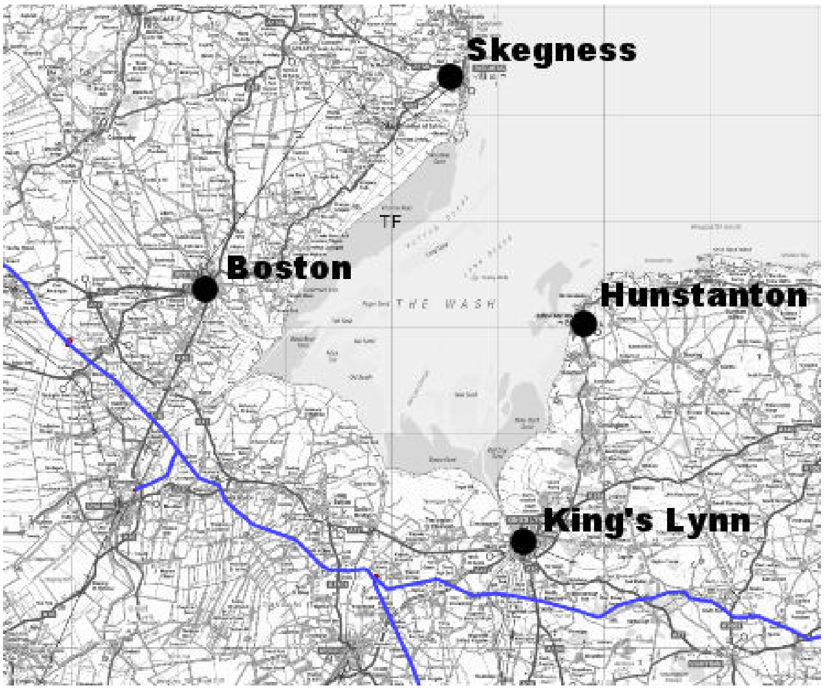


Figure 20: national electricity transmission system (Courtesy: *National Grid*).

1.2.10 Submarine utilities

In the current situation no (inter)national submarine cables and pipelines land within the Wash estuary. However there are numerous oil and gas platforms located in front of the Lincolnshire and Norfolk coastline, see figure 21. The shore connections of these platforms land on the main land either north or south of the Wash estuary.

As a consequence of climate change the development of, *and* demand for renewable energy took a rise in the recent past. This has led to plans to build several offshore wind farms[16] in front of the mouth of the Wash estuary, outside the European Marine Site, see figure 22. Also plans exist to connect the Lincs offshore wind farm with the main land via a submarine cable through the middle of the Wash estuary (figure 23)[17].

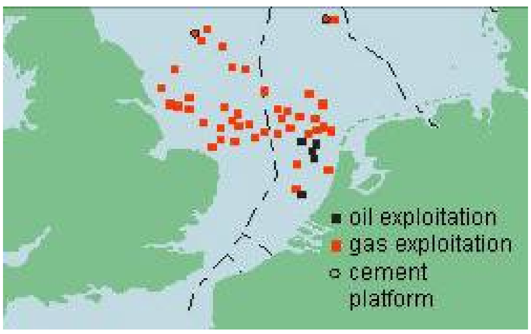


Figure 21: location of oil and gas platforms.

[16] Source: Review of reef effects of offshore wind farm structures and potential for enhancement and mitigation, Department for Business, Enterprise & Regulatory Reform, January 2008.
[17] Source: Lincs Offshore Wind Farm, Environmental Statement, Non-Technical support, Centrica energy January 2007.

Since the English Government has decided to currently focus on wind energy instead of the construction of tidal barrages in the large estuaries there is a large change that the cable will be installed and thus represents a boundary condition in case a barrage is to be constructed. For now it seems as if the lobby for wind energy has won the battle from the lobby advocating tidal energy schemes.

Figure 22: proposed wind farms in front of the Wash estuary. (Courtesy: *Centrica energy*)

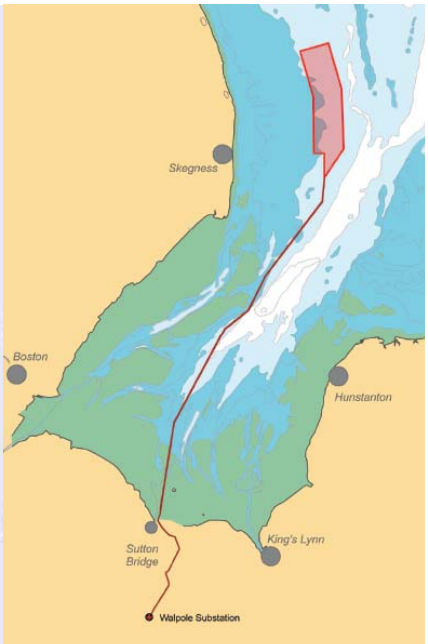


Figure 23: Lincs Offshore Wind farm. (Courtesy: *BERR*)

1.2.11 Stakeholder interests

Key stakeholders	Flood protection	Livelihood	Environmental values	Scenic values	Cultural values	Historical values	Water quality
Government organisations	+	n.a	+	+	+	+	+
The Wash Tidal Barrier Corporation	+	+	-	-	-	-	+
Inhabitants / land owners	+	+	+/-	+/-	+/-	+/-	+
Agricultural sector	+	+	-	+/-	+/-	+/-	+/-
Fishing industry	+	+	+	-	-	-	+
Ports & shipping traffic	+	+	-	-	-	-	-
Tourism sector	+	+	+	+	+	+	+
Nature conservation organisations	-	n.a.	+	+	n.a	n.a	+
Heritage and cultural organisations	+	n.a.	n.a	+	+	+	n.a.
Military	+	n.a.	n.a.	n.a.	n.a.	n.a.	n.a.

+ = important; +/- = neutral; - = unimportant; n.a. = not applicable

Table 4: stakeholder interests.

1.3 Current flood defence

This section starts with a description of the present day coastline bordering the Wash estuary and the adjacent stretches of the Norfolk and Lincolnshire coastlines. Secondly the current flood defences surrounding the Wash estuary are described. Finally the UK's current flood defence policy is reviewed.

1.3.1 Present day coastline

The present day coastlines are described using the *Encyclopedia of the World's Coastal Landforms* [Bird, 2010]. The intention is to give a rough indication in order to get some feeling with the different coastal systems present in the area.

The Lincolnshire coastline

The western and southern boundaries of the Wash estuary are part of the Lincolnshire coast, which is low-lying (including the hinterland). Figure 25 shows a map depicting this part of the estuarine coastline. The North Sea coast is mostly sandy, backed by dunes, while the coastline bordering the Wash estuary is characterized by salt marshes and mudflats, backed by an embankment protecting the Fenlands.

Moving from Saltfleet in southern direction the coastline is dominated by very wide intertidal muddy sand flats, fringed with a sandy beach that is backed by parallel dune ridges. The oldest dune ridges date from the 14th century and are covered with woodland and scrub, while the younger dunes are covered with marram grass. Towards Theddlethorpe the brim consisting of intertidal flats and sandy beach backed by dune ridges reduces in width, until near Mablethorpe only a narrow beach, consisting of sand and gravel, with many groins backed by a seawall remains. The coastline between Mablethorpe and Skegness remains artificial and has a long history of coastal regression. The only exception is the stretch of coast near Chapel St. Leonards were the narrow sandy and shingle beach is backed by dunes.

Figure 24: coastal dunes at Theddlethorpe (Courtesy: *Geostudies*).

Near Ingoldmell, just north of Skegness, a very dynamic pattern of coastal spits is present. The shape of the spits clearly indicates a governing alongshore sediment transport in southern direction into the Wash estuary. The sediment stems partly from the Humber estuary, situated just north of the Wash estuary, the North Sea and the coastline stretch between Mablethorpe and Skegness.

The sand and shingles beach widens in the direction of Skegness, were the seafront consists of a promenade fronted by sand and shingle beaches. South of Skegness, until Gibraltar Point the coastline is characterized by parallel dune ridges that are separated by swales[18].

[18] Swale is a low tract of land that is moist or marshy. (NL: duinvallei.)

These swales indicate spits that grew successively in front of each other on the intertidal flats. Again the spits point in southern direction. *Just south of Gibraltar Point a bay is situated were the mouth of Steeping River is located. The river mouth is deflected by the southward growth of the spit forming Gibraltar Point. The river flows between artificial banks until the intertidal zone is reached.*

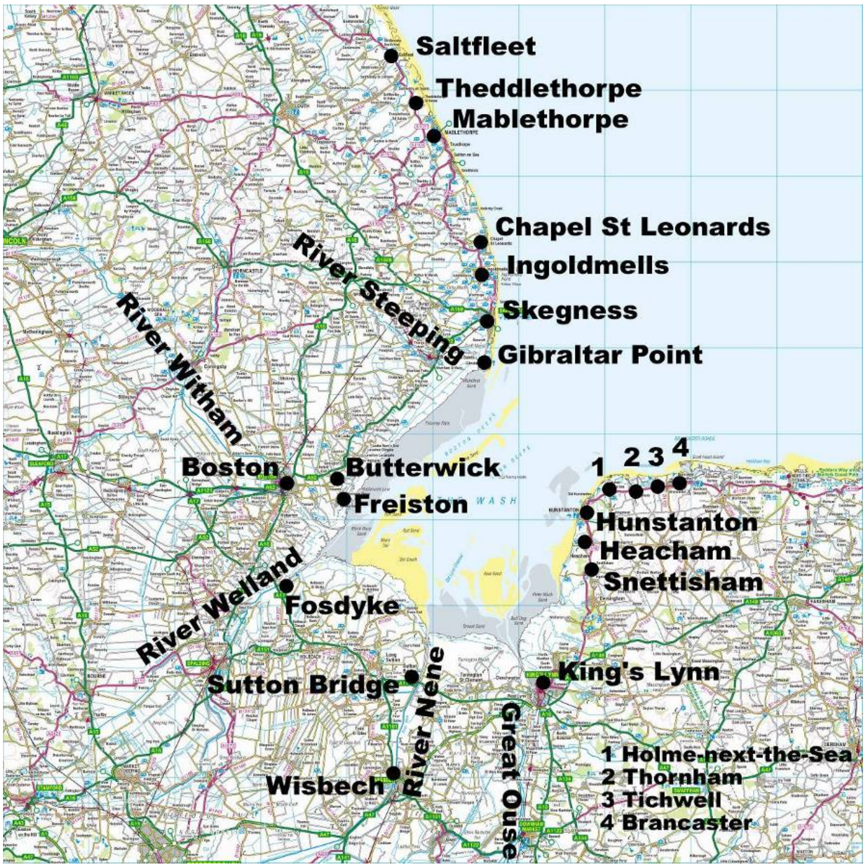


Figure 25: Lincolnshire and Norfolk coastline (Courtesy: *Ordnance Survey*).

South of the river mouth the coastline consists of salt marshes descending to intertidal mudflats, backed by embankments protecting the low-lying Fenlands. Here one can clearly recognize up to four embankments in inland directing, marking the historical stages in land reclamation works. In this section of the coast, from Gibraltar Point to Boston the belt with salt marshes is not as wide as on the southern shores of the Wash estuary. At the height of the small village of Freiston, near Butterwick, reclaimed marshland was abandoned in the 1990s, when gaps were cut in the embankments. Slowly the meadows are changing into salt marshes and mudflats. Cutting the embankments, thus creating Freiston shore, serves a dual purpose. On the one hand new nature area is created, while on the other hand the mudflats and salt marshes serve as sea defence.

In the southwest corner of the estuary, near Boston, the rivers Welland and Witham discharge into the Wash estuary. Both rivers are trained, even on the intertidal flats close to the coast. The whole southern shoreline of the estuary is characterized by low-lying Fenlands, protected by embankments that are on the sea side fringed with wide salt marshes that abruptly turn into mud flats via a small escarpment.

The Wash estuary it self is characterized by large intertidal mudflats alternating with large sand banks and deep channels like the Boston Deeps near the mouth of the rivers Welland and Witham and the Lynn Deeps, Cork Hole and Bull Dog channel near the mouth of the Great Ouse.

The Norfolk coastline

The eastern boundary of the Wash estuary is formed by a part of the Norfolk coastline, see figure 25 for a map. This part of the Wash estuary coastline is generally speaking low-lying, however the hinterland is situated above mean sea level, alternating with cliffs near Hunstanton.

Starting at Brancaster, following the coastline in western direction, one first encounters salt marshes backed by coastal dunes and the coastal plain. These salt marshes are protected by a large barrier island that consists of shingle[19] *ridges overlain by dunes, fronted by a sand and shingle beach.* As can be seen from the shape of the spits and barrier islands along this part of the coastline, the predominant direction of the alongshore sediment transport is directed towards the Wash estuary. *Further to the west near Titchwell and Thornham the coast in fringed with intertidal sand flats on which shingle ridges and dunes are formed, however it is a very dynamic environment. Were the ridges and dunes survive, mud flats develop in their lee. After which salt marshes are developing.*

At Holme-next-the-Sea a salt marsh developed on a sand flat that became enclosed by a shingle spit, again backed by coastal dunes and the coastal plain. From Holme-next-the-Sea in the direction of Hunstanton the ground rises to the Hunstanton plateau. Here the coastline consists of eroding vertical cliffs bordering a sandy shore with local reefs of Carstone[20]*, see figure 26.*

Following the coastline in southern direction towards Heacham the cliffs decline toward a bluff that diverges landward as a low escarpment in Chalk. The sand and gravel beach is fronted by large intertidal sand flats. In the direction of Snettisham a shingle beach develops, that turns into a sand spit that subsequently diminishes into salt marshes backed by embankments protecting the Fenlands and fronted by a widening intertidal area.

Figure 26: Hunstanton cliffs. White Chalk over Red Chalk and Carstone (Courtesy: *Geostudies*).

This intertidal area consists mainly of muddy sand flats. The character of the coastline remains the same until the mouth of the river Nene on the border with Lincolnshire. Just north of King's Lynn the Great Ouse flows into Wash estuary through the salt marshes. Both rivers are trained.

[19] Very coarse gravel.
[20] A type of sandstone that is orange when weathered and otherwise greenish-brown.

1.3.2 Coastal defences

In this section a rough overview is given regarding the current coastal defences present along the Wash estuary and the adjacent Lincolnshire and Norfolk coastlines.

The low-lying Fenlands bordering the Wash estuary along the southern and western sides are protected by earth embankments covered with grass. These so called sea banks form, together with the salt marshes and mudflats in front of them, the primary sea defences. As a result of the historical stages in land reclamation, at many locations remnants of earlier embankments lie behind the primary flood defences. However these defences do not have a formal flood protection function, but probably will reduce the consequences in case coastal flooding occurs.

The southeast shoreline between Wolferton Creek and Hunstanton (see figure 29) is protected by a managed shingle ridge on the shore face. This ridge is fronted by large sand and mudflats and backed up by a second line of defence, consisting of an earthen embankment with a grass cover. Some sections along this part of the coastline are protected by earth embankments and revetments. These defences also protect a low-lying area, however the area is much smaller compared to the southern and western boundaries since the hinterland reaches a level higher than mean sea level much quicker.

The reason that the shingle ridge is managed is due to the fact that the ridge has a natural tendency to move in landward direction as a result of the wave and wind action during a storm. This process is likely to speed up as a result of the sea level rise and increase in the expected number of severe storms and higher wave heights resulting from the climate change. Since there are several holiday parks situated along this stretch of the coast and because the direct hinterland lies below mean sea level it was apparently decided to keep the shingle ridge at its current location.

Figure 27: managed shingle ridge and beach near Snettisham. (Courtesy: *The Wash Shoreline Management Plan 2*)

The Hunstanton seaside is not prone to the risk of flooding beyond the promenade, but the shoreline is protected against erosion by a combination of sea walls, wave return walls and wooden and concrete groyne structures. Finally there are undefended sea cliffs at the north side of Hunstanton. These cliffs are composed of Carstone and chalk and are nowadays allowed to erode naturally, but there is evidence that in the past the cliffs were defended at their base. The main failure mechanism is undercutting as a result of erosion.

The adjacent Lincolnshire coastline from Saltfleet to Maplethorpe is protected predominantly by natural defences consisting of sand dunes, sandy beaches and intertidal sand flats. However some short stretches of the coastline are protected by sea walls. From Maplethrope to Skegness the sea defences consist of combination of a veneer beach[21] and engineered structures or armoured dunes [Scott Wilson, 2009]. Beach nourishments are carried out along

[21] Veneer beach = thin sand veneer overlying a glacial till foundation.

this stretch of the coastline (350,000 – 850,000 m^3 per year [Scott Wilson, appendix C, 2009]). The coastal defences near Gibraltar Point consist of dunes ridges and sand spits backed by earthen embankments.

The Norfolk coastline is defended by sand dunes from Old Hunstanton to Holme-next-the-Sea. The hinterland consists of high grounds. The remaining shoreline to Brancaster is defended by earthen embankments fronted by salt marshes and mud flats, near Brancaster sand dunes are present [Environment Agency SMP2, 2009].

In the table below the indicative length of the different types of coastal defences are listed.

Stretch of coastline	Type of defence	Length[1)] [km]
Saltfleet - Maplethorpe	Predominantly natural defences	9
Mapletorpe - Skegness	Sea walls and revetments	25
Gibraltar Point	Natural defences backed by an earthen embankment	4
Gibraltar Point – Wolferton Creek	Earthen embankments and Salt marshes	89
Wolferton Creek- Snettisham Scalp	Shingle ridge backed by an earthen embankment Sea wall	2.5 0.55
Snettisham Scalp - Heacham	Shingle ridge backed by an earthen embankment	3.5
Heacham - Hunstanton	Mixture of concrete sea walls and a shingle ridge backed by an earthen embankment	2.5
Hunstanton	Promenade	2
Hunstanton - Old Hunstanton	Cliffs	2
Old Hunstanton – Holme-next-the-Sea	Natural defences	4
Holme-next-the-Sea - Brancaster	Earthen embankments	8

[1)] The tabulated length includes the parts of the river that are part of the Shoreline Management Plan as these stretches are regarded coastal defences.
Note: the length of the coastal defences is set equal to the length of the coastline, in reality however the length of the coastal defences is bound to be longer. This assumption is made due to lack of available information.

Table 5: primary coastal flood defences.

Most of the current flood defences surrounding the Wash estuary are designed for a Standard of Protection (SoP) of 1:200, although several small stretches and hotspots have a SoP of 1:50. The majority of these defences is expected to fail somewhere in epoch 1 (present day–2025) and epoch 2 (2025-2055), without active intervention [Environment Agency SMP2, 2010]. The defences along the Lincolnshire coastline also have a SoP of 1:200 and are expected to fail in epoch 2 [Scott Wilson, 2009]. The SoP along the Norfolk coastline is most probably also 1:200, however no information regarding this subject was found in the Shoreline Management Plan 2. Without active intervention the current embankments are expected to fail in epoch 1 and 2. [Environment Agency SMP2, 2009]. In The Wash Shoreline Management Plan 2 [Environment Agency SMP2, appendix F, 2010], figures are given that allow for the deduction of an average profile of the sea defences present along the Wash estuary, see figure 28.

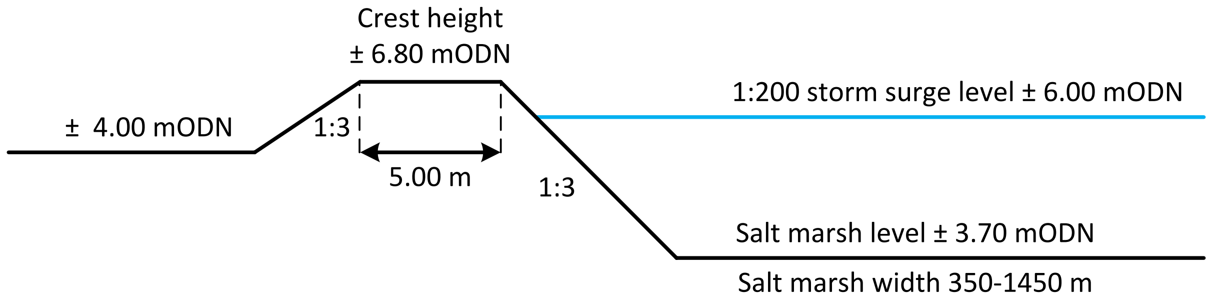


Figure 28: average cross-section of the flood defences bordering the Wash estuary.

Note: The squares indicate the downstream boundaries of the Catchment Flood Management Plans.

Figure 29: Wash estuary coastline and tidal reaches (Courtesy: *Ordnance Survey*).

1.3.3 River defences

As future changes in water level due to climate change also have an impact on the tidal reaches of the rivers discharging into the Wash estuary, a short description of the river defences within the tidal reach is given in this section.

All five tidal rivers discharging into the Wash estuary are trained, also beyond the tidal influence. River training within the tidal reach is necessary in order to provide flood protection and also to maintain both their drainage[22] function and the navigability of the shipping channels. The risk of flooding outside the tidal reach occurs mainly during periods of heavy rainfall. The main reasons for this are the fact that the lower river reaches are located within the Fenlands which are very flat and therefore there is almost no slope present towards the Wash estuary (1: 10,000 on average) and the fact that the rivers depend on the tidal cycle to freely discharge into the estuary. Therefore climate change will have the greatest impact on the flood safety in the catchment areas bordering the Wash estuary, both in terms of more intense and more frequent rainfall occasions, resulting in a larger discharge, and higher water levels on the estuary (sea level rise and larger and more frequent storm surges) that will reduce the discharge capacity as the duration of tide locking[23] increases.

River	Tidal limit at	Length[1)] [km]
Steeping	Clough bridge (from Burg Sluice)	4
Burgh sluice relief channel and Cowbank drain	Burgh Sluice	4
Witham	Grand Sluice (from Black Sluice)	5
Stonebridge drain *South 40 foot drain*	Maud Foster Sluice Black sluice (from Maud Foster Sluice)	9 3
Welland	Spalding	21
Nene	Dog-in-a-Doublet Sluice	77
Great Ouse[2)]	Denver Sluice	50
Old Bedford river	Brownshill Staunch (from Denver sluice)	80

[1)] Length from the border of the Shoreline Management Plan.
[2)] In case the Denver Sluice complex is open the tidal limit is situated at Brownshill Staunch near Earith. However the Denver Sluice is used for gravity discharge of fresh water and hence is closed during high tide.
Note: the tabulated values are already multiplied by two.

Table 6: primary river flood defences.

The following measures are planned for the near future to compensate for the consequences of climate change:

- The Haven[24] tidal flood defence near Boston, this is a tidal barrier across the river Witham [Jacobs Babtie, 2006 and River Witham Catchment plan, Environment Agency, December 2009];
- construction of tidal gates in the river Nene at Wisbech [River Nene Catchment plan, Environment Agency, December 2009];
- replacement of the crest walls on the embankments of the tidal river Ouse by 2035 and raising of the embankments along the Old Bedford River [The Great Ouse Tidal River Strategy, Environment Agency, 2009].

[22] The drainage system in the Fenlands is based on pumping the water from the watercourses into the main rivers, that in turn discharge into the Wash estuary. The pumping is necessary because much of the Fenlands lies below mean sea level.
[23] The time during which it is not possible for the river discharge to freely flow into the estuary because the outside water level is to high.
[24] The Haven is the entire stretch of the tidal river Witham.

These measures may not be necessary when a storm surge barrier is build across the mouth of the Wash estuary. The assessment regarding the necessity of these measures in case the construction of a storm surge barrier turns out to be feasible, falls outside the scope of this thesis. In table 6 the distances from the Wash estuary coastline until the tidal limit of the main rivers are shown, see also in figure 29.

The river dikes are assumed to have 1:3 slopes and a freeboard of 1.00 m above the maximum tidal level in mODN, see figure 30 below.

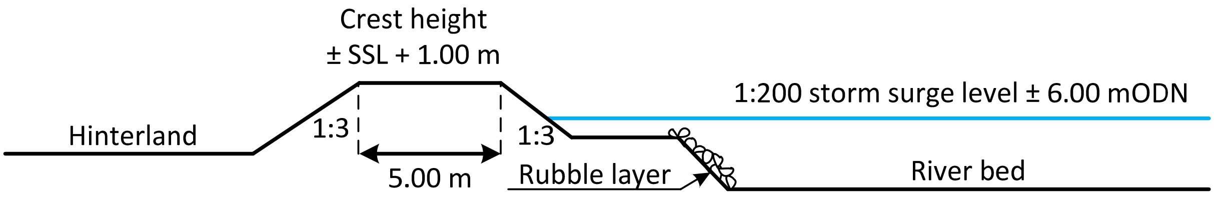


Figure 30: average cross-section of the river defences bordering the tidal rivers.

1.3.4 UK's flood defence policy

A condensed and somewhat simplified overview of the UK's flood defence legislation is presented in figure 31. After the summer of 2007 floods Sir Michael Pitt was asked by the UK Government to assess what happened, which has led to a set of recommendations to reform the management of flood and coastal erosion risk in England. In a nutshell the Flood and Water Management Act is the Governments response to the Pitt review and gives the Environment Agency, local authorities and other bodies duties and powers regarding preparing and putting in place strategies for managing flood risk in their areas. The powers are either directly given by this Act or in the form of amendments to the Coastal Protection Act, Land Drainage Act and Water Resources Act.

The Flood Risk Regulations 2009 implement the EU Flood Directive in UK Legislation. While the Environment Act requires the Environment Agency to supervise matters relating to flood defence, including land drainage in England. The Water Act deals with water supply and sewage and also encloses the legal obligation to draw up River Basin Management Plans, which is dictated by the EU Water Framework Directive. Finally it amends the Water Resources Act.

The Coastal Protection Act provides coastal protection authorities with general powers for coastal protection (erosion) and coastal defence works.

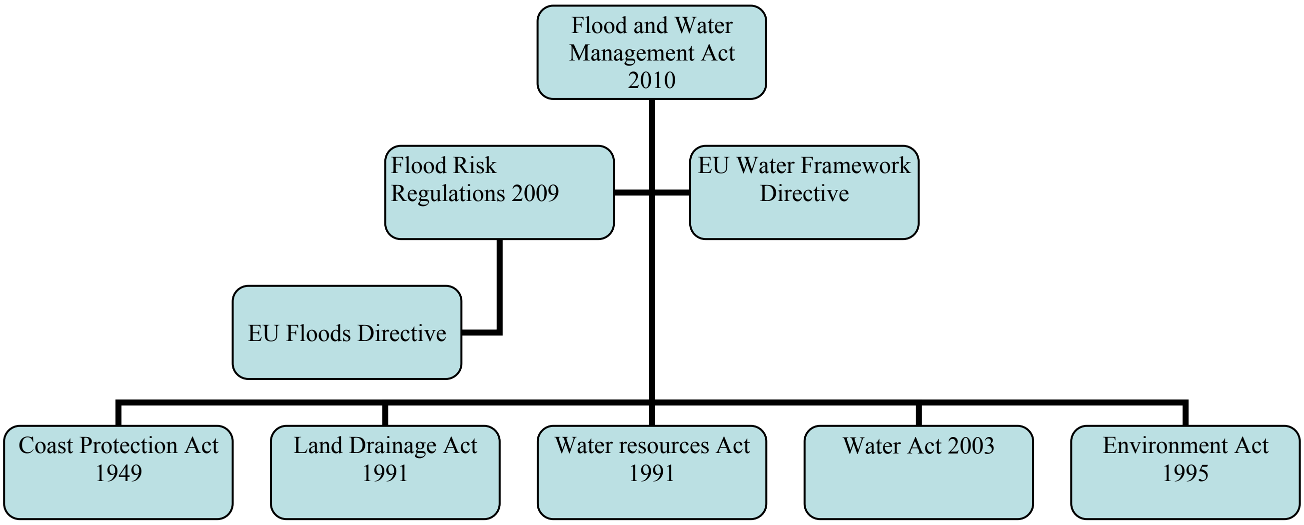


Figure 31: UK's flood defence legislation.

The leading flood defence authority in the UK is the Department for Environment, Food and Rural Affairs which is responsible for law making and the national flood defence and coastal erosion policy. The Environment Agency is responsible for the translation of these laws and policies into a national flood defence and coastal erosion management strategy. Besides this the Environment Agency is also the principal flood risk management operating authority and responsible for managing the flood defence from designated main rivers, the sea and coastal erosion.

The Lead Local Authorities and Internal Drainage Boards have the same obligations and are allowed to transfer these obligations to one another (if both are in agreement), their obligations are:

- developing and implementing local flood defence and coastal erosion strategies that fit in the national strategy;
- managing ordinary watercourses and coastal erosion problems;
- implementing measures and building and maintaining flood defence structures.

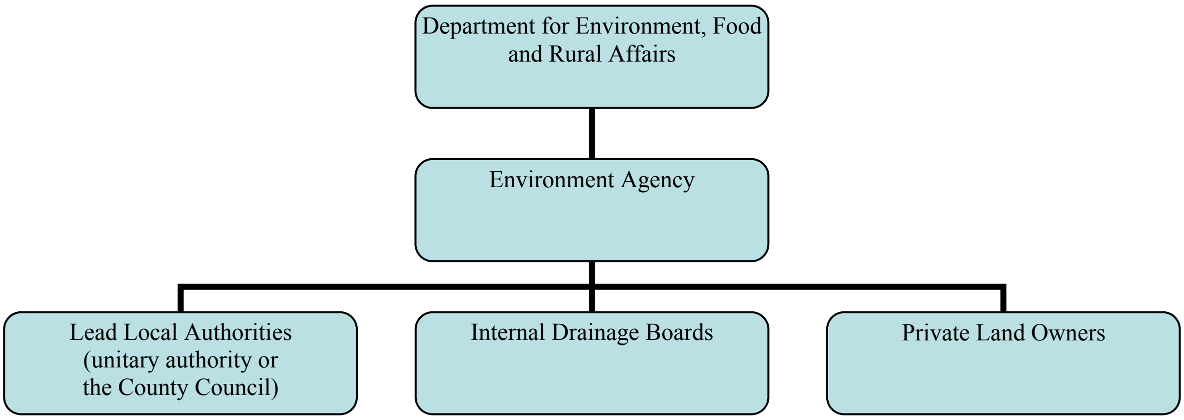


Figure 32: UK's flood defence authorities.

As can be seen in figure 32, private land owners are allowed to construct and maintain their own flood defences in the UK as long as these do not conflict with the national policy. In principle this setup provides The Wash Tidal Barrier Corporation with the possibility of constructing a privately owned storm surge barrier. In reality however it will not be that easy as the Wash estuary is heavily protected by environmental laws and the UK Government already stated that it is not convinced of the necessity of such a barrier.

Within UK's flood policy no flood defence standards or target risks are defined, the policy is defined in terms of the general aim of reducing risks to people and the natural environment, and the requirement to achieve value for money[25]. Safety levels are expressed as a Standard of Protection. [Ministry of Transport Public Works and Water Management, 2000]. As shown in table 7 indicative Standards of Protection are available based on the land use in an area. However these indicative standards do not represent an entitlement to defence at a given level but are intended as guidelines [Ministry of Transport Public Works and Water Management, 2000]. According to the table the Wash estuary coastal and fluvial defences should be in land use bands B or C, which, at least for the coastal defences, corresponds with reality (see section 1.3.2). For strategically important areas, such as the Thames estuary, a Standard of Protection of 1:1000 is used as design standard.

Land use band	Indicative Standards of Protection	
	River flooding return period [yr]	Coastal flooding return period [yr]
A	50-200	100-300
B	25-100	50-200
C	5-50	10-100
D	1.25-10	2.5-20
E	< 2.5	< 5

A = large urban areas at risk of flooding.
B = large extensive urban areas with some high-grade agricultural land and/or environmental assets of international importance requiring defence.
C = large areas of high-grade agricultural land or environmental assets of national significance at risk from flooding or impeded drainage, with some properties also at risk of flooding.
D = mixed agricultural land with occasional, often agricultural-related properties at risk from flooding. Agricultural land may be probe to flooding or water-logging. May also apply to environmental assets of local significance.
E = low-grade agricultural land, often grass, at risk from flooding or impeded land drainage, with isolated agricultural properties at risk from flooding, or environmental assets at little risk of frequent inundation.
Source: *Ministry of Transport Public Works and Water Management, 2000.*

Table 7: indicative Standard of Protection.

The Association of British Insurers used in their report on coastal flood risk along the English East Coast slightly different indicative Standard of Protection (SoP) levels [ABI, 2006]:

- rural areas defended to a minimum1:50 year SoP;
- small towns defended to a minimum 1:200 year SoP;
- larger towns defended to a minimum 1:500 year SoP;
- strategically important areas defended to a 1:1000 year SoP.

These SoP standards are of later date than those mentioned in table 7. Since it has proved to be impossible to find the current Standard of Protections standards on the Environment

[25] Appropriate standards for new defences are assessed on the basis of an economic analysis that compares the present value costs of different standards of defence against the present value of the avoided damage.

Agency website and because the values do not differ from each other to a great extend, the SoP levels as used in the Association of British Insurers report will be used.

National coastal flood management policy

For coastal flood and erosion management the national shoreline management policy is translated into one of the following four policies [Environment Agency SMP2, 2010]:

1) Hold the line: this involves holding the defence on its existing alignment.
2) Advance the line: this involves building new defences seaward of the existing defence line.
3) Managed realignment: this involves allowing the shoreline to move seaward or landward, with associated management to limit the effect on land use and environment.
4) No active intervention: this involves no investment in coastal defences or operations.

The boundaries at Gibraltar Point and Old Hunstanton match the neighbouring Shoreline Management Plans (Flamborough Head to Gibraltar Point Shoreline Management Plan and North Norfolk Shoreline Management Plan), see figure 33. These boundaries are implemented as a result of the need to treat the Wash estuary as one complete system. The boundaries in the rivers Witham, Welland, Nene and Great Ouse match the downstream boundaries of the respective Catchment Flood Management Plans, see figure 29.

Within The Wash SMP2 a distinction is made between four policy zones where common issues are to be faced (zones 1 to 4), also taking account of their external relationships to each other (e.g. sediment transport). Along the immediate adjacent Lincolnshire coastline three policy units are distinguished (policy units N to P), the adjacent Norfolk coastline is divided into three so-called Super Frontages, of which Super Frontage 1 and part of Super Frontage 2 are of importance as they compose the immediately adjacent Norfolk coastline.
The Policy Development Zones are (see figure 33):

Zone 1: from River Steeping at Gibraltar Point to Wolferton Creek;
Zone 2: from Wolferton Creek to south Hunstanton, where the land begins to rise;
Zone 3: Hunstanton Town;
Zone 4: Hunstanton Cliffs;
Policy unit N: South of Humberston Fitties to Teddlethorpe St Helen;
Policy unit O: Viking gas terminal to southern end of Skegness;
Policy unit P: Seacroft to Gibraltar Point;
Super Frontage 1: Old Hunstanton Dunes to Thornham;
Super Frontage 2: western end of Brancaster Bay to the eastern end of Stiffkey Bay.

In the table 8 management options for the different Policy Development Zones regarding the short-term, medium-term and long-term, as stated in the Wash SMP2, the Flamborough Head to Gibraltar Point SMP2 and the North Norfolk SMP2, are summarized.

Policy Development Zone	Short-Term (present day-2025)	Medium-Term (2025-2055)	Long-Term (2055-2105)
Zone 1	HtL	HtL / MR	HtL / MR
Zone 2	HtL	HtL / MR / NAI	HtL / MR / NAI
Zone 3	HtL	HtL	HtL
Zone 4	NAI	NAI	NAI / HtL
Policy unit N	HtL	HtL	HtL
Policy unit O	HtL	HtL	HtL / MR
Policy unit P	HtL	HtL	HtL / MR
Super Frontage 1			
Old Hunstanton Dunes	HtL	MR	MR
Holme Dunes	MR	MR	MR
Thornham sea bank	HtL	MR/HtL	HtL
Thornham	NAI	NAI	NAI
Super Frontage 2			
Thornmham-Titchwell	NAI	NAI	NAI
Titchwell RSPB reserve	HtL	HtL	HtL
Titchwell village	NAI	NAI	NAI
Brancaster Marsh	HtL / MR	HtL	NAI
Royal West Norfolk golf club	HtL	HtL	HtL
Brancaster & Brancaster Staithe	HtL	HtL	HtL

HtL = hold the line, MR = managed realignment, NAI = no active intervention.

Table 8: overview short-term, medium-term and long-term management options.

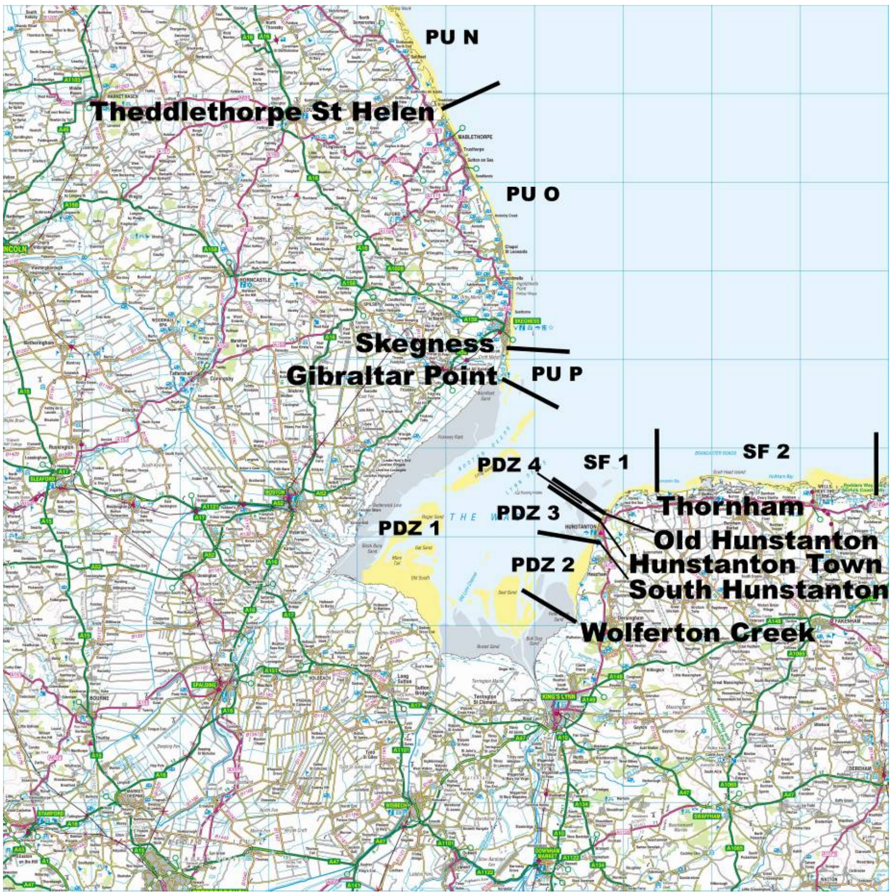


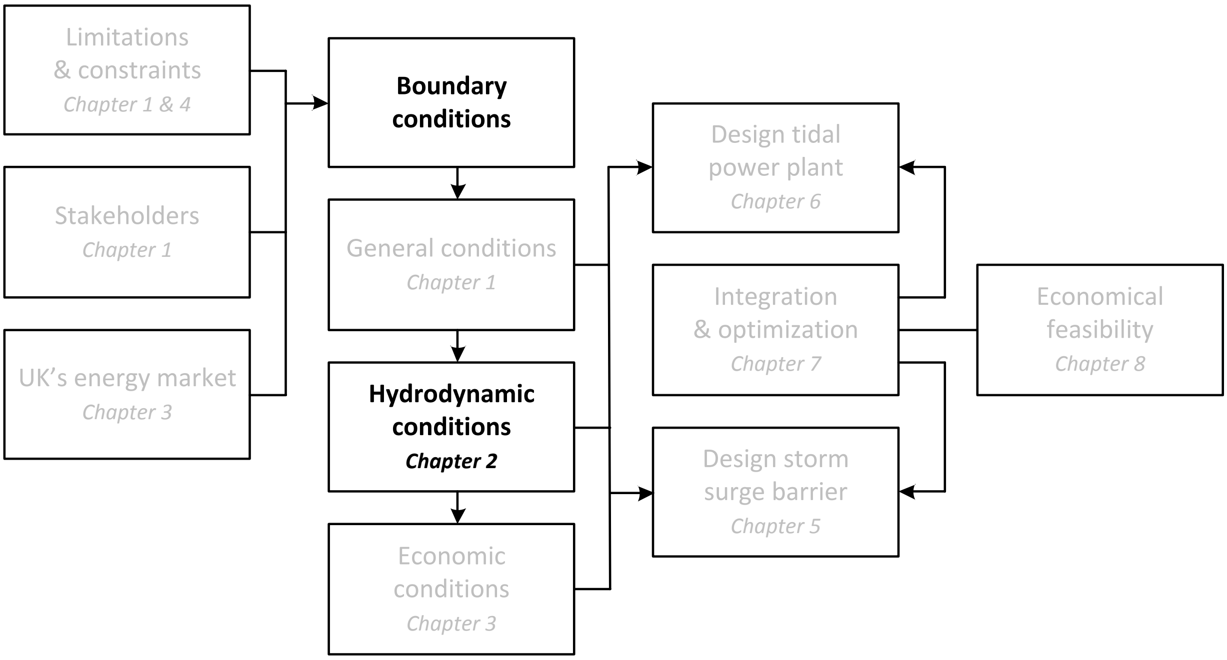
Figure 33: Policy Development Zones.

National river flood management policy

The Catchment Flood Management Plans (CFMP's) provide the national policy for managing the flood risk from rivers, including the influence of high tides. The five general CFMP Policy Options are as follows:

1) P1: no active intervention;
2) P2: reduce existing flood risk management actions, accepting increase of risk over time;
3) P3: continue with existing or alternative actions to manage flood risk at the current level, accepting that flood risk will increase over time from this baseline;
4) P4: Take further action to sustain the current level of flood risk into the future (responding to the potential increases in risk from urban development, land use change and climate change);
5) P5: take further action to reduce flood risk (now and/or in the future).

The CFMP policies covering the inland areas from Gibraltar Point to Wolferton Creek (see figure 33) are all Policy Option 4. The area between Wolferton Creek and Hunstanton is at low to moderate risk of river flooding, which means that it is generally possible to reduce existing flood risk management actions. Hence Policy Option 2 is applicable to the hinterland behind this section of coastline.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

2 HYDRODYNAMIC CONDITIONS

In this chapter the coastal and fluvial hydrodynamic conditions present within the Wash estuary and the intertidal part of the rivers discharging into the estuary will be discussed. Starting with the tide, next the offshore and near shore wave and wind conditions. Then the fluvial hydrodynamic boundary conditions and the sediment transport. Finally the impact of climate change on the system is assessed.

2.1 Tide

The generation of the astronomical tides will be the starting point, followed by a harmonic analysis of the tidal wave on the continental shelf in the North Sea, offshore of the Wash estuary. This is done in order to be able to determine the characteristic of the tide. Next a description of the shallow water tides is given. Then the propagation and deformation of the tidal wave when entering the estuary will be described. After which the tidal range and velocity of the tidal current within the Wash estuary will be determined, as is the tidal power potential of the tidal wave.

2.1.1 Astronomical tides

When designing a tidal power plant the key factor is the astronomical tide, therefore it is important to gain insight in the character of the tide within the Wash estuary. In appendix 5 Newton's equilibrium theory is used to explain the origin of the daily inequality and the spring-neap cycle.

The daily inequality is a result of the declination of the earth axis and is of importance with respect to the character of the tide at a certain location on earth. This tidal character (diurnal, semidiurnal or mixed) is determined by means of the form factor F, which is defined as the ratio between the sum of the two main diurnal components (K_1 and O_1) and the two main semidiurnal components (M_2 and S_2). For more detailed information the reader is referred to appendix 5.

The amplitudes of the main tidal diurnal and semidiurnal constituents from the two measuring stations of the UK Tide Gauge Network nearest to the Wash estuary are presented in table 9, as is the computed form factor.

Tidal category	M_2 [m]	S_2 [m]	K_1 [m]	O_1 [m]	F [-]
Cromer	1.568	0.533	0.145	0.158	0.14
Immingham	2.260	0.741	0.155	0.171	0.11

Source: British Oceanographic Data Centre & Proudman Oceanographic Laboratory.

Table 9: harmonic constants along the English east coast.

As was to be expected the tide on the North Sea in front of the Wash estuary is characterized as a semidiurnal tide. Since the mean spring tidal range is approximately 6.25 m, which is larger than 4 m, the tidal environment is characterized as a macro-tidal regime.

Besides the influence of the daily inequality the amplitude of the astronomical tide is also influenced by the relative position of sun and moon with respect to the earth. When sun and moon are in line with each other the amplitude of the tide is largest, referred to as spring tide.

During neap tide the tidal amplitude is smallest as sun and moon are 90° out of phase. Due to the elliptic orbit of the moon around the earth and the elliptic orbit of the earth-moon system around the sun the differential pull on the earth's water masses does not remain constant over the year. Hence, the amplitude of the spring-neap cycle changes during the lunar month due to the influence of the moon and during a year as a result of the sun's influence.

As the influence of the moon is largest (see appendix 5) one could conclude that analysing one month of tidal data will be sufficient. However since the influence of the sun on the astronomical tide still amounts to 31% of the total tide, it may be better to use one year of tidal data in the analysis of the tidal power potential. Due to a lack of data, the performed analysis of the astronomical tide is based on 1.5 month of tidal prediction data.

2.1.2 Shallow water tides

In the previous section Newton's equilibrium theory of tides has been used to explain several important concepts regarding the generation of the astronomical tide. But in reality the presence of the continents and the limited water depth in the open oceans prevent the generation of the equilibrium tide. The propagation of a tidal wave into the marginal seas, coastal zones and estuaries can be analysed using the St. Vernant equations, also known as the shallow water equations, which represent a coupled system of differential equations describing the relation between water level and discharge as function of time and distance.

$$B \cdot \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (2.1)$$

And

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A_s}\right) + g \cdot A_s \cdot \frac{\partial h}{\partial x} + c_f \cdot \frac{|Q| \cdot Q}{A_s \cdot R} = 0 \qquad (2.2)$$

Were:

B	:	storage width	[m]
h	:	water level	[m]
t	:	time	[s]
Q	:	discharge	[m^3/s]
x	:	distance	[m]
A_s	:	current-carrying cross-section	[m^2]
c_f	:	friction coefficient	[-]
R	:	hydraulic radius	[m]

The sea-borne tidal asymmetry is transferred into the estuary where, as a result of the further decreasing depth, non-linear effects are being enhanced, resulting in an increasing tidal asymmetry (see also appendix 5). Some amplitude amplification is to be expected as the tide propagates into the estuary. Due to the presence of bottom friction both the incoming and reflected wave are partly damped, resulting in a wave pattern with a partly standing character and a partly propagating character. As a result of the partly standing wave character a phase difference between water level and current velocity is to be expected (current velocity leads the water level variation). Apparently damping of the tidal wave due to bottom friction has a large effect in the Wash estuary as the amplitude of the tidal wave only increases approximately 0.10 m from the mouth of the estuary to the landward side of the basin (see section 2.1.3), despite a considerable decrease in water depth towards the end of the basin.

Also the wave celerity decreases in shallower water, resulting in a shortening of the wave length as the wave period remains constant, see equations 2.3 and 2.4.

$$L = c \cdot T \tag{2.3}$$

And

$$c = \sqrt{g \cdot d} \tag{2.4}$$

Were:

L	:	wave length	[m]
c	:	wave celerity	[m/s]
T	:	wave period	[s]
g	:	gravitational acceleration	[m/s^2]
d	:	water depth	[m]

The average depth of the Wash estuary is less than 10 m, although the deepest sections of the Lynn Deeps are 40 to 50 m below Ordnance Level Newlyn (ODN) [The Wash SMP2, appendix C, 2010], see also figure 34. The length of the basin is approximately 25 km. Using equations 2.3 and 2.4 and a water depth of 10 m, the wave celerity and wave length of the four main tidal constituents are computed, see table 10 for the results.

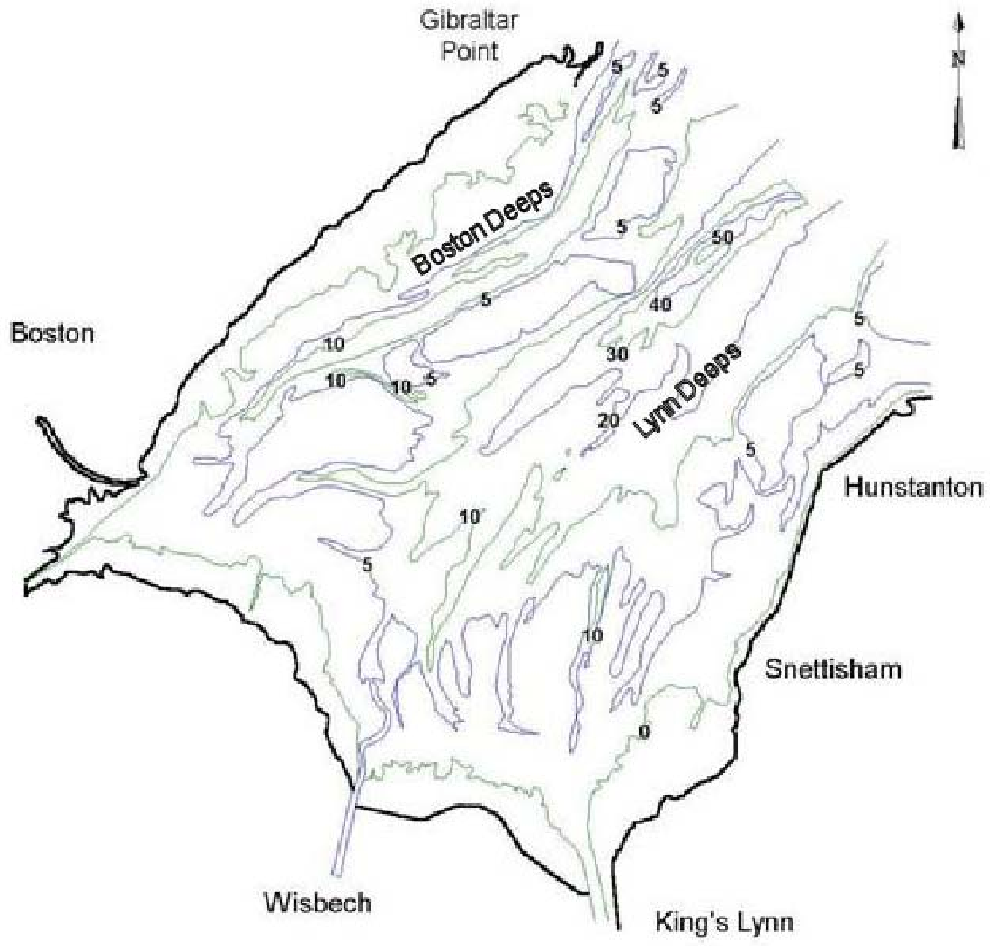


Figure 34: simplified bathymetry of the wash estuary, contour lines in m below ODN. (Courtesy: *Royal Haskoning*)

Tidal constituent	Wave period [hr]	Wave length [km]	Amplitude [1)] [m]
M_2	12.42	443	2.260
K_1	23.93	853	0.155
S_2	12.00	428	0.741
O_1	25.82	256	0.171

Wave celerity of all constituents is 9.9 m/s.
[1)] Because no data with respect to the amplitudes of the four main tidal constituents is available at the time for the tide within the Wash estuary, the data of the Immingham measuring station of the UK Tide Gauge Network is used. This station was preferred over the Cromer station since the mean tidal amplitude is closer to that of the Wash estuary. Mean tidal amplitude Immingham 4.20 m; Cromer 2.92 m.
(Source: British Oceanographic Data Centre & Proudman Oceanographic Laboratory)

Table 10: wave length and amplitude of the four major tidal constituents.

From table 10 can be concluded that the basin length is in the order of 1/20 of the wave length of the M_2 and S_2 tidal constituents, being the main tidal constituents along the English eastern shoreline. Therefore a storage basin approach can be used to describe the change in time of the water level within the future basin and also to assess the influence of the barrier on the tidal amplitude behind it, see equations 2.5 and 2.6.

$$A_b \cdot \frac{dh}{dt} = Q_{in} - Q_{out} \qquad (2.5)$$

And

$$h_{NS}(t) - h_b(t) = \frac{L}{g \cdot A_s} \cdot \frac{dQ}{dt} + \chi \cdot \frac{|Q| \cdot Q}{g \cdot A_s^2} \qquad (2.6)$$

Were:

A_b	:	basin area	[m^2]
h	:	water level	[m]
t	:	time	[s]
Q	:	discharge	[m^3/s]
h_{NS}	:	water level on North Sea	[m]
h_b	:	basin water level	[m]
L	:	length	[m]
g	:	gravitational acceleration	[m/s^2]
A_s	:	current-carrying cross-section	[m^2]
χ	:	loss coefficient	[-]

2.1.3 Properties of the tide in the Wash estuary

The Wash estuary is characterized as a semidiurnal macro tidal regime, the estuary's geometry consists of a large deep central main channel, the Lynn Deeps, a smaller secondary channel located along the western shoreline of the estuary (Boston Deeps) and extensive intertidal sand and mud flats, for a large part fringed by salt marshes. The total water covered area of the estuary amounts to approximately 615 km^2 during high tide and approximately 325 km^2 during low tide, the remaining 290 km^2 consists of intertidal sand and mud flats [Dare, 2004]. The five tidal rivers discharging into the estuary have all formed tide dominated deltas at the landward end or the basin, consisting of sand and mud flats fringed by salt marshes.

This section deals in succession with the following subjects:
- the vertical tide;
- the horizontal tide;
- tidal asymmetry;
- tidal prism;
- tidal window;
- energy potential.

Vertical tide

According to the *Encyclopedia of the World's Coastal Landforms* [Bird, 2010] the mean spring tidal range is 6.40 m at Hunstanton and diminishes along the east coast of the Wash estuary to 5.90 m at King's Lynn. On the west coast the mean spring tidal range at Skegness is 6.10 m and increases along the coast to 6.80 m at Boston (Tabs Head). This is in good agreement with the mean spring tidal range based on data from the tidal prediction service provided by Admiralty EasyTide[26], see table 11. The tabulated mean spring tidal ranges are based on a 1.5 month period and are of the same order of magnitude. Therefore the long term mean values as stated in the Encyclopedia of the World's Coastal Landforms will be used to determine the energy potential of the tide in the Wash estuary.

In figure 35 the location of the tidal prediction sites is depicted, the predicted tidal signals at all sites are included in appendix 6.

The average mean neap tidal range within the Wash estuary varies from 3.10 m near the mouth to 3.20 m at the landward side [The Wash SMP2, appendix C, 2010]. This is also in good agreement with the tidal predictions provided by Admiralty EasyTide, hence the long term data from the Wash SMP2 will be used to determine the energy potential of the tide in the Wash estuary.

	SK	BOS	HUN	KLY	TAHE	OWK	WES
Mean Sea Level	3.93	3.56	4.10	3.77	4.08	3.75	4.06
Mean High Water	6.13	5.64	6.46	5.91	6.58	6.23	6.50
Mean Low Water	1.73	1.48	1.73	1.63	1.58	1.27	1.61
Mean High Water Spring	6.85	6.50	7.35	6.85	7.45	7.07	7.37
Mean Low Water Spring	1.00	1.40	0.98	1.30	0.80	0.58	1.05
Mean High Water Neap	5.10	4.52	5.38	4.80	5.35	5.08	5.30
Mean Low Water Neap	2.67	1.72	2.67	2.00	2.58	2.17	2.42
Mean Spring tidal range	5.85	5.10	6.37	5.55	6.65	6.48	6.32
Mean Neap tidal range	2.43	2.80	2.72	2.80	2.77	2.92	2.88
Mean Tidal range	4.40	4.16	4.73	4.29	5.00	4.96	4.89

Levels in m above CD (CD = -3.00 m ODN)

SK	=	Skegness	TAHE	=	Tabs Head
BOS	=	Boston	OWK	=	Outer Westmark Knock
HUN	=	Hunstanton	WES	=	West Stones
KLY	=	King's Lynn			

Table 11: water levels and tidal ranges.

What stands out is that near the mouth of the estuary the mean spring tidal range is highest along the eastern shoreline, while at the landward side the highest mean spring tidal range occurs at the western shoreline. This effect is not visible in the figures regarding the mean neap tidal range, as both near the mouth and at the landward side the mean neap tidal range is highest along the eastern shoreline. Since the prevailing wind direction is from west to east,

[26] http://easytide.ukho.gov.uk/EasyTide/EasyTide/index.aspx

wind set-up can be ruled out. The same holds for the effect of Coriolis as the change in flow direction between ebb tide and flood tide would average out the difference. Therefore it must be an effect induced by the estuary's bathymetry.

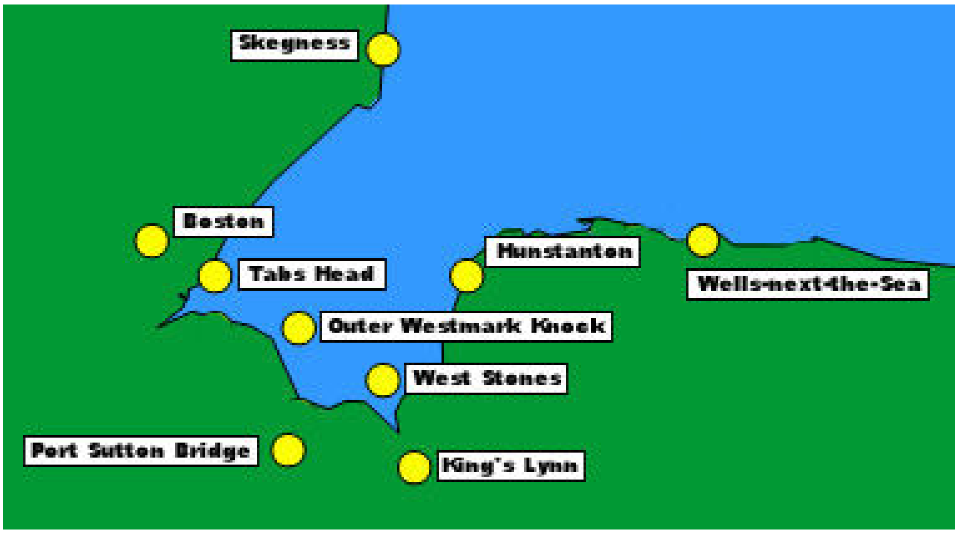


Figure 35: location of the tidal prediction sites.

Near the mouth of the Wash estuary the average mean spring tidal range is 6.25 m, due to the decreasing depth further into the estuary the average mean spring tidal range increases to 6.35 m near the landward side of the estuary. The average mean neap tidal range varies from 3.10 m near the mouth to 3.20 m near the landward side. In table 12 an overview is given of the mean tidal range that can be used to compute the tidal energy potential at several locations in the Wash estuary.

	Mean spring tidal range [m]	Mean neap tidal range [m]	Mean tidal range [m]
At the mouth	6.25	3.10	4.70
At one third	6.28	3.13	4.73
Halfway	6.30	3.15	4.74
At two third	6.32	3.17	4.75
At the landward side	6.35	3.20	4.78

Table 12: mean tidal range at several locations within the Wash estuary.

During a storm event, low atmospheric pressure[27] in combination with high wind speeds cause a wind induced set-up (or set-down), resulting in an extra rise of the water level in excess of the predicted variation of the astronomical tide. Extreme tidal levels occur when a storm surge coincides with spring tide. In table 13 an overview is given regarding these extreme tidal levels at several locations in the Wash estuary [The Wash SMP2, appendix C, 2010].

[27] The contribution of low atmospheric pressure is minor [Hume et al, 2002].

From tables 11 and 13 it can be established that an extreme storm surge increases the mean spring tidal water level by approximately 2.00 to 3.00 m, depending on the location within the estuary and the return period. The extreme wave height of wind sea and swell waves must be superimposed on these storm surge levels, see section 2.2.

Location	Return period				
	1:50	1:100	1:200	1:500	1:1000
Burg sluice[1]	7.76	7.90	8.03	8.21	8.34
Mouth Witham	8.64	8.78	8.93	9.12	9.27
Mouth Welland	8.66	8.80	8.95	9.14	9.29
Mouth Nene	8.71	8.86	9.01	9.21	9.35
Mouth Great Ouse	8.78	8.93	9.08	9.28	9.43
Snettisham Scalp	8.71	8.86	9.02	9.22	9.37
Heacham	8.67	8.82	8.97	9.18	9.33
Hunstanton	8.60	8.76	8.91	9.11	9.27

Levels in m above CD (CD = -3.00 m ODN)
[1]River Steeping near Gibraltar Point.

Table 13: extreme tidal levels.

Horizontal tide
A southerly directed residual flow passes down the Lincolnshire coastline, turning south-eastwards across the mouth of the Wash estuary and continues further along the Norfolk coastline [Dare, 2004]. The strongest flood flow enters the estuary from the north, a weaker flood flow enters from the east. The flood flow enters the estuary predominantly through the Lynn Deeps that are situated in the centre of the estuary and progresses further into the estuary following the main channels in south-westward direction. The ebb flow leaves the estuary predominantly along the estuary margins in north-eastward direction, see figure 36 for the residual flow pattern within the estuary.

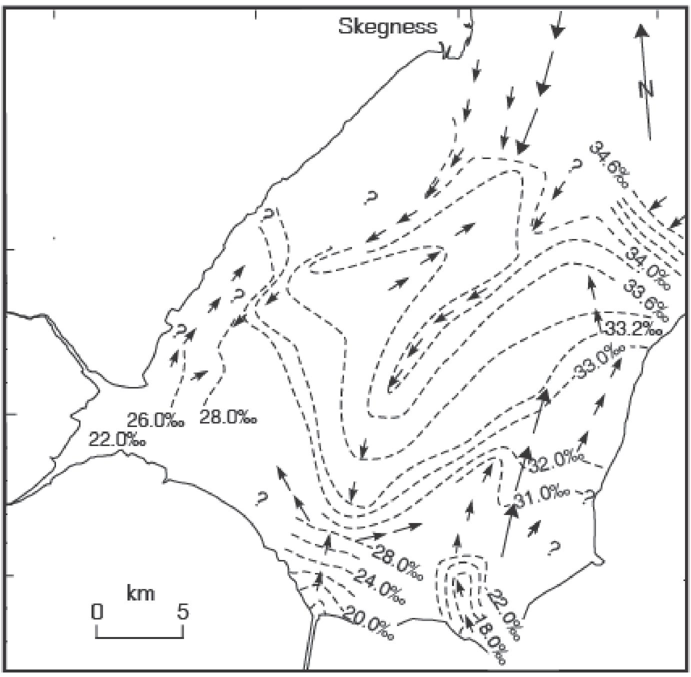


Figure 36: residual tidal flow direction.
(Courtesy: *Wingfield et at, 1978*)

The strongest tidal currents within the Wash estuary occur in the main channel (Lynn Deeps) during spring tide. The maximum depth averaged spring tidal flood current velocity is in the order of 1.20 m/s, the maximum depth averaged spring tidal ebb velocity in the order of 1.00 m/s. The maximum depth averaged neap tidal flood and ebb tidal current are in the order of respectively 1.02 m/s and 0.56 m/s [The Wash SMP2, appendix C, 2010].

Tidal asymmetry
As described in the section 2.1.2 the amplitudes of both the horizontal and vertical tide are damped progressively as a result of bottom friction, the depth decreases considerably further into the basin and the partly standing wave pattern resulted in a phase difference between the horizontal and vertical tide ($0 < \text{phase shift} < \pi/2$). As a result both the vertical and horizontal tide are deformed.

The vertical and horizontal deformation of the horizontal tide are very important factors in relation to the net sediment transport processes within the Wash estuary. The horizontal deformation of the horizontal tide results in a skewed velocity signal and relates to the transport of coarse sediment. The vertical asymmetry of the horizontal tide relates to the transport of fine sediment and results in a saw-tooth velocity signal.

As is explained in appendix 5 the Lynn Deeps are characterized by flood dominance, resulting in a net transport of course sediment into the estuary. In contrast to the central part of the estuary both along the eastern and western boundary the residual flow direction is in ebb direction (see figure 36), which indicates ebb dominance and hence net sediment transport towards the North Sea. However the net trend is the overall import of sediment in the estuary basin [The Wash SMP2, appendix C, 2010], see also figure 37.

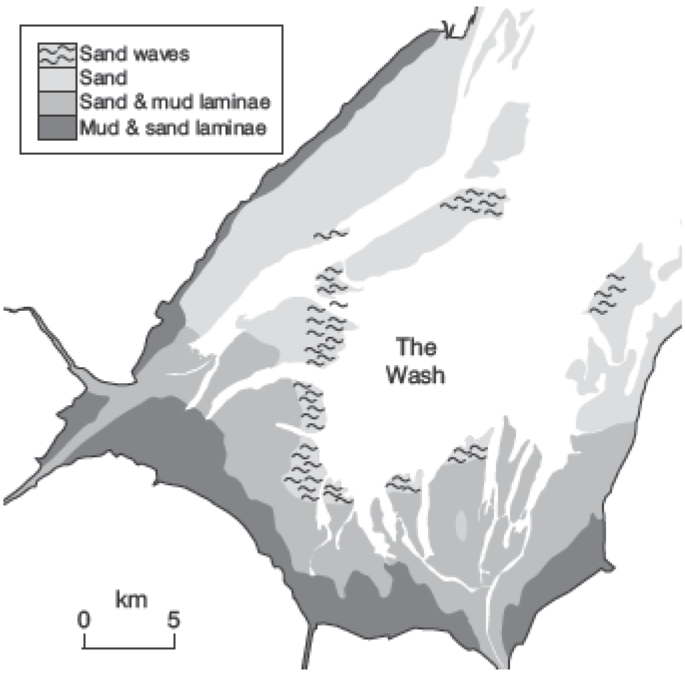


Figure 37: distribution of intertidal sediments in the Wash.
(Courtesy: *Ke et al, 1996, after Wingfield et al 1978*)

The governing process with respect to the sediment transport of fines is the difference in duration between high-water slack and low-water slack. The location within the estuary were flood dominance occurs, Lynn Deeps, the high-water slack duration is longest and hence net transport of fines in landward direction occurs. This is consistent with the distribution of intertidal sediments as depicted in figure 37.

The margins of the estuary, were ebb dominance occurs, are characterized by a longer low-water slack duration. According to theory this will result in a net export of fines. This seems not to be the situation, as mud is also present along the eastern and western shoreline. However because of the large intertidal area another mechanism plays a role. Due to the small water depth and large concentration fines in the water column strong settling occurs, apparently this process compensates for the short high-water slack duration.

Horizontal and vertical asymmetry of the vertical tide influence the water level within the estuary. Already at the mouth of the Wash estuary the falling period is longer than the rising period, this vertical asymmetry of the vertical tide increases slightly as the tidal wave progresses further into the estuary, see appendix 7. The asymmetry is most pronounced near the ports of Boston and King's Lynn that are located some distance upstream the tidal rivers Witham and Great Ouse respectively. Keeping in mind the propagation speed of the tidal wave explains this vertical asymmetry. During rising tide the crest of the tidal wave propagates into the estuary, hence the water depth is larger and as a result the wave celerity is larger than during the falling tide.

From table 14 can be concluded that horizontal asymmetry of the vertical tide is barely present at the mouth of the Wash estuary. During spring tide the high waters are slightly higher above mean sea level than the low waters and during neap tide it is the other way around. This asymmetry progressively increases in landward direction.

	SK	BOS	HUN	KLY	TAHE	OWK	WES
MSL	3.93	3.56	4.10	3.77	4.08	3.75	4.06
MHWS - MSL	2.92	2.94	3.25	3.08	3.37	3.32	3.31
MSL - HLWS	2.93	2.16	3.12	2.47	3.28	3.17	3.01
MHWN - MSL	1.17	0.96	1.28	1.03	1.27	1.33	1.24
MSL - MLWN	1.26	1.84	1.43	1.77	1.50	1.58	1.64

MSL in m above CD (CD = -3.00 m ODN)

SK = Skegness
BOS = Boston
HUN = Hunstanton
KLY = King's Lynn
TAHE = Tabs Head
OWK = Outer Westmark Knock
WES = West Stones

MSL = Mean Sea Level
MHWS = Mean High Water Spring
MLWS = Mean Low Water Spring
MHWN = Mean High Water Neap
MLWN = Mean Low Water Neap

Table 14: water levels and tidal ranges.

Tidal prism

According to Ke et al, 1996 the spring tidal prism[28] in the Wash estuary amounts to $2.8 \cdot 10^9$ m^3, a simple calculation using equation 2.7 results in an estimation of the spring tidal prism of $2.9 \cdot 10^9$ m^3, which is of the same order of magnitude.

$$P = \overline{R} \cdot \overline{A} \qquad [2.7]$$

Were:

P	:	tidal prism	[m^3]
$\overline{R}$	:	mean tidal range	[m]
$\overline{A}$	:	average basin area	[m^2]

The tidal prism is an important factor with respect to the sluicing capacity of the tidal power plant and for the sediment transport in the estuary. As a result of the construction of a tidal barrage the tidal prism is likely to decrease and thus has consequences for the water level and morphology within the estuary and the adjacent Lincolnshire and Norfolk coast lines.

	Mean tidal range [m]	**Tidal prism [m^3]**
Mean spring tide	6.25	$2.9 \cdot 10^9$
Mean tide	4.70	$2.2 \cdot 10^9$
Mean neap tide	3.10	$1.5 \cdot 10^9$

Average basin area is taken to be 470 km^2

Table 15: tidal prism.

Tidal window

The port of Boston has no tidal window[29] as the approach channels are kept to depth. The ports of King's Lynn and Sutton Bridge have a tidal window that depends on the draught of the vessels. The marinas at Fosdyke and Wisbech have a tidal window of 2hrs on either side of High Water.

Energy potential

The annual energy potential resulting from the tidal range within the Wash estuary can be computed using the following rule of thumb:

$$E_p = \frac{N_{hr} \cdot N_{M2} \cdot \overline{R}^2 \cdot A_b \cdot \gamma_w}{2 \cdot N_s} \qquad [2.8]$$

Were:

E_p	:	annual energy potential of the basin	[kWh]
N_{hr}	:	number of hours per year; 8760	[hr]
N_{M2}	:	number of half tidal cycles per day; 3.87	[-]
N_s	:	number of seconds per day; 86400	[s]
$\overline{R}$	:	mean tidal range	[m]
A_b	:	basin area	[m^2]
γ_w	:	volumetric weight of sea water	[kN/m^3]

[28] Here tidal prism is defined as the volume of water exchanged in a basin between mean high tide and mean low tide.
[29] Source: www.ports.org.uk

As the tidal barrage also has to function as a storm surge barrier, it is most likely that the barrage will be situated near the mouth of the estuary. Taking into account the mean tidal range at the mouth (4.70 m), the total basin area (615 km^2) and the density of water (1025 kg/m^3), this results in a potential energy yield of 26,800 GWh per annum.

The main parameters in equation 2.8 are the mean tidal range, the basin area and the density of water. In the remainder of this section their influence on the annual energy potential in the Wash estuary will be established. This is done in order to be able to determine the governing parameter in the Wash estuary.

Although the square of the mean tidal range is taken in equation 2.8, it is expected that the influence on the annual energy potential is not governing because the variation is only 0.08 m from the estuary's mouth to the landward side (table 12). This is supported by the results of the sensitivity analysis presented in table 16. The difference between the lowest and highest annual energy yield is approximately 3%.

Mean tidal range [m]	**Annual energy potential** [GWh]
4.70	26,800
4.73	27,143
4.74	27,258
4.75	27,373
4.78	27,720

Basin area is taken 615 km^2, the density of water 1025 kg/m^3

Table 16: sensitivity of annual energy potential with respect to the mean tidal range.

As five tidal rivers discharge into the estuary the water is likely to have a density somewhere in between the densities of salt water and fresh water. Table 17 shows that the lowest and highest energy yield differ approximately 2.5% and therefore the density is not the governing factor.

Density of water [m]	**Annual energy potential** [GWh]
1025	27,258
1020	27,125
1015	26,992
1010	26,859
1005	26,726
1000	26,593

Basin area is taken 615 km^2, the mean tidal range is taken 4.74 m.

Table 17: sensitivity of annual energy potential with respect to the density of water.

The Wash estuary has an area of approximately 615 km^2, in table 18 the energy potential is presented for the whole basin and for ⅔, ½ and ⅓ of the basin area. As expected in the Wash estuary the basin area is the governing factor with respect to the energy potential.

Basin area [km^2]	Annual energy potential [GWh]
615	27,258
410	18,172
308	13,629
205	9,086

Mean tidal range is taken 4.74 m, the density of water 1025 kg/m^3

Table 18: sensitivity of annual energy potential with respect to the basin area.

2.2 Waves

The orientation of the East English coastline near the mouth of the Wash estuary is such that wind sea and swell coming from the directional sector north to east will directly enter the Wash estuary. Due to the geometry of the main channel in the Wash estuary, waves coming in from a north-eastern direction will penetrate deep into the estuary as their direction is in line with the direction of the main channel (Lynn Deeps).

As can be seen in figure 38 wind sea predominantly arrives from northern to eastern directions, while swell waves arrive predominantly from a northern direction. Wind waves travel predominantly along a northeast to southwest axis, either in onshore direction or in offshore direction. This seems strange as the wind direction at this latitude is predominantly west, see also section 2.3. However the selected offshore location (53.4° N, 1.18° S) in front of the Wash estuary is lying at the leeside of the UK coastline, hence the wind waves are both depth and fetch limited. During approximately 22.5% of the time swell waves come from the north, also the highest swell waves originate from the north.

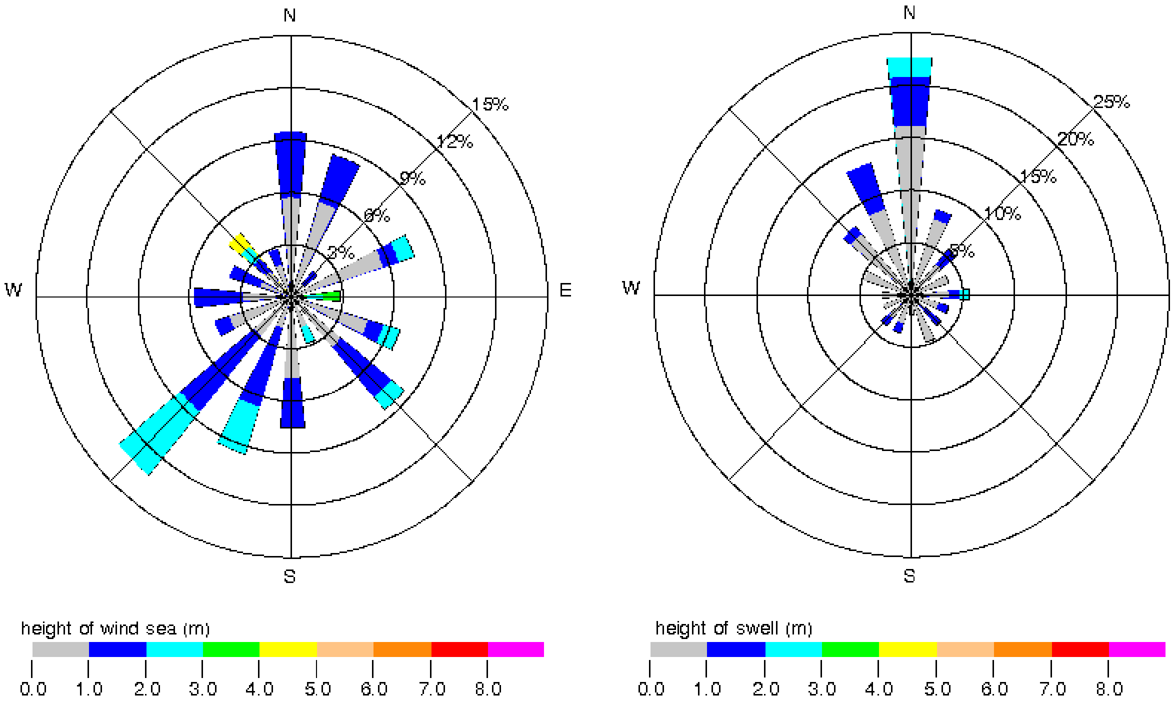


Figure 38: on the left: wave height rose regarding wind sea. On the right: wave height rose regarding swell (Courtesy: *BMT ARGOSS*).

2.2.1 Offshore extreme wave conditions

In order to determine the offshore extreme wave conditions on the North Sea in front of the mouth of the Wash estuary the initial distribution approach is used, for more detailed information the reader is referred to appendix 8.

Table 19 shows the offshore extreme significant wave height for different storm durations and return periods. The duration of the storms is chosen because storm durations longer than 12 hours are important for the swell conditions (typical 12 to 15 hours on the North Sea), while shorter storm durations are important for wind sea conditions (typically 6 to 8 hrs on the North Sea).

SoP		H_s for storm duration [m]			
Return period [yr]	Q_s [-]	6 hrs	8 hrs	12 hrs	15 hrs
50	0.02	8.40	8.25	8.02	7.89
200	0.005	9.55	9.41	9.20	9.08
500	0.002	10.35	10.23	10.03	9.91
1000	0.001	10.98	10.86	10.68	10.57
2000	0.0005	11.62	11.51	11.34	11.23
10,000	0.0001	13.17	13.08	12.94	12.84

SoP = standard of protection
H_s = extreme significant wave height

Table 19: offshore extreme significant wave height for several return periods and storm durations.

2.2.2 Nearshore extreme wave conditions

Between May 1999 and May 2000 the Environment Agency deployed a Waverider buoy in the centre of the Wash estuary's mouth at a depth of 24 m-CD [The Wash SMP2, appendix C, 2010]. The results of the measurements are presented in the table below. Cooper found that the intertidal environment of the Wash is estuary effective in dissipating wave height by, on average, 83% and in dissipating wave energy by, on average, 91% with respect to the incident wave conditions [Cooper, 2005].

	$H_{s,mouth}$ [m]	T [s]	$H_{s, saltmarsh}$ [m]
Maximum	2.81	26.9	0.48
Minimum	0.06	3.0	0.01
Mean	0.61	5.8	0.10

Source: *Cooper, 2005*.
H_s = significant wave height
T = wave period
Note: probably the maximum period should be 16.9 s.

Table 20: significant wave height in the Wash estuary according to Cooper.

The near shore wave height is assumed to be 5 m, with a corresponding peak period of 14 s[30].

[30] Source: Prof. dr. ir. J.K. Vrijling, Delft University of Technology.

Furthermore wind sea will be generated within the basin itself, but these waves will typically have smaller periods and a lower wave height than those entering the estuary from the North Sea. As a result of the basin's internal geometry waves are predominantly generated by wind action along a northeast to southwest axis. A first indication of the internally generated significant wave height and corresponding peak period can be found using Bretschneider's equations:

$$\tilde{H} = 0.283 \cdot \tanh\left(0.53 \cdot \tilde{d}^{\,0.75}\right) \cdot \tanh\left(\frac{0.0125 \cdot \tilde{F}^{0.42}}{\tanh\left(0.53 \cdot \tilde{d}^{\,0.75}\right)}\right) \quad [2.9]$$

And

$$\tilde{T} = 7.54 \cdot \tanh\left(0.833 \cdot \tilde{d}^{\,0.375}\right) \cdot \tanh\left(\frac{0.077 \cdot \tilde{F}^{0.25}}{\tanh\left(0.833 \cdot \tilde{d}^{\,0.375}\right)}\right) \quad [2.10]$$

Were:

$\tilde{H}$	:	$\frac{g \cdot H_s}{U^2}$	[-]
$\tilde{T}$	:	$\frac{g \cdot T_p}{U}$	[-]
$\tilde{F}$	:	$\frac{g \cdot F}{U^2}$	[-]
$\tilde{d}$	:	$\frac{g \cdot d}{U^2}$	[-]
g	:	gravitational acceleration	[m/s^2]
F	:	fetch	[m]
U	:	wind velocity at 10 m	[m/s]
d	:	water depth	[m]
T_p	:	peak wave period	[s]

An average water depth of 10 m, a wind speed of 34 m/s and a fetch of 25 km results in a significant wave height of 2.47 m and a corresponding peak period of 6 s. These are ballpark figures for the internally generated extreme significant wave height and corresponding peak wave period.

2.2.3 Tsunamis

The tsunami event in the Indian Ocean in December 2004 has lead to the assessment of the risk of a tsunami reaching the UK coastline. The assessment was performed by the British Geological Survey and identified potential sources of tsunamis in the following regions [British Geological Survey, 2005]:

- UK coastal waters (North Sea basin);
- northwest Europe continental slope;
- plate boundary area west of Gibraltar;
- Canary islands;
- Mid-Atlantic Ridge
- North America's eastern continental slope;
- Caribbean.

Historical and geological evidence shows that in the past tsunamis indeed have reached the UK coastline, therefore the possibility of future events cannot be dismissed. In 1931 the most severe earthquake registered in the UK occurred in the North Sea near the Dogger Bank[31]. This event was used by the British Geological Survey to model the effects of a possible tsunami in the North Sea, as this is the only region that is likely affect the coastline of eastern England [British Geological Survey, 2005]. The overall conclusions from the simulations are that the probability of occurrence of a tsunami event in the North Sea is very low and that the wave height when reaching the coastline is between 0.8 and 2.0 metres, which is in the order of typical winter storm surges on the North Sea [British Geological Survey, 2005]. Hence in designing a storm surge barrier a tsunami wave will not be considered, after all storm surges are already taken into account and as a result of the very low probability of occurrence of a tsunami, the probability of a tsunami occurring at the same time as a storm surge is even lower.

2.3 Wind speed

The region between 30° N and 60° N in which the Wash estuary is situated the wind climate is predominantly dominated by strong and variable westerly winds. The wind rose depicted in figure 39 confirms this, as the wind comes for the larger part from the directional sector west to south. Although wind from the directional sector north to east is less frequent, this directional sector is of great importance since the generated wind and swell waves propagate straight into the Wash estuary and travel along the main channel (Lynn Deeps) deep into the basin. Wind from the other directions only generates internal wind waves that have typically lower wave heights and periods than the North Sea waves, as is already discussed in section 2.2.

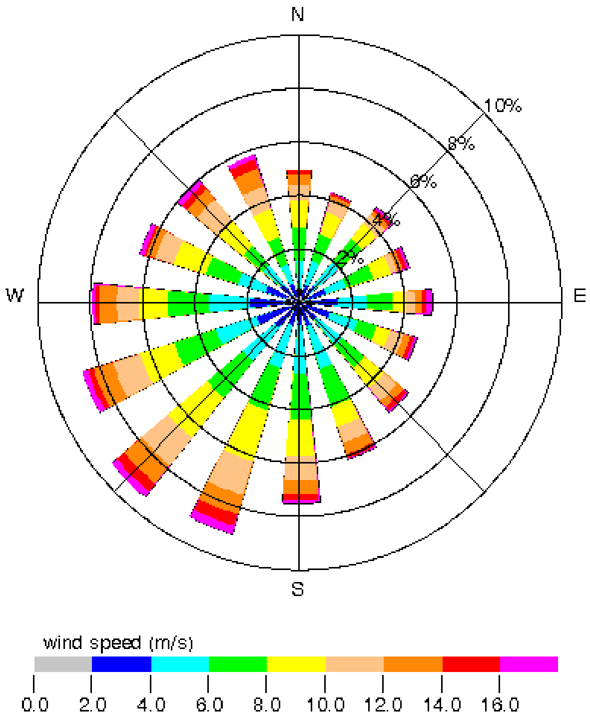


Figure 39: wind rose (Courtesy: *BMT ARGOSS*).

[31] Source: http://en.wikipedia.org/wiki/1931_Dogger_Bank_earthquake

Again the initial distribution approach is used to determine the extreme offshore wind conditions, see appendix 8. Due to the persistent character of wind and because of the low roughness of the sea surface, the nearshore wind conditions are assumed to be the same as the offshore conditions. In table 21 the extreme wind speeds presented for different storm durations and return periods.

SoP		U_s for storm duration [m/s]			
Return period [yr]	Q_s [-]	6 hrs	8 hrs	12 hrs	15 hrs
50	0.02	21.3	21.7	21.4	21.2
200	0.005	24.7	25.2	25.1	25.0
500	0.002	27.6	28.0	28.1	28.1
1000	0.001	30.2	30.5	30.8	30.8
2000	0.0005	33.1	33.2	33.7	33.8

SoP = standard of protection
U_s = extreme wind speed

Table 21: U_s for different return periods and storm durations.

2.4 River discharge

According to the River Basin Management Plan of the Anglian River Basin [Environment agency, 2009], the basin covers an area of 27,890 km^2 (see figure 40). The average slope of the downstream sections of the tidal rivers Witham, Welland, Steeping, Nene and Great Ouse is approximately 1:10,000; which is very flat. As mentioned earlier this is a consequence of the historic draining of the Fenlands, which resulted in a polder landscape where much of the land is situated at or below mean sea level.

All rivers discharge into the estuary by means of gravity flow via sluices, this is obviously only possible when the outside water level is low enough. The discharge sluices are located in Boston (river Witham), Spalding (river Welland), Dog-in-a-Doublet (river Nene) and Denver (Great Ouse), see also figure 29 in section 1.3.2.

In table 22 an overview is given regarding both the mean and peak discharge derived from the annual hydrograph by the Centre for Ecology and Hydrology. From the river Steeping the discharge is unknown as there is no measurement station present along its course.

River	Catchment area [km^2]	Mean discharge [m^3/s]	Peak discharge [m^3/s]
Great Ouse	3430	15.67	33.75
Nene	1634	9.30	23.86
Welland	717	3.76	8.76
Witham	298	1.87	3.96
Total	6079	30.6	70.33

Source: *Centre for Ecology and Hydrology*.

Table 22: mean and peak discharge derived from the annual hydrograph.

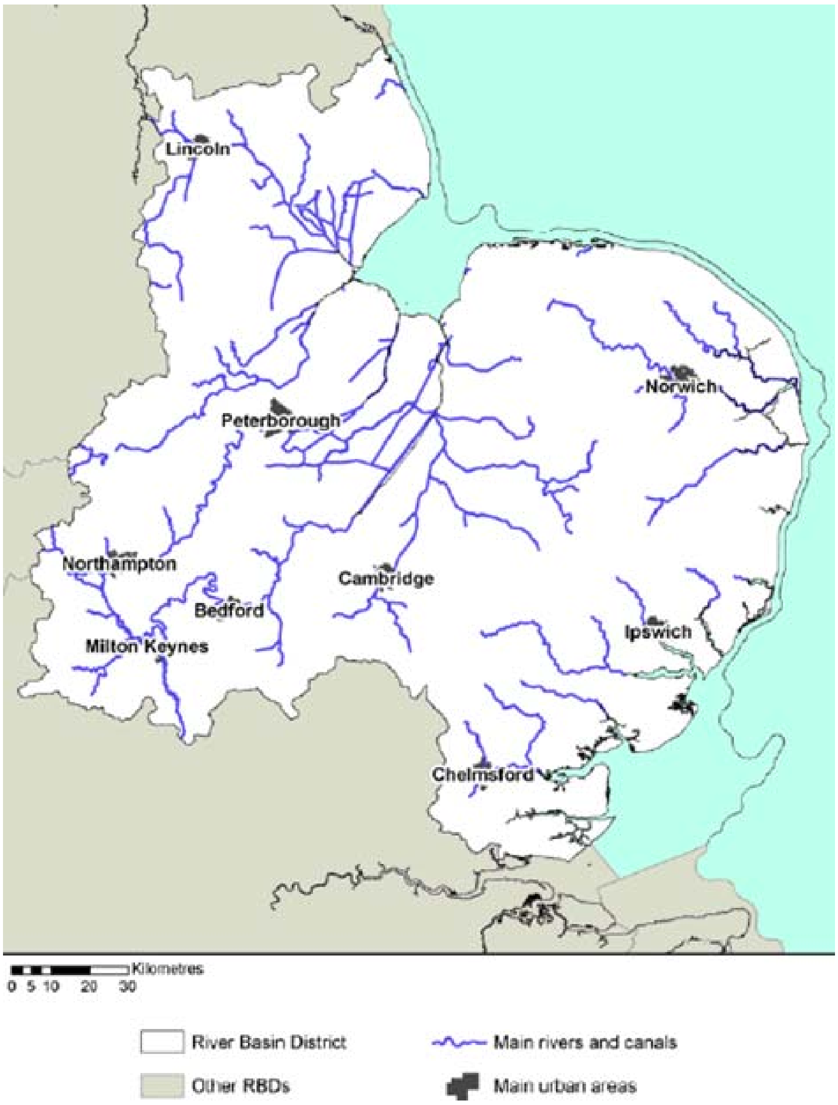


Figure 40: Anglian River basin District. (Courtesy: *Environment Agency*)

The figures in table 22 originate from the most downstream situated measurement stations, however in reality the catchment area of each river is much larger. As the annual precipitation pattern in the catchment areas is likely to be more or less the same the figures stated in table 22 have been corrected by multiplying the tabulated discharges with a factor that expresses the ratio of the real surface area over the partial surface area, see table 23 for the resulting mean and peak discharges. For the purpose of this feasibility study the approach followed is deemed acceptable.

River	Catchment area [km^2]	Mean discharge [m^3/s]	Peak discharge [m^3/s]
Great Ouse	8596	39.27	84.58
Nene	2270	12.92	33.15
Welland	1680	8.81	20.53
Witham	3000	18.83	39.87
Total	15,546	79.83	178.13

Table 23: corrected mean and peak discharge.

The observant reader distinguishes in figure 40 two smaller rivers flowing into the Wash estuary near its mouth, the river Steeping near Gibraltar Point on the western shoreline, see figure 41, and the river Heacham on the eastern shoreline near Heacham. In the followed approach the discharge of these smaller rivers is included in the discharge of the rivers Witham and Great Ouse respectively.

Figure 41: river Steeping outfall at Gibraltar Point (Courtesy: *unknown*).

When a combination of a storm surge barrier and tidal power plant is constructed in the Wash estuary, the river discharge has to be stored within the basin during the time the barrier is closed. With respect to the large surface area of the estuary the combined river discharge is not likely to have much influence on the water level. This is stressed by a quick and dirty hand calculation (ignoring precipitation in the basin itself) :

$$\Delta h = \frac{Q_p \cdot t_{closure}}{A_b} = \frac{180 \cdot 3 \cdot 24 \cdot 60 \cdot 60}{615 \cdot 10^6} \approx 0.08\ m \qquad [2.11]$$

Were:

Δh	:	increase in water level	[m]
Q_p	:	combined peak discharge from all rivers	[m^3/s]
$t_{closure}$	:	duration of the closure	[s]
A_b	:	basin area	[m^2]

Hence in case the storm surge barrier is closed for three days during times of peak discharge in the rivers, the water level in the Wash estuary increases by approximately 8 centimetres. So even in the unlikely situation that the barrier is closed when the basin is fully filled the river discharge into the estuary will pose no threat for the flood safety provided by the coastal defences. With respect to the fluvial flood defences the high water level on the basin will prevent the rivers from discharging into the estuary and may pose a threat to the hinterland as embankments can be overtopped, that is why normally spoken the barrier will be closed well before high water.

It is almost certain that the total discharge into the estuary is increased as a result of the discharge of effluent by sewage treatment plants in the region, however no figures are available. Based on the above computation it seems safe to conclude that this will not be of much influence on the estuary's water level in case a storm surge barrier is constructed.

2.5 Morphology

The sediment transport processes in the Wash estuary are dominated by alongshore currents and tidal processes, resulting in the deposition of at least 6.8 million tonnes of marine sediment per year [Ke et al, 1996]. The tidal rivers discharging into the estuary deposit a much smaller quantity, 10,000 to 100,000 tonnes per annum [Ke et al, 1996]. Much of the marine sediment originates from erosion of the Lincolnshire and North Norfolk coastline and enters the Wash estuary via alongshore transport and tidal currents. Another source is suspended sediment present in the North Sea and most probably suspended sediment originating from the Humber estuary, just north of the Wash estuary, carried by the tidal currents into the Wash estuary, see figure 42. Several towards the Wash estuary directed sand spits are recognisable along the Norfolk coastline, also the sand spit at Gibraltar Point is visible.

Figure 42: sediments within the Wash estuary, April 19th 2011. (Courtesy: *www.Eosnap.com*)

2.6 Impact of climate change

After the last ice age the rising sea level flooded the lower lands, ultimately leading to the present day coastline and the extensive development of salt and fresh water marshes. During the Iron Age the water levels peaked, fell again during the Roman era and started to rise again in early medieval times. Both natural and human induced sea level rise continued until today and is expected to do so in the future. However, there is a large uncertainty surrounding the future rate of sea level rise caused by the thermal expansion of water as a result of global warming, the melting of land ice and the natural variability in water level. The Department of Environment, Food and Rural Affairs (DEFRA) 2006 guidance provides values for relative sea level rise along the English east coast for three future time periods [Environment Agency SMP2, 2010]. In table 24 these rates of relative sea level rise for the three epochs, are presented. The UK Climate Impacts Programme published an update of its projections in 2009 (UKCP09). The rates as stated in table 24 are well within the range that UKCP09 predicts and have been prescribed by the Environment Agency in all Shoreline Management Plans. However, one should realize that there is a large uncertainty involved in establishing these figures.

Time period	Relative sea level rise [mm a year]	Total sea level rise [mm]	Cumulative sea level rise [mm]
Epoch 1 2009-2025	4	64	64
Epoch 2 2025-2055	8.5	255	319
Epoch 3			
2055-2085	12	360	679
2085-2105	15	450	1129

Source: *DEFRA sea level rise guidance for the east of England.*
Note: a vertical land movement of -0.8 mm per annum is assumed by DEFRA.

Table 24: rates of relative sea level rise used in all Shoreline Management Plans.

Relative sea level rise represents the combined effect of regional land subsidence and absolute changes in mean sea level. According to Hume et al [2002] the regional land subsidence amounts to -1.0 mm per annum in Lincolnshire and -1.2 mm per annum in Norfolk, these figures are confirmed by Buglass and Brigham [2007]. The figure used by DEFRA is somewhat smaller, -0.8 mm/yr. But for a hundred year period the difference would amount to an error of 2 to 4 cm. In view of the uncertainties of the climate change predictions, this difference can be ignored.

The land subsidence occurring behind the coastal defences is caused by the continuous drainage of the soil, resulting in land subsidence and the oxidation of peat (where present). Also the land is not regularly flooded anymore since it was reclaimed and hence no marine sediments are deposited anymore. The combination of both processes has resulted in a situation where the land on the landward side of the flood defences is often situated at a lower level compared to the intertidal area within the adjacent Wash estuary. This height difference between the intertidal area and the land protected by the flood defences is still increasing and will continue to do so in the future.

Besides sea level rise, climate change is likely to cause increased storminess (a larger occurrence and more severe storm events) as well as larger rain intensities. This will have an impact on the flood protection levels provided by the present day sea and river defences and also influence the sediment supply and morphology within the Wash estuary (Increased storminess leads to more erosion especially in case of the soft sediments present at the UK's eastern coastline, see also figure 37 in section 2.1.3). Table 25 shows indicative sensitivity ranges that are set by the Environment Agency and indicate the predicted future changes in hydraulic boundary conditions. These surcharges have to be taken into account when designing flood defences [Environment Agency, 2006].

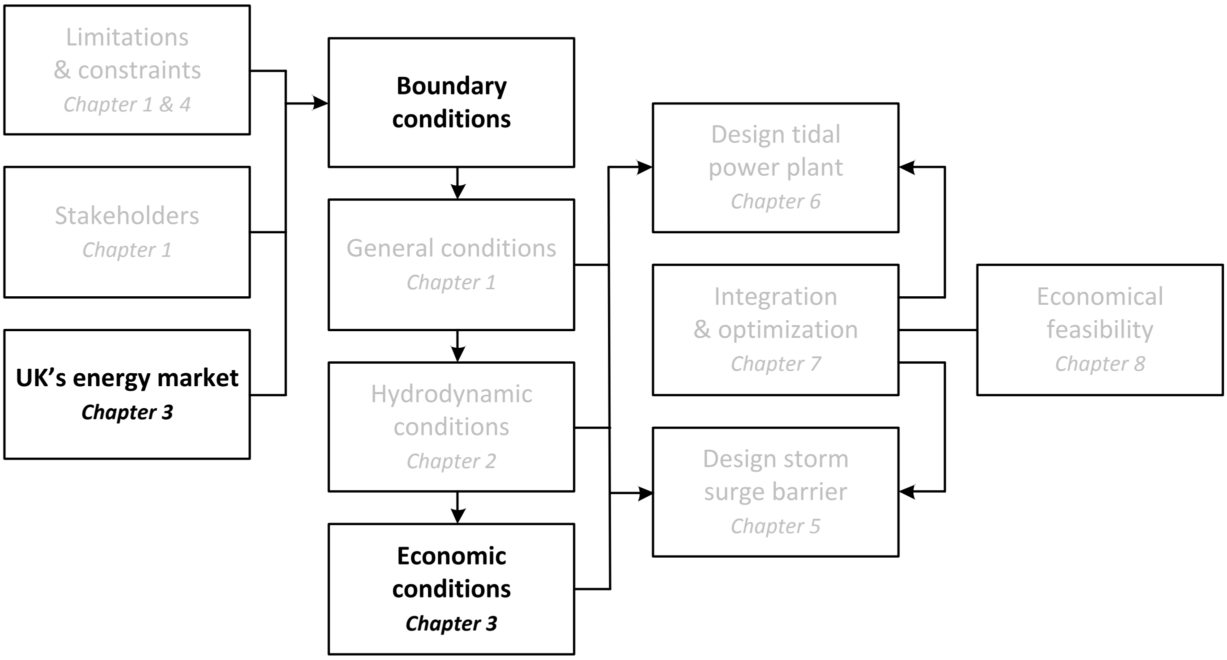
Parameter	1990-2025	2025-2055	2025-2085	2085-2115
Peak river flow	+10%	+20%		
Extreme wave height	+5%		+10%	
Offshore wind speed	+5%		+10%	

Source: *Flood and Coastal Defence Appraisal Guidance 2006* [Environment agency, 2006].

Table 25: indicative sensitivity ranges.

Figure 43 shows the area that would flood without the presence of flood defences during extreme storm surges that coincide with spring tide, such as the storm surges that occurred in 1901, 1906, 1944, 1953, 1976, 1978, 1982 and 1993. These storm surges caused widespread damage and disruption along the English east coast [ABI. 2006]. This clearly indicates the shear impact a breach in the flood defences surrounding the Wash estuary might have.

Figure 43: area flooded during a 1:200 yr storm surge in absence of flood defences.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary conditions
General conditions
Chapter 1
Hydrodynamic conditions
Chapter 2
Economic conditions
Chapter 3
Design tidal power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm surge barrier
Chapter 5
Economical feasibility
Chapter 8

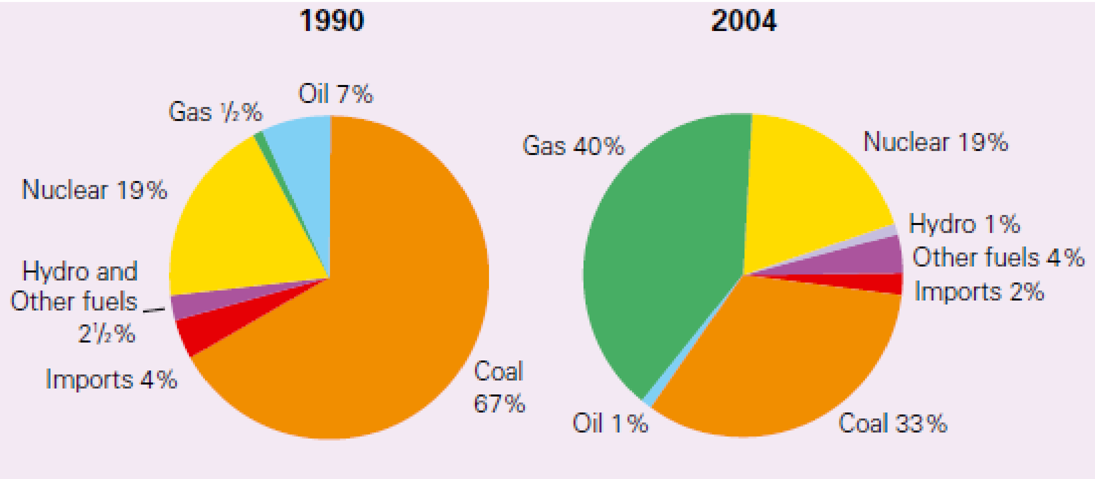
3 UK'S ENERGY MARKET

The long term vision of the UK Government regarding the country's energy supply is that the 2050 climate change objectives[32] must be achieved, while ensuring secure and affordable energy supplies. In order to achieve these goals, the current energy market, that is heavily dependent on fossil fuels, must be reformed towards a low carbon energy market. This means that renewable energy sources (solar energy, wind energy, water power, etc.), nuclear energy and fossil fuel combined with carbon capture and storage are bound to get a larger market share on the expense of traditional coal and gas fired electricity generation. Within this light the UK Government has committed that 15% of its total energy consumption comes from renewable sources by 2020, which means that approximately 30% of the UK's electricity generation should be provided by renewable energy sources [HM Treasury, 2010]. This creates opportunities for the generation of tidal energy in the UK. However, the focus of the Government seems to be more on onshore and offshore wind energy schemes.

In this chapter first the historical developments of the UK's energy market will be sketched. Next the short, medium and long term objectives of the current energy policy will be treated, followed by an overview of the most important European and national policies that must enable the achievement of these objectives. Last but not least the current level of energy prices in the UK energy market is explored.

3.1 Short history

Before the 1960's 90% of the energy production in the UK was provided by coal fired energy plants, the remaining 10% was provided mostly by oil fuelled electricity production. During the 1960's until the 1980's, nuclear energy gradually got foothold within the UK's energy market. By the late 1990's nuclear power plants provided 26% of the national electricity supply [Redpoint, 2010]. Since then no new nuclear power plants have been commissioned and its market share declined due to the retirement of the first generation power plants. Within the light of the Government's low carbon energy policy, plans exist to build new nuclear power plants in the near future.



Source: *DTI, 2005*.

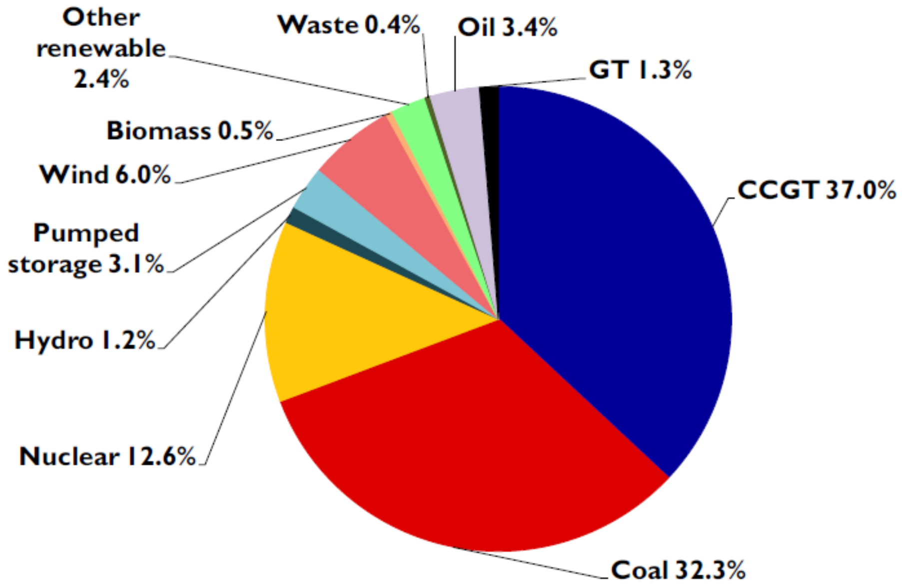
Figure 44: energy mix by fuel type, 1990 vs. 2004.

[32] The UK Government has committed to a legally binding target to cut greenhouse gas emissions by 80 %, from 1990 levels, by 2050 [HM Treasury, 2010].

Besides one of the most reliable supplies in Europe, the UK's energy market is also one of the most liberalised energy markets in the world. As a result of this liberalization the gas market was opened up [Redpoint, 2010] on the expense of the coal fired power plants. This so-called "dash for gas" started in 1993 and continued until the early 2000's, see also figure 44. At the same time of the energy market liberalization a Non-Fossil Fuel Obligation was introduced, which remained the primary renewable support scheme until it was replaced by the Renewables Obligation in 2002 [Redpoint, 2010].

Before 1990 the only renewable energy source of some scale in the UK was hydro power, predominantly situated in Scotland. Since the mid 1990's a steady increase in renewable energy production capacity is noticeable, mainly in the form of landfill gas and biomass fired power plants. From the mid 2000's a significant growth can be seen with respect to wind farms, as a result of which nowadays wind energy is the second largest renewable energy source in the UK. These wind farms are for the larger part land based, however since 2009 also large offshore wind farms have been installed. The construction of offshore wind farms is expected to speed up in the coming years as onshore and offshore wind energy play a key role in reaching the 2020 target [DECC, 2010].

Marine energy in the form of wave power and tidal stream power are also a priority for the UK Government, since the UK coast has large potential and because the Government strives to develop a new world leading UK based energy sector [DECC, 2010]. In spite of the fact that already since the 1920's the feasibility of tidal barrages is studied, no tidal range power plant was ever built. According to the Sustainable Development Commission the reasons for this are mainly the high capital costs and, more recent, environmental concerns.



CCGT = Combined Cycle Gas Turbine[33]
GT = Conventional Gas Turbine

Figure 45: UK's electricity generation capacity (Courtesy: *Redpoint estimates, 2010*).

[33] Combined Cycle Gas Turbine = the turbine's generator generates electricity and heat in the exhaust is used to make steam, which in turn drives a steam turbine that generates additional electricity.
Conventional Gas Turbine = turbine in which electricity is generated and the heated gasses are exhausted to the atmosphere. Many old gas-fired electricity plants are of this type.

As can be seen in figure 45; nowadays natural gas and coal are still the primary energy sources in the UK (71%), followed by nuclear energy (13%). All renewable energy sources together have a market share of only 14%.

3.2 Objectives for energy policy

On the short term (2020) the security of energy supply is still guaranteed in the UK. But in order to meet de carbon emission reduction targets, the contribution of renewable and low carbon energy sources to the energy mix must increase considerably, see figure 46.

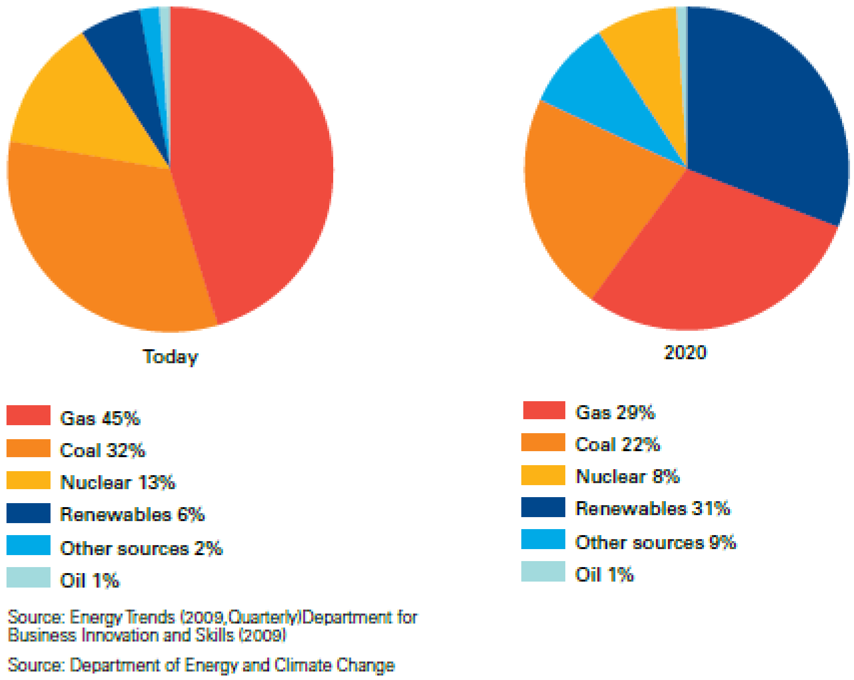


Figure 46: energy mix by fuel type, 2009 vs. 2020.

The main objectives concerning the energy policy of the UK Government on the short, medium and long term are listed below. For a description of all spear heads of the UK energy policy, the reader is referred to appendix 10.

Main short term policy objectives:

- large investments in onshore and offshore wind energy in order to meet the 2020 climate change objectives;
- becoming a world leader in the low carbon and environmental sector.

The main medium term (2020-2050) policy objectives are:

- large investments in nuclear power, fossil fuel generation with Carbon Capture and Storage (CCS) and renewable energy sources (mainly wind energy), in order to maintain security of supply;
- diversification of energy sources;
- low carbon energy resources have to replace fossil fuels in both transport and domestic heating;
- becoming a world leader in the low carbon and environmental sector.

On the long term (2050 and beyond) the main policy objectives are:
- to have ensured a secure, clean and affordable energy supply through a independently regulated and competitive energy market [HM Treasury, 2010];
- continuing the medium term objectives.

The energy mix in 2050 will be characterized by large contributions from wind energy, nuclear power and fossil fuel power in combination with CCS. Potentially important contributions may be made from other renewable energy sources. The use of oil will further decline, however gas will remain a important energy source.

Regarding the policy objectives for the short, medium and long term it can be concluded that Government supported development of tidal range power plants is most likely to occur in the medium and long term, depending on the development of the global energy demand. On the short term all effort is directed to the construction of onshore and offshore wind farms, tidal range power plants are not considered to be an option due to their environmental impacts. However this does not mean that a tidal range power plant is not technical or economical feasible. Depending on developments on the global energy market and the availability of fossil fuels, in the long term using the vast tidal range energy potential may become important in sustaining the way of life in the UK and therefore the economical benefits may be overshadowing environmental interests.

3.3 Current energy policy

In this section the UK's Government's strategy to reach the 2050 climate change objectives will be treated, starting with European policy that forms one of the central pillars under the UK's energy policy. Next the national Renewable Energy Strategy will be discussed as this strategy is of importance in the framework of this thesis. Other important policies, like the Household Energy Efficiency, Climate Change Levy and Carbon Capture and Storage Incentive, though important in reaching the UK's climate change objectives, are not directly related to this thesis's subject and therefore will not be discussed.

3.3.1 European policy

One of the major pillars under the European Climate Policy is the European Union Emissions Trading System (EU ETS). In principle the system comes down to putting a price on carbon emissions, see appendix 10 for a more elaborate treatment.

Year	Carbon price [£/ tonne CO_2]
2010	14.10
2020	16.30
2030	70.00
2040	135.00

Source: *Mott MacDonald, 2010.*

Table 26: carbon prices.

The idea behind the EU ETS is that large sources of carbon dioxide, like heavy industry and electricity generating plants, are encouraged to reduce their emissions or trade emissions. The trading results in a carbon price, see table 26, and hence ensures that throughout the system emissions cuts are made there where they are cheapest. With respect to the electricity market this ensures on the longer term that producing electricity from high carbon sources will be replaced by low carbon sources.

3.2.2 National policy

The UK Government is planning to reach its 2050 climate changes objectives through a combination of regulatory and financial measures, which are:

- the Renewables Obligation Order, which requires 30% of the UK's electricity to be generated from renewable energy sources by 2020;
- Feed-in-Tariffs for small scale renewable energy generation (up to 5 MW). This are fixed prices that are not linked to the wholesale market prices and provide a high level of security for investors not traditionally involved in the production of electricity.

Renewables Obligation Order

The Renewables Obligation (RO) primarily focuses on large scale renewable electricity generation by energy companies. Licensed electricity suppliers are obliged to source an increasing proportion of their annual sales from renewable energy or pay a penalty. Different electricity generators are issued Renewables Obligation Certificates (ROCs) for each MWh of eligible renewable electricity they produce [DECC, 2010]. Different technologies receive different numbers of ROCs, thus taking into account differences in technology costs. See table 27 for some characteristic values.

Electricity generation type	ROCs per MWh
Hydro-electric	1
Onshore wind	1
Offshore wind	1.5
Wave	2
Tidal stream	2
Tidal barrage	2
Tidal lagoon	2
Standard gasification	1
Advanced gasification	2
Dedicated biomass	1.5

ROCs = Renewables Obligation Certificates.
Source: *DECC, 2010.*

Table 27: differentiation of ROCs by technology.

As said the RO requires electricity suppliers to source at least part of their electricity from renewable energy generators. These obligation levels are set annually by the Department of Energy and Climate Change (DECC), see table 28 for an overview of past and future obligation levels.

The electricity generators can sell their ROCs to electricity suppliers or traders in order to receive a premium on top of their electricity price. When an electricity supplier does not have acquired enough ROCs proportionate to the electricity that was sold, a penalty has to be paid, the so-called buy-out price. This price is annually updated by the Office for Gas and

Electricity Markets (Ofgem), in the base year 2002-2003 the buy-out price was £30/MWh, in 2009-2010 £37.19 and it will be £36.99 in the year 2010-2011[34].

Obligation period	Obligation level [ROCs/MWh]
2002-2003	3.0
2009-2010	9.7
2010-2011	11.1
2011-2012	12.4
2012-2013	15.8

Source: *website DECC.*

Table 28: Renewables Obligation Certificates per MWh.

Since the introduction of the Renewables Obligation the amount of renewable energy generated had been tripled, see figure 47. Because the UK's main electricity network is located close to the Wash estuary and the networks ability to exploit tidal power is deemed large, see figure 10.6 in appendix 10, the RO may offer opportunities. Despite the fact the RO is not specifically meant for this purpose a electricity generator company may be interested to participate in a tidal range scheme in the Wash estuary.

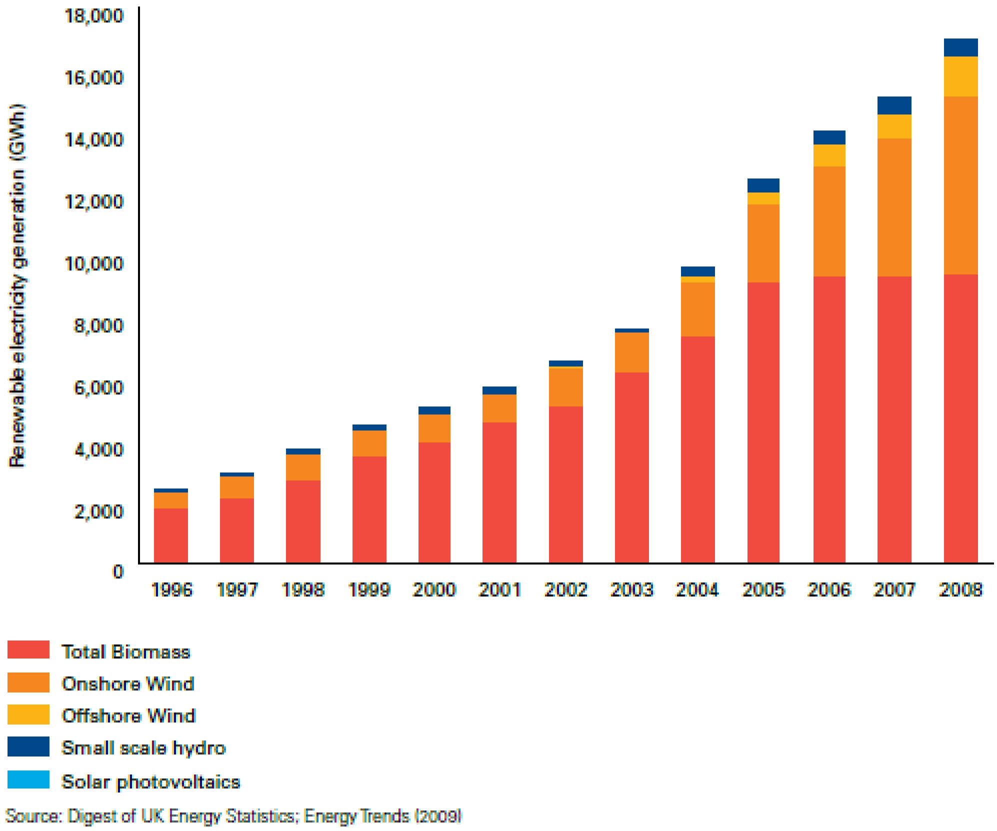


Figure 47: growth in renewable energy generation from 1996 to 2008.

[34] Source: Ofgem information note on the Renewables Obligation buy-out price.

Feed-in-Tariffs
The Feed-in-Tariffs (FIT) are meant to support eligible small scale low carbon electricity technologies financially. The scheme supports projects up to a 5 MW limit by requiring electricity suppliers to pay generation tariffs to the owners of the scheme, based on the number of kWh they generate. In case a surplus of energy is available and this surplus is exported to the electricity network a guaranteed additional export tariff of 3 p/kWh is to be paid by the electricity supplier [Energy Trends, 2011]. The FIT support:
- new anaerobic digestion schemes;
- solar photovoltaic schemes;
- hydro schemes;
- wind schemes.

The present target groups are individual households, organisations, communities and businesses not traditionally engaged in the electricity market, but the new Government has proposed to introduce a FIT for renewable electricity schemes with a generation capacity larger than 5 MW [DECC, 2010]. So this may be an interesting development regarding the economic feasibility of a tidal power plant in the Wash estuary.

3.4 Current UK energy prices

The energy prices in this section are given as the average lifetime levelised energy generating costs (LEC). The LEC represents the price at which a specific source should generate energy in order to break even. As shown in the equation below the LEC is computed as the ratio of the net present value of the total of construction, operating and maintenance costs during the economic lifetime over the net present value of net electricity generation during the economic lifetime:

$$LEC = \frac{\sum_{t=1}^{n} \frac{I_t + M_t + F_t}{(1+r)^t}}{\sum_{t=1}^{n} \frac{E_t}{(1+r)^t}} \qquad [3.1]$$

Were:

LEC	:	average lifetime levelised electricity generation costs	[£/MWh]
I_t	:	capital costs in year t	[£]
M_t	:	fixed operating and maintenance costs in year t	[£]
F_t	:	variable operating and maintenance costs in year t	[£]
E_t	:	net electricity generation in year t	[MWh]
r	:	discount rate[1]	[-]
n	:	economic lifetime of power plant	[yr]

1) At present a discount rate of 10% is advised by DECC, source: Mott MacDonald 2010 and Parsons Brinkerhoff 2010.

The variable operating and maintenance costs include forecasted changes in carbon and fuel prices, which are likely to increase the LEC of high carbon emission power plants in the future. On the other hand nuclear and renewable energy sources are very likely to benefit from these developments as they do not emit carbon dioxide and do not rely on fossil fuels, hence there operational costs will be relatively low compared to those of high carbon energy schemes and thus these techniques become more competitive. However the drawbacks of renewable energy are first of all the fact that these schemes require high upfront investments and therefore tend to be more sensitive with respect to future uncertainty in the electricity prices and secondly that most of these technologies are still at the beginning of their learning curve, see appendix 10.

In 2010 both Mott MacDonald and Parsons Brinkerhoff published figures on LEC in the UK. In both studies fuel and carbon emission costs are included, as is a 10% discount rate. The prices in both studies are based on cost data of recent tender contacts [Mott MacDonald 2010 and Parsons Brinkerhoff 2010]. The results of both studies need careful interpretation as they are based on cost estimates and not the actual costs after construction, but it is believed that these figures are accurate enough to determine the economic feasibility of a tidal power plant in the Wash estuary.

In the table 29 the results of both studies are presented. In the Parsons Brinkerhoff study the stated price per kWh is based on the assumption that the electricity is delivered at the power plant's high voltage grid connection. This is done in order to exclude current uncertainties concerning transmission costs due to the geographical distribution of generating types. Because different scenarios with respect to future developments in fossil fuel and carbon prices were regarded in the study, cost ranges are defined.

In the Mott MacDonald study the transmission costs are included, which may lead to a skewed comparison as at any one location the transmission costs may differ considerably. On the other hand the transmission costs are an important cost factor. The Mott MacDonald study adopts the central projections, made by DECC, for both the future fuel and carbon price developments.

Technology	LEC range [1) [p/kWh]	LEC [2) [p/kWh]
Natural gas turbine, no CO_2 capture	5.5-11	8
Natural gas turbine, with CO_2 capture	6-13	11.3
Coal, with CO_2 capture	10-15.5	14.2
New nuclear energy	8-10.5	9.9
Onshore wind farm	8-11	9.4
Offshore wind farm	15-21	16.1
Tidal range power (Severn estuary)	15.5-39	-

[1) Source: *Parsons & Brinkerhoff 2010.*

[2) Source: *Mott MacDonald 2010.*

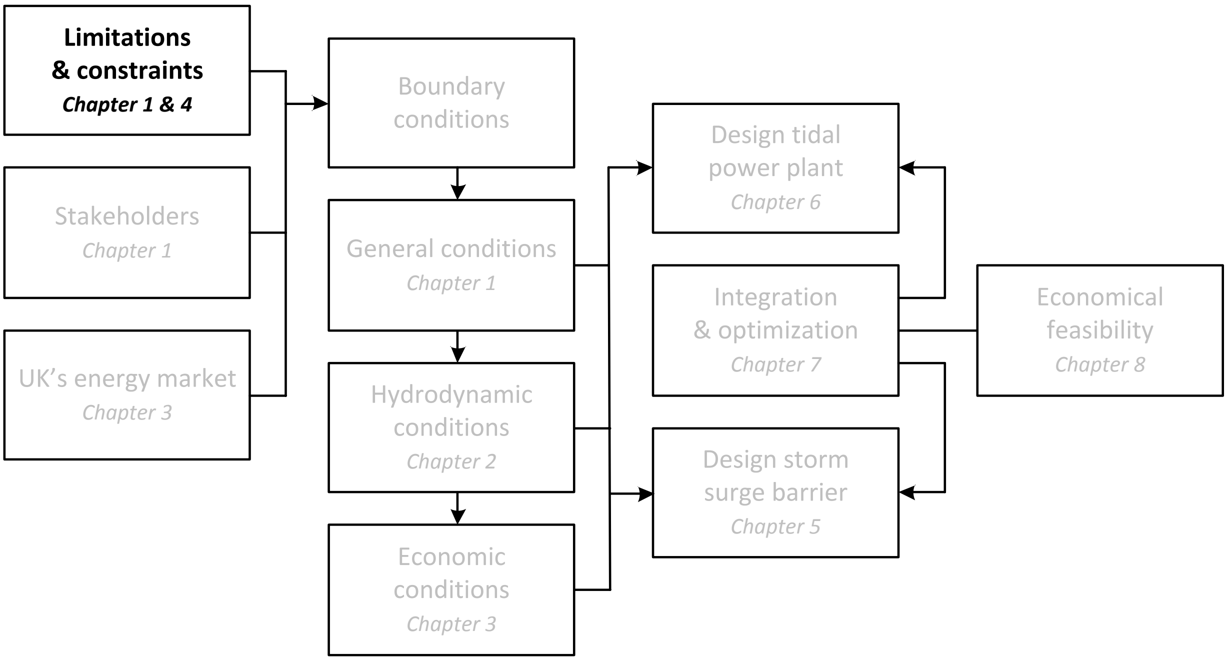
Table 29: UK energy LEC for different generation technologies.

It is to be expected that both studies should lead to more or less the same results as they are both based on the same data and development scenarios and also that the Mott MacDonald figures should be close to the middle of the ranges as defined by Parsons and Brinkerhoff, as Mott MacDonald used central projections for both fuel and carbon prices. Comparison of the figures learns that, although the Mott MacDonald figures lie within the cost ranges found by Parsons and Brinkerhoff, they lie more close to the upper and lower boundaries of the price ranges. Possible explanations for the difference are included in appendix 10.

The Parsons and Brinkerhoff figures will be used to determine the economical feasibility of a tidal power plant within the Wash estuary because:

- the figures include also high and low projections for future fossil fuel and carbon prices, therefore taking into account uncertainties regarding the future developments on the global energy market;
- the transmission costs are excluded, making the comparison between technologies more fair.

From this chapter can be concluded that in order to be competitive with other low carbon energy sources the cost of the electricity generated by a tidal power plant in the Wash estuary should lie within a price range of 8-11 p/kWh.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

4 TERMS OF REFERENCE

The main goal of this project is improving the flood safety in the hinterland bordering the Wash estuary by means of increasing the Standard of Protection of the flood defences. Since the project is a private initiative revenues have to be generated in order for the project to be profitable, this is where a tidal power plant enters the picture. In this chapter the preconditions and Terms of Reference applicable to the feasibility phase of this project, combining a storm surge barrier with a tidal power plant of some sort, are stated.

In order to maintain an overview it is decided to split the project in two parts, namely: the storm surge barrier and the tidal power plant. Of course this initial separation does not imply that these parts can be regarded as separate projects, they are indeed intertwined. Therefore during the design stages the relationship between both structures and the consequences of design decisions for the other component of the project will be kept in mind at all times. Both structures are therefore designed in conjunction. First the preconditions applying to both structures are described, after which for each structure the Terms of Reference will be stated.

4.1 Preconditions

The preconditions primarily relate to the main economical sectors providing the larger part of the regional inhabitants with their livelihoods. Of course the heavily protected natural environment within the Wash estuary itself also plays a large role in providing preconditions.

4.1.1 Natural environment

Preconditions related to the natural environment are:

- the structure should have as little impact as possible on the flora and fauna present within the estuary. This means that the currently present morphological and hydrodynamic conditions that result in the estuary’s unique dynamic environment must be affected as little as possible;
- the structure must be integrated in the landscape as much as possible;
- the visual impact on the seascape must be kept to a minimum;
- the large Common Seal colony must keep its present resting and breeding grounds and also must be able to travel to and from their hunting grounds in the North Sea (P.M., falls outside the scope of this study);
- legally required mitigation measures (P.M., falls outside the scope of this study).

4.1.2 Economical sectors

Preconditions related to the main economical sectors in the region are:

<u>*Tourist industry:*</u>

- recreational ship traffic must be able to reach the North Sea, most vessels sail close to the coast and leave the estuary near Hunstanton. Therefore a small navigation lock is to be constructed near Hunstanton [Wash Tidal Barrier Corporation plc, 2009];
- the bathing water must remain of good quality;
- maintain the present seascape as much as possible and integrate the structures as much as possible in the landscape.

Fishing industry:
- the commercial exploited mussel and cockle beds must be kept intact;
- since brown shrimp is of great commercial importance the fishing is to be as much as possible spared. On the other hand no measures will be taken with respect to pink shrimp fishery as this branch is of minor commercial importance an mainly takes place in the deeper waters of the North sea (>10 m).

Commerial shipping:
- it must remain possible for commercial ships (coasters) to travel between the ports and the North Sea, therefore a deep water navigation lock is to be constructed with a capacity suited to accommodate 1800 ship movements per annum [Wash Tidal Barrier Corporation plc, 2009].

4.1.3 Other preconditions

Preconditions related to other activities and interests in the Wash estuary and adjacent Fenlands are stated below.

Militairy activities:
- the Holbeach weapons training range, located at the landward end of the basin is kept operational;
- it is very likely to find unexploded ordnance and remnants of exploded ordnance on the former Wainfleet weapons training range just of the coast near Gibraltar point, see figure 48.

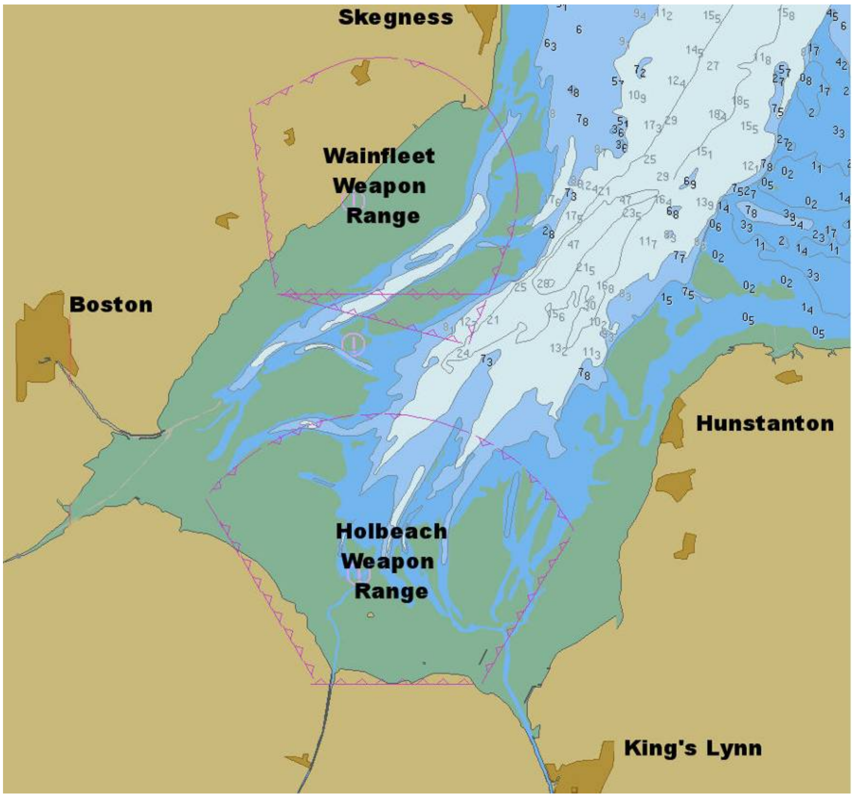


Figure 48: (former) military activities in the Wash estuary (Courtesy: OpenCPN).

Historic environment

- the Roman, medieval and post-medieval sea defences must be kept intact as they are part of the very few principal areas in the UK with surviving historic flood defence field monuments;
- monumental buildings present in the Fenlands bordering the Wash estuary must be kept intact. The main concentrations are found just south of Skegness and on high ground along the eastern shoreline.

Submarine infrastructure

- the grid connection cable of the Linc offshore wind farm passes through the Wash estuary, see also figure 23 in section 1.2.10 (P.M. for the construction phase).

4.2 Terms of Reference

As stated before the Terms of Reference are stated separately for the storm surge barrier and the tidal power plant, but both structures are designed in conjunction.

4.2.1 Storm surge barrier

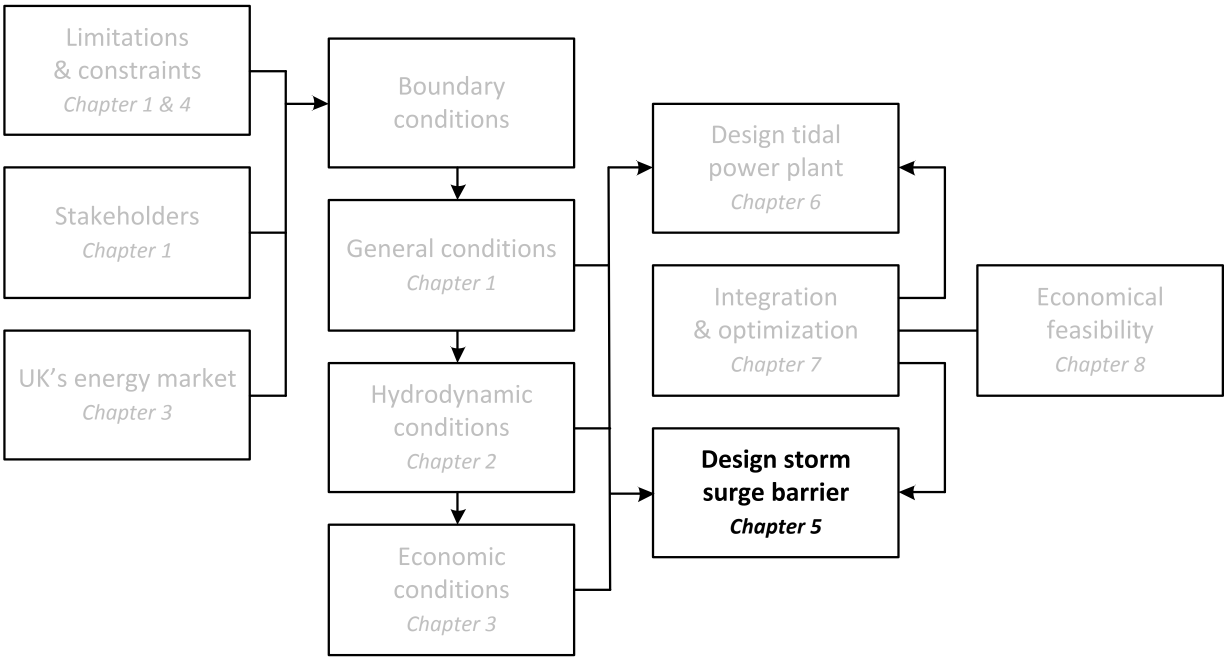
The Terms of Reference with respect to the storm surge barrier are:

- the Standard of Protection offered to the hinterland must be increased to 1:500 year [Wash Tidal Barrier Corporation plc, 2009];
- the design lifetime of the structure is 120 years and during that time 5% damage is allowed;
- due to the absence of any demand of transportation between Hunstanton and Skegness, no road or rail connection is required atop the structure. Except for a service road for access and maintenance [Wash Tidal Barrier Corporation plc, 2009];
- the possible impacts with respect to flood safety and coastal erosion along the adjacent Lincolnshire and Norfolk coastlines, resulting from the construction of a storm surge barrier across the Wash estuary, must be mitigated [P.M.];
- river discharge must be guaranteed at all times;
- wave overtopping is allowed;
- the design height must take into account a surcharge for SLR of 1.30 m;
- the design conditions as defined in chapter 2 must be increased to compensate for the effect of climate change. Effectively this means that:
    - the peak river discharge is to be increased by 20%;
    - the offshore wind speed is to be increased by 10%;
    - the extreme wave height is to be increased by 10%;
    - the peak period is to be increased by 5%.

4.2.2 Tidal power plant

With respect to the tidal power plant the Terms of Reference are:

- the design lifetime of the structural components from the power house and the sluices must be 120 years;
- the energy generating costs must lie within the range of 8-11 p/kWh, which is competitive with respect to other low carbon energy sources, based on an design lifetime of 120 years and including the costs of the storm surge barrier (construction costs, operation and maintenance costs and refurbishment costs);
- the turbines should be fish friendly.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

5 STORM SURGE BARRIER

There are many different types of storm surge barriers which can be divided into two main groups, namely movable barriers and permanent barriers. When the length of the barrier becomes long, usually a combination of a movable and permanent barrier is built.

Examples of movable barriers are:

- vertical lift gates (Eastern Scheldt storm surge barrier);
- vertical axis sector gates (Maeslant storm surge barrier, see figure 49);
- horizontal axis sector gates (tainter gates);
- rising sector gates (Thames storm surge barrier, see figure 50);
- visor gates;
- flap gates (Venice flood barrier);
- inflatable barriers (Ramspol, see figure 51);

Figure 49: Maeslant storm surge barrier (NL).
(Courtesy: *Het Keringhuis*)

Figure 50: Thames barrier (UK).
(Courtesy: *Andy Roberts, 2004*)

Figure 51: inflatable barrier near Ramspol (NL).
(Courtesy: *Delta Marine Consultants*)

Figure 52: Brouwersdam (NL).
(Courtesy: *FTT Procesontwikkeling*)

Permanent barriers consist of (armoured) embankments (see figure 52), from which the core consists of rock, soil, caissons or a combination of the three. In case of a barrier across the Wash estuary sand is readily available near the barrage site, although the flow velocity is probably too high for a sand closure. The nearest rock quarries are located in Northumberland. For the Great Yarmouth East Port project the stone was imported from Scandinavia [Johansen, 2008], as this turned out to be cheaper for that specific project. Wave overtopping should be limited in case of an embankment dam in order to prevent the need of armouring the complete inner slope.

In the remainder of this chapter the considered alternatives are described, as are the design considerations. At the end of the chapter the most suitable alternative is determined.

5.1 Zero alternative

The zero alternative is the alternative to which all other alternatives will be compared too. According to governing policy the current management strategy is hold the line, which means that the current Standard of Protection (SoP =1:200) is maintained and therefore only the effects of climate change are compensated for[35]. The effects of climate change are a relative sea level rise of 1.30 m (over the design lifetime of the storm surge barrier) and an increase of Hs and Tp of 10% and 5% respectively[36]. Hence, after taking into account the predicted effects of climate change, the design offshore significant wave height becomes 5.50 m and the corresponding peak period 14.7 s.

In figure 53 the average cross-section of the earthen embankments surrounding the Wash estuary is shown, the embankments are grass covered and do not have a revetment. It is assumed that the embankments consist of clay with a grass cover [Google Earth, 2011]. The average storm surge level is 6.00 mODN.

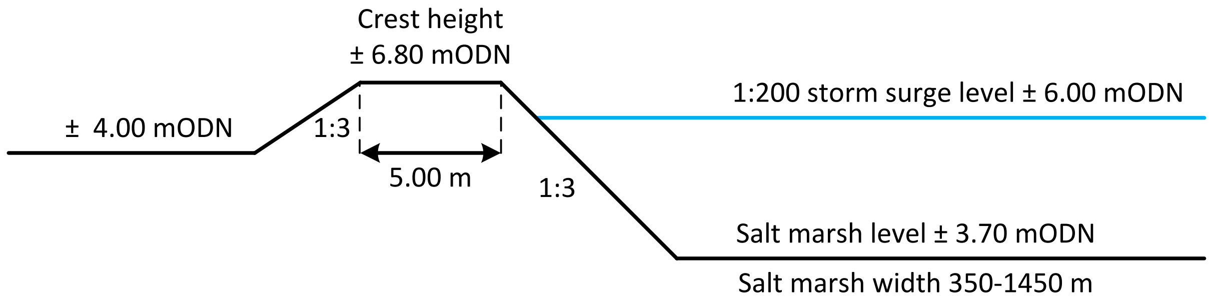


Figure 53: average cross-section of the flood defences bordering the Wash estuary.

As can be seen in table 31 on page 113, the increase of saltmarsh height compensates for part of the expected relative sea level rise. This is taken into account by increasing the height of the bed level at the seaward side of the levees with a value: design life time *times* average increase in saltmarsh height over all four epochs. Next the new water depth is used to determine the design wave height at the toe of the structure, using the breaking limit of waves in shallow water (based on the breaking limit of Miche and breaking of the larger waves 1.5-2·Hs):

[35] Along the western shoreline of the Wash estuary there are a few short stretches with SoP 1:50 present, these are thought to be included via the law of large numbers.
[36] See section 2.6, tables 28 and 29.

$$\frac{H_s}{h} \approx 0.4 - 06 \qquad [5.1]$$

Were:

H_s	:	near shore significant wave height	[m]
h	:	water depth	[m]

In case 0.6·h > $Hs_{\text{deep water}}$ the near shore significant wave height is taken equal to the offshore significant wave height, otherwise the value of 0.6 times the water depth is taken as the near shore significant wave height.

The peak period is computed using Brettsneider, after which the crest level above design still water level is computed using a overtopping discharge of 1 l/s/m[37] and taking into account settlement of the old embankment and subsoil, using Koppejan's equation. Furthermore 10% settlement of the new material is considered to occur.

$$\varepsilon = U \cdot \left(\frac{1}{C_p} + \frac{1}{C_s} \cdot \log\left(\frac{t}{t_1}\right)\right) \cdot \ln\left(\frac{\sigma}{\sigma_1}\right) \quad \text{and} \quad \Delta Z = \varepsilon \cdot d \qquad [5.2]$$

Were:

ε	:	relative compression	[-]
U	:	degree of consolidation (U = 1)	[-]
C_p	:	primary compression coefficient (C_p = 30)	[-]
C_s	:	secondary compression coefficient (C_s = 400)	[-]
t	:	time period considered (120 yr)	[day]
t_1	:	reference time period (t_1 = 1 day)	[day]
σ	:	new effective stress	[kN/m^2]
σ_1	:	initial effective stress	[kN/m^2]
ΔZ	:	settlement during life time	[m]
d	:	layer thickness	[m]

The values used for both the primary and secondary compression coefficients correspond to weak sandy firm clay, according to table 1 of the National Annex to EN 1997-2 Eurocode 7: Geotechnical Design.

After computing three alternatives (landward, seaward or central reinforcement, see figure 54) of adapting the present day levees to the expected effects of climate change, it turned out to be cheapest to reinforce the levees at the landward side. Except for the section from Wolferton Creek to Hunstanton, where the presence of buildings required a seaward reinforcement of the levees.

[37] Since no data are available on the quality of both the clay and the grass cover , the overtopping discharge is set to 1 l/s/m. This is regarded to be a conservative value, especially since overtopping tests in The Netherlands on a real dike in 2007 have shown that in that specific case 30 l/s/m was acceptable on a slope consisting of a good quality clay covered by a good quality grass cover [Eurotop, 2007].

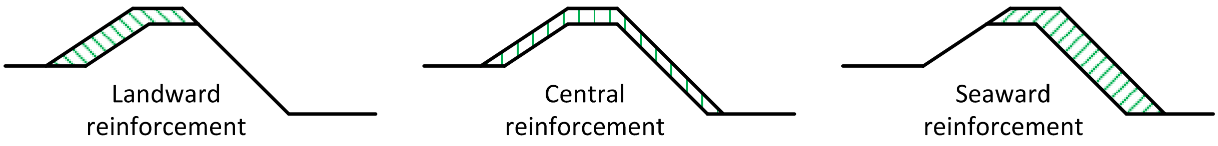


Figure 54: landward, seaward and central reinforcement.

Tabel 32 shows the length of the coastal defences per section, while in table 33 an overview is presented of the expected cost of adapting the coastal defences to the expected effects of climate change. This ballpark cost estimate is based on unit costs for earthen embankments and revetments as stated in appendix F of The Wash SMP2 [Environment Agency, 2010], which are computed using the Environment Agency's Flood Risk Management Estimation Guide – Unit cost database[38].

For an earthen embankment the unit cost are based on the largest project size, a slope of 1:3 and a crest width of 5 m, resulting in a ballpark figure of £ 24 per m^3 fill, excluding compensation for landowners. With respect to revetments the same assumptions are made, however it is also assumed that approximately 5 m of the outer slope requires protection and that the thickness of the revetment structure is approximately 1.5 m. This results in a ballpark figure of £ 27 per m^3 fill. The range of the unit cost is plus or minus 30% [Environment Agency, 2010, Appendix F]. In the performed computation these ballpark figures are updated to a 2012 price level using an average real interest rate[39] of 4%, see table 30.

$$C_{2012} = C_{2006} \cdot (1 + r)^n \qquad [5.3]$$

Were:

C_{2012}	:	price level in 2012	[£/m^3]
C_{2006}	:	price level in 2006	[£/m^3]
r	:	real interest rate	[-]
n	:	number of years	[-]

Unit cost	2006 price level [£/m^3]	2012 price level [£/m^3]
Embankment	24	30
Revetment	27	34

Table 30: indexation 2006 ballpark unit cost.

[38] 2006 price level
[39] Real interest rate = nominal interest rate – inflation.

Increase in saltmarsh height							
From	**To**	**Epoch 1**	**Epoch 2**	**Epoch 3**	**Epoch 4**	**Average**	
Gibraltar Point	Horse shoe	7	7	7	7	7	[mm/yr]
Horse shoe	North bank of Haven	7	7	7	7	7	[mm/yr]
North bank of Haven	Eastern bank of Nene	4	4	4	4	4	[mm/yr]
Eastern bank of Nene	Eastern bank of Wolferton Creek	7	7	7	7	7	[mm/yr]
Eastern bank of Wolferton Creek	Hunstanton	0	0	0	0	0	[mm/yr]

Source: The Wash SMP2, appendix F, 2010.

Table 31: average increase of saltmarsh height.

Length of coastal defences				
From	**To**	**Revetment**	**Earthen embankment**	
Gibraltar Point	Horse shoe	0	14000	[m]
Horse shoe	North bank of Haven	0	14200	[m]
North bank of Haven	Eastern bank of Nene	0	39000	[m]
Eastern bank of Nene	Eastern bank of Wolferton Creek	0	22000	[m]
Eastern bank of Wolferton Creek	Hunstanton	3800	7250	[m]
	Total	3800	96450	[m]

Table 32: length of coastal defences.

Cost of adapting coastal defence			
From	**To**	**Cost**	
Gibraltar Point	Horse shoe	76.80	[10^6 £]
Horse shoe	North bank of Haven	77.90	[10^6 £]
North bank of Haven	Eastern bank of Nene	234.38	[10^6 £]
Eastern bank of Nene	Eastern bank of Wolferton Creek	120.69	[10^6 £]
Eastern bank of Wolferton Creek	Hunstanton	76.24	[10^6 £]
	Total cost	586.01	[10^6 £]

Table 33: cost of adapting the coastal defences (SoP 1:200)

Figure 55: coastal defences zero alternative.

The river levees are assumed to have 1:3 slopes and a freeboard of 1.00 m above the maximum tidal level in mODN, see figure 56. Again the majority of the levees consist of grass covered clay embankments without revetments. However in the city centres often quay walls are present, it is assumed that these quay walls consist of sheet piling .

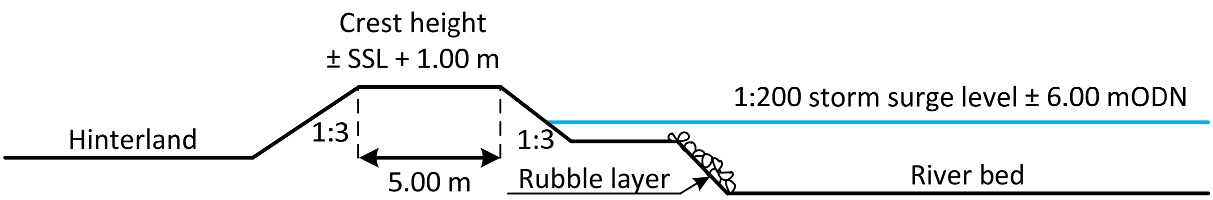


Figure 56: average cross-section of the river defences bordering the tidal rivers.

Only the effects of climate change are compensated for. Since no data are available of the present day and future upstream water levels, it is assumed that the levees have to be raised 1.30 m, which corresponds to the expected sea level rise. This is regarded as a worst-case scenario and is to be expected to result in an overestimation of the cost. Both settlement of the subsoil and of the new material are taken into account the same way as was done for the coastal defences.

The ballpark costs for raising a quay wall can be obtained as follows[40]:

$$I_{quay\ wall} = F_{index} \cdot I_{sheet\ pile} + I_{building} \qquad [5.4]$$

Were:

$I_{quay\ wall}$	:	characteristic value for raising a quay wall	[£/m^2]
F_{index}	:	factor representing indexation of cost (1.5)	[-]
$I_{sheet\ pile}$	:	characteristic value for lengthening a sheet pile wall	[£/m^2]
$I_{building}$	:	characteristic value for adapting adjacent buildings	[£/m^2]

Using a rule of thumb stating that the retaining height corresponds to approximately ⅓ of the total required sheet pile length, means that for raising a quay wall 1.00 m the sheet piles should be lengthened 3.00 m. Taking the median value of the price range provided by Ir. van der Toorn and using an exchange rate of € 1.00 equals £ 0.83[41] results in a ball park figure for raising a quay wall 1.00 m of:

$$I_{quay\ wall} = 1.5 \cdot 3 \cdot (1350 \cdot 0.83) + (10{,}000 \cdot 0.83) \approx 13350\ \text{£/m}^2$$

In table 34 the length and type of the tidal river defences are presented, while table 35 gives an overview of the estimated cost of adapting the tidal river defences.

Length of riverine defences				
River	**Total length**	**Quay wall**	**Earthen embankments**	
Steeping	8000	0	8000	[m]
Witham	17000	6460	10540	[m]
Welland	21000	0	21000	[m]
Nene	77000	4620	72380	[m]
Great Ouse	50000	1500	48500	[m]
Total	173000	12580	160420	[m]

Table 34: length of riverine defences.

[40] According to Ir. A. van der Toorn, lecturer at Delft University of Technology, a rough characteristic value for raising a quay wall consisting of sheet piling, lies between € 1200 – 1500 per m height per running meter. Mr. van der Toorn advises to use a factor 1.5 for indexation purposes. Also Mr. van der Toorn advises to take into account additional cost for the required adaptations to adjacent buildings, summing up to € 10,000 per meter height per running meter.
[41] Source: www.wisselkoersen.nl , exchange rate on February 8th 2012.

Cost of adapting tidal river defences		
River	**Cost**	
Steeping	9.43	[10^6 £]
Witham	127.25	[10^6 £]
Welland	24.76	[10^6 £]
Nene	184.12	[10^6 £]
Great Ouse	111.89	[10^6 £]
Total cost	457.45	[10^6 £]

Table 35: cost of adapting the river defences (SoP 1:200).

The total estimated cost for the zero alternative sum up to £ 1043M.

5.2 Alternative 1: enhance current flood defences to SoP 1:500

In alternative 1 the standard of protection (SoP) is raised to 1:500, which is the same SoP as a future storm surge barrier would be designed for. This alternative is used to compare the estimated cost between constructing a storm surge barrier across the mouth of the Wash estuary (including a tidal power plant) and the required adaptation of both the existing coastal and riverine defences. The reason for the comparison is that most of the levees surrounding the estuary are situated in an agricultural area with very few buildings and because the levees do not have to be as high as a future storm surge barrier since much of the incoming wave energy is already dissipated by the saltmarshes and sand and mud flats before reaching the levee.

The approach followed, is the same as was used for the zero alternative, only now the design still water level, excluding relative sea level rise, is raised to 6.20 mODN (corresponding to the average 1:500 storm surge level). Table 36 shows the estimated cost for adapting the coastal defences to the expected consequences of climate change, while table 37 gives an overview of estimated costs for raising the riverine defences.

Cost of adapting coastal defence			
From	**To**	**Cost**	
Gibraltar Point	Horse shoe	86.64	[10^6 £]
Horse shoe	North bank of Haven	87.88	[10^6 £]
North bank of Haven	Eastern bank of Nene	262.83	[10^6 £]
Eastern bank of Nene	Eastern bank of Wolferton Creek	136.15	[10^6 £]
Eastern bank of Wolferton Creek	Hunstanton	83.66	[10^6 £]
	Total cost	657.16	[10^6 £]

Table 36: cost of adapting the coastal defences (SoP 1:500).

Cost of adapting tidal river defences		
River	**Cost**	
Steeping	10.00	[10^6 £]
Witham	128.04	[10^6 £]
Welland	26.26	[10^6 £]
Nene	189.53	[10^6 £]
Great Ouse	115.84	[10^6 £]
Total cost	469.67	[10^6 £]

Table 37: cost of adapting the river defences (SoP 1:500).

The total estimated cost for alternative 1 sum up to £ 1127M, which is £ 84M more than the zero alternative. Again the amount does not include compensation for landowners.

5.3 Alternative 2: storm surge barrier

Alternative 2 investigates the construction cost of a storm surge barrier with a Standard of Protection (SoP) of 1:500 across the Wash estuary. As was already mentioned earlier in the introduction to this chapter, three types of storm surge barriers are distinguished. Of which the following two are considered in the underlying case:

1) a combination of a movable and permanent barrier, alternative 2a;
2) a permanent barrier, also known as a closure dam, alternative 2b.

Since the Wash estuary is characterized by a large intertidal area, approximately 290 km^2, a complete movable barrier is not considered as vast shallow areas inevitably have to be crossed. In which case a permanent barrier is far more cost effective.

5.3.1 Barrier line

In order to be able to select the most suitable barrier line, first the most important influence factors have to be identified. Determining factors for the positioning and construction of a storm surge barrier are:

- the water depth across the barrier line. For a permanent barrier the water depth determines the volume of the dam body per m^1 barrier, which should be as small as possible as the amount of construction material needed increases quadratic with depth. Also the volume has a large effect on costs and building time. In case of a movable barrier also both construction cost and building time increase with water depth;
- the length of the dam;
- the position and depth of the main channels and the location of shallow areas;
- the flow velocities in the main channels;
- the type of material of which the seabed consists;
- the presence of natural support points like e.g. rock outcrops cliff faces etc..

The geometry of the Wash estuary basin is such that both the width of the basin as a whole and the width of the main channels does not vary much (10-15%). The main channels penetrate deep into the estuary and the inter tidal flats are primarily orientated parallel to these channels, see figure 57. However the depth of the Lynn Deeps decreases considerably from the mouth of the estuary towards the landward side of the basin. The deepest part (-50 mODN = -47 m CD) of the Lynn deeps forms a sort of underwater canyon that is situated across the mouth of the estuary over a length of approximately 8.5 km, see figure 57. As was already stated in section 2.1.5 the maximum depth average flow velocities in the main channels are in the order of 1.00 m/s.

The seabed material is not a determining factor in the underlying case, as the bottom deposits in the whole basin consist of sand and gravel (appendix 1). Also the distribution of intertidal sediment is fairly constant over the estuary, see figure 37 in section 2.1.5. In the area of interest with respect to finding a suitable barrier line, the inter tidal flats consist of sand, while the saltmarshes have a more muddy character. The only natural support point within the estuary are the Hunstanton cliffs, however presently these are eroding. Therefore the cliff base should be stabilized in order to use the cliffs as a support point. Current management policy is to let the undercutting process and the resulting cliff erosion proceed.

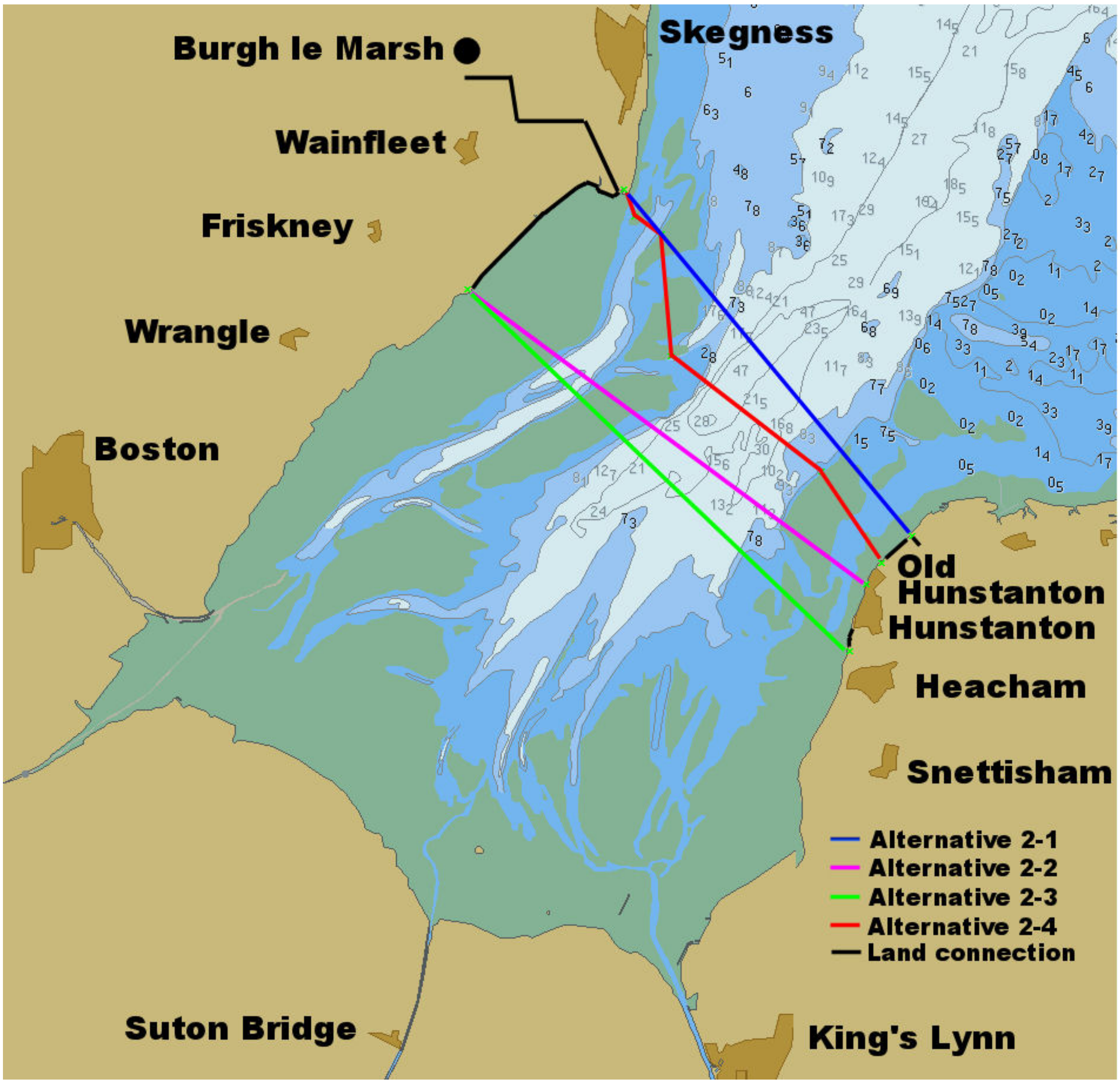


Figure 57: alternative barrier lines (depth in m-CD).

Other more specific preconditions to the Wash estuary, as formulated in chapter 4, are:

- the location of the high country in Lincolnshire at the western shoreline;
- the presence of commercial shellfish beds, see appendix 3;
- the brown shrimp fishery in the channels in between the main channels;
- commercial shipping and pleasure boating;
- presence of unexploded ordnance and remnants of exploded ordnance on the intertidal flats at the former Wainfleet weapenrange, see chapter 4 figure 48.

Taking into account the above mentioned criteria four alternative barrier lines were selected, see figure 57. Alternative 2-1 represents the shortest possible barrier line, from Gibraltar Point to Old Hunstanton. The second alternative (2-2) intersects the Lynn Deeps perpendicularly in order to shorten the crossing of the deepest part as much as possible. Alternative 2-3 crosses the Lynn Deeps further into the estuary were the depth is significantly less. Finally alternative 2-4 crosses the Lynn Deeps perpendicularly at the narrowest section. Furthermore the Boston Deeps are intersected perpendicularly, while the interconnecting stretches follow the inter tidal flats and shallows as much as possible.

The black lines in figure 57 represent the part of the barrier line situated on land. The distance between the Wash estuary's western shoreline and the higher grounds in Lincolnshire is shortest near Burgh le Marsh, further south the Fenlands become considerably wider over a very short distance, see figure 6 in section 1.1.2. All alternatives involve constructing a 10 km long embankment from Gibraltar Point to Burgh le Marsh. The routing is such that large caravan parks and hamlets present in the area are avoided, as are the historically important buildings. Alternatives 2-2 and 2-3 require additional strengthening of circa 8 km existing embankment. Along the eastern shoreline the high grounds continue for the larger part within the proximity of the coastline. Alternative 2-1 involves the strengthening of approximately 1 km of dunes, while alternative 2-3 involves the strengthening of 2 km of embankment/ seawall.

In Figure 58 the depth profiles along the alternative barrier lines are depicted. Since both water depth and barrier length are, in case of the Wash estuary, the most governing factors, these parameters will be used to determine the most suited alternative. From table 38 could be concluded that alternative 2-1 is most suited, however this alternative also crosses the deepest part of the Lynn Deeps (which is not taken into account in the comparison).

The longest crossing of the estuary is approximately 15% longer than the shortest crossing, while the longest total length differs approximately 40% with the shortest route. As the main difference in length originates from the length of the land based section of the barrier, which is relatively cheap to construct, it does not seem logical to give the distance criterion much weight in the decision process.

Barrier line alternative	From	To	Shore based length [m]	Estuary length [m]	Total length [m]
2-1	Gibraltar Point	Old Hunstanton	11150	18650	29800
2-2	Friskney	Hunstanton Cliffs	18200	21383	39583
2-3	Friskney	Heacham	20200	21586	41786
2-4	Gibraltar Point	Old Hunstanton	10150	20047	30197

Table 38: length of the alternative barrier lines.

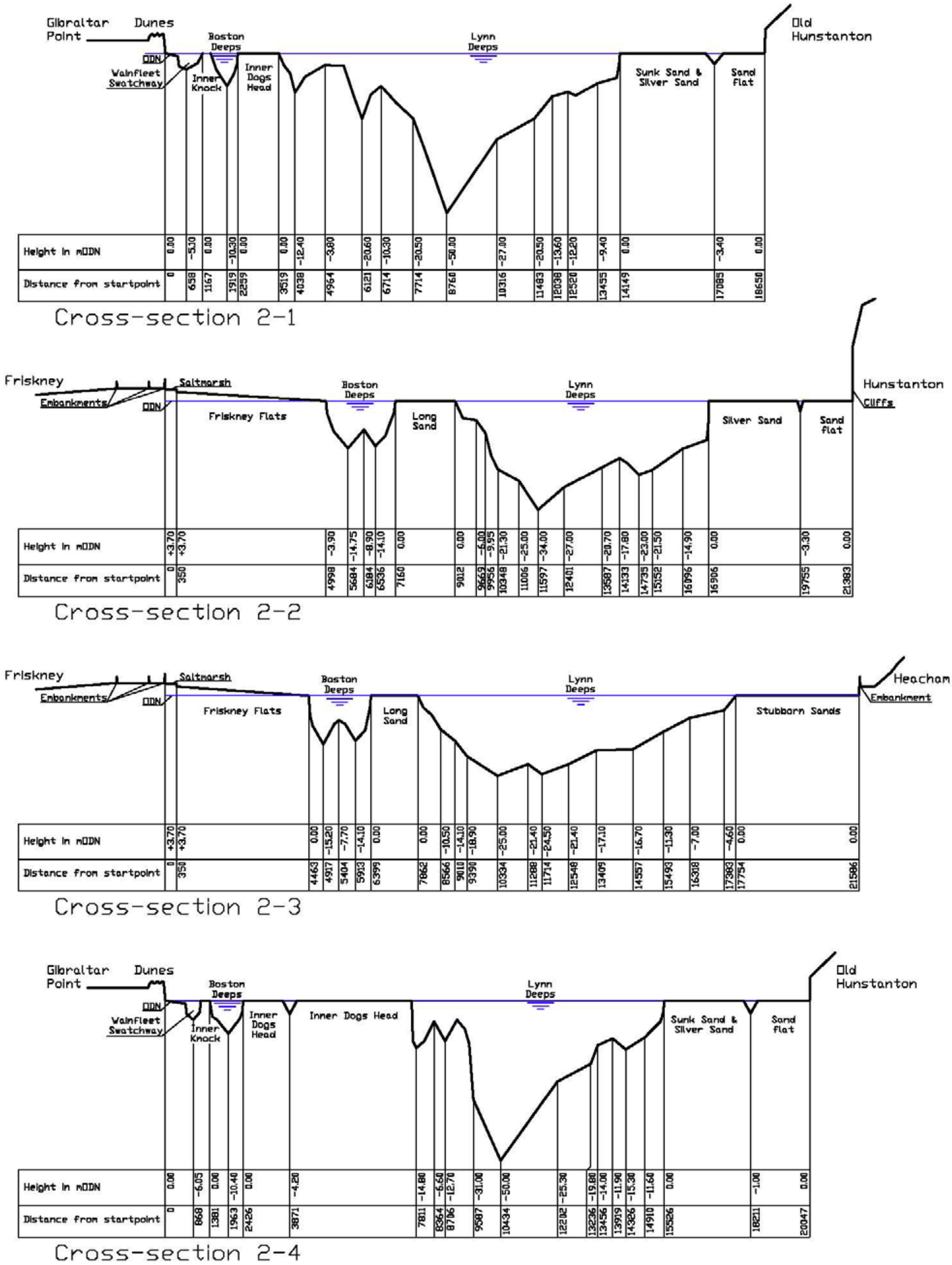


Figure 58: cross-sections barrier line alternatives (Not to scale).

Since the volume/height, and therefore the cost, of the barrier is closely related to the water depth it seems more appropriate to use this parameter as the basis for the determination of the most suitable barrier line alternative. Because of the different character of a movable barrier compared to a permanent barrier two different approaches are followed. In case of the combined barrier the weighted water depth is used, while regarding a permanent barrier the total volume needed to construct the barrier is determined.

Barrier line alternative	From	To	Weighted water depth [mODN]	Maximum water depth [mODN]	Width Lynn Deeps [m]
2-1	Gibraltar Point	Old Hunstanton	-11.37	-50.00	8606
2-2	Friskney	Hunstanton Cliffs	-8.37	-34.00	7894
2-3	Friskney	Heacham	-8.26	-25.00	9862
2-4	Gibraltar Point	Old Hunstanton	-9.77	-50.00	7715

Table 39: weighted water depth.

Table 39 shows that alternatives 2-1 and 2-4 intersect on average the deepest parts of the Wash estuary, as was to be expected based on the cross-sections depicted in figure 58. The 1.60 m difference in weighted depth between both alternatives is explained by the fact that in alternative 2-4 the barrier line intersects both the Boston and Lynn Deeps perpendicularly and that the interconnecting sections follow the shallows as much as possible. Clearly either alternative 2-2 or 2-3 will be the most suited barrier line, depth wise. Overall alternative 2-3 is shallower, but on the other hand the section crossing the Lynn Deeps is approximately 2 km longer and also the basin behind the barrier will be several km^2 smaller. This will result in less revenues from the energy generated by the tidal power plant, see table 40.

Barrier line alternative	Basin area high tide [km^2]	Basin area low tide [km^2]	Average basin area [km^2]	Annual energy potential [1)] [GWh]
2-1	615	325	470	20,480
2-2	500	265	380	16,560
2-3	465	245	355	15,470
2-4	580	305	440	19,175

The density of water is taken to be 1025 kg/m^3, the mean tidal rang is taken 4.70 m.
[1)] The annual energy potential is computed using equation 2.8.

Table 40: annual energy potential.

Hence regarding water depth and basin size alternative 2-2 is selected to be the most suited barrier line for a combined barrier. In case of a permanent barrier alternative 2-3 is most suited, since that alternative requires significantly less construction material (see table 41).

Barrier line alternative	From	To	Total volume [10^6 m3]
2-1	Gibraltar Point	Old Hunstanton	71.90
2-2	Friskney	Hunstanton Cliffs	62.25
2-3	Friskney	Heacham	60.95
2-4	Gibraltar Point	Old Hunstanton	71.60

Table 41: total volume permanent barrier.

Other important factors to consider are:

- the fact that alternatives 2-2 and 2-3 cross the former Wainfleet weapons training range (see figure 48 in chapter 4), were it is very likely to find unexploded ordnance and remnants of exploded ordnance, which will increase the construction costs;
- the fact that alternatives 2-2 and 2-3 have to cross the river Steeping, for which measures are required to guarantee the discharge and with respect to the marina.

Although the factors mentioned above contribute to the construction costs, the associated costs are not determined within the scope of this report.

5.3.2 Alternative 2a: combination of movable and permanent barrier

Because the mouth of the Wash estuary is approximately 20 km wide and large parts are relatively shallow (depth < 10 m), while the main channel is very deep (depth > 50 m), a combination of a permanent barrier on the shallow parts and movable sections across the main channels seems an obvious solution. However as a result of the fact that the barrier has a dual function, the movable parts of the barrier have to be opened and closed frequently and within 15 minutes [Clark, 2007]. The aforementioned movable barriers are not suited for such a purpose as the opening and closing operation takes longer, furthermore these structures are intended for low frequency use and are quite expensive, see table 42.

Barrier	**Year**	**Cost [10^6 €]**	**Cost 2012 [4)] [10^6 €]**	**Length [m]**	**Retaining height [m]**	**Maximum head [m]**
Eastern Scheldt barrier [1)] *Vertical lift gates (60)*	1986	1136	3149.5	2400	14	5
Hartel barrier [1)] *Vertical lift gates (2)*	1991	140	319	170	9.3	5.5
Nakdong estuary barrier [1)] *Vertical lift gates (2)*	2010	125	135.2	200	10	2
Seabrook barrier [2)] *Vertical lift & sector gates*	2012	114.7	114.7	130	8	4
Eastern Scheldt barrier [3)] *including dam sections*	1986	2723	7550	8000	14	5

1) Source: Toorn, van der, 2010
2) Source: Jonkman, 2012
3) Source: Meurs, 1984; total building costs 6 billion guilders, exchange rate € 1.00 equals fl. 2.20371.
4) Price level 2012, using equation 5.3 and a real interest rate of 4%.

Table 42: construction cost movable barriers.

The cost mentioned in the table above concern only the movable parts of the barrier. Except for the last line, were also the permanent sections are included for the Eastern Scheldt barrier in The Netherlands. Comparing the total construction cost of the Eastern Scheldt barrier to the construction cost of the movable sections only, shows that the movable barrier sections are relatively more expensive compared to the permanent barrier sections, as 42% of the total cost is allocated to 30% of the barrier length.

Since the construction costs of a barrier are strongly correlated to the retaining height and length of the barrier and also the maximum head over the structure, it is possible to derive a characteristic value for each of the barriers mentioned in table 42, using equation 5.5.

$$C_{new} = I_{ref} \cdot L_{new} \cdot h_{new} \cdot H_{\max, new} \quad [5.5]$$

Where:

C_{new}	:	cost new barrier design	[£]
I_{ref}	:	characteristic value reference design	[£/m^3]
L_{new}	:	length of the new barrier	[m]
h_{new}	:	retaining height of the new barrier	[m]
$H_{max, new}$	:	maximum head over the new barrier	[m]

Next the spread of the individual characteristic values is assessed in order to determine whether or not an average characteristic value can be used to derive a rough first estimate of the construction cost of a combined barrier in the Wash estuary, based on the best estimates of the main dimensions of the Wash barrier. In table 43 the individual characteristic values are presented.

Barrier	**Cost 2012 [10^6 £ [1)]]**	**L_{ref} [m]**	**H_{ref} [m]**	**$H_{max, ref}$ [m]**	**I_{ref} [£/m^3]**
Eastern Scheldt barrier	2614	2400	14	5	15,560
Eastern Scheldt barrier[2)]	7550	8000	14	5	13,485
Hartel barrier	264.8	170	9.3	5.5	30,455
Nakdong estuary barrier	112.2	200	10	2	28,050
Seabrook barrier	92.2	130	8	4	22,165

[1)] Source: www.wisselkoersen.nl , exchange rate on February 8th 2012: € 1.00 equals £ 0.83.
[2)] Including permanent barrier sections.

Table 43: characteristic values for several reference designs.

The average characteristic value, regarding the moving barriers only, amounts to 24,000 £/m^3. The characteristic values of the Hartel, Nakdong and Seabrook barriers are in reasonable agreement with this average value. The characteristic value of the Eastern Scheldt barrier on the other hand is significantly lower (± 16,000 £/m^3). A possible explanation for this lower value is the fact that the Eastern Scheldt barrier consists of 60 gates, which leads to more cost effectiveness due to economic scale benefits and the repetitive character of the construction works. Computing the average characteristic value for the Hartel, Nakdong and Seabrook barriers only, results in 27,000 £/m^3. This is in better agreement with the individual characteristic values of these barriers. Hence one could argue that, due to the repetitive character of the Eastern Scheldt barrier, it should not be included. However a future barrier in the Wash estuary will resemble the Eastern Scheldt barrier much closer than the other three barriers. So why not base the first rough cost estimate of a future Wash barrier solely on the characteristic value of the Eastern Scheldt barrier? The main reason for not following this approach is the fact that by using several barriers, site specific characteristics are averaged out, which in turn results in a much more reliable average characteristic value.

Since the movable section of a future Wash barrier will have a length in the order of 8 km and the permanent section will have a length in the order of 13 km, it is certain that construction works will have a repetitive character and that economics of scale will apply. Therefore a characteristic value of 27,000 £/m^3 will certainly result in an overestimation of the construction costs. Even using 24,000 £/m^3 will most likely lead to excessively high construction costs, because economics of scale and the repetitive character of the construction works do not apply to the Hartel, Nakdong and Seabrook barriers. On the other hand a characteristic value of 14,000 £/m^3 will most certainly lead to an underestimation of construction cost, as a result of the following facts:

- the ratio between movable and permanent sections of the Eastern Scheldt barrier is 0.3, while for the Wash barrier this ratio is 0.4. Hence the Wash estuary will be more costly, as a movable section is more expensive to build;
- there is no averaging out of the site specific characteristics of the Eastern Scheldt barrier.

Using a characteristic value of 16,000 £/m^3 still does not tackle the problem regarding the site specific characteristics. In reality the characteristic value will lie somewhere in the range between 16,000 £/m^3 and 24,000 £/m^{3}and most probably more close to the lower bound than to the upper bound. Therefore it is decided to increase the lower bound by 20%, resulting in a characteristic value of 19,000 £/m^3.

In this stage the best estimates of the Wash barrier length, retaining height and maximum head are 21 km, 11.70 m and 7.40 m respectively. The retaining height is taken as the crest level of the permanent sections of the barrier above MSL, while for the maximum head difference it is assumed that at the sea side of the barrier the design conditions are present (7.40 mODN) and that the water level in the basin corresponds to the water level after turbining for an ebb generation scheme (0.00 mODN). Hence, using equation 5.5, a first rough cost estimate for a combined barrier across the Wash estuary amounts to:

$$C_{Wash\ \ barrier} = 19{,}000 \cdot 21{,}000 \cdot 11.70 \cdot 7.40 = \text{£ } 34.55 \cdot 10^9$$

This amount has to be raised with the cost of the land based part of the barrier line which are found using equation 5.6:

$$C_{land\ \ based} = I_{fill} \cdot \sum_{i=1}^{N} L_i \cdot A_i \qquad [5.6]$$

Where:

$C_{land\ based}$	:	cost land based section barrier line	[£]
I_{fill}	:	characteristic value fill material	[£/m^3]
L_i	:	length of levee section i	[m]
A_i	:	area of fill levee section i	[m^2]

The land based section of the Wash barrier consists of reinforcing approximately 8 km of existing levees bordering the estuary and approximately 10.2 km compartment dam to connect the shoreline to high grounds near Burgh le Marsh. The typical cross-sections are already depicted in figure 53 in section 5.1. The cost of the land based section is estimated to be:

$$C_{land\ \ based} = 30 \cdot [8000 \cdot 206 + 10200 \cdot 300] = \text{£ } 141 \text{ M}$$

Hence the total cost estimation for a future combination of a movable and permanent barrier across the Wash estuary amounts to £ $34.69 \cdot 10^9$. Compensation for land owners is not included.

As it turns out, constructing a combined barrier is much more expensive than raising the existing levees. The main benefit of the movable sections, compared to a permanent barrier, is that ship traffic can pass the barrier site freely under normal conditions, hence there is no need for a navigation lock. However this advantage is offset by the fact that the storm surge barrier is to be combined with a tidal power plant.

5.3.3 Alternative 2b: permanent barrier

Since a future storm surge barrier across the Wash estuary is to be combined with a tidal power plant, a permanent barrier may turn out to be more cost effective. Mainly because in the previous section it is shown that the movable sections of a storm surge barrier are generally more expensive than the permanent sections. However it should be kept in mind that the movable parts are situated at the deepest sections of the barrier line. As the volume of a permanent barrier increases quadratic with depth, it may be better to place caissons on a sill across the main channels of the estuary. This option is discussed in chapter 7.

In order to be able to determine the preliminary cross-section of a permanent barrier, first the design still water levels at both sides of the barrier have to be assumed. At the North Sea side the 1:500 storm surge level is 6.11 mODN (including 2.00 m storm surge) at the mouth of the estuary, including a relative sea level rise of 1.30 m over the design life time of the structure the design still water level (SWL) is 7.40 mODN. The offshore significant wave height is 5.5 m and the corresponding peak period is 14.7 s.

Along the Wash estuary the current SoP of the levees is 1:200. Hence, at the basin side the design SWL is found by taking the 1:200 storm surge level (5.91 mODN), subtracting the storm surge at the North Sea, as this is blocked by the storm surge barrier, and adding the required compensation for relative sea level rise and wind set-up generated in the basin itself. The wind set-up for a rectangular basin in an equilibrium state can be approximated as follows:

$$S = \frac{F}{2} \cdot C_2 \cdot \frac{u_{10}^2}{g \cdot d} \qquad [5.7]$$

Where:

S	:	total wind set-up	[m]
F	:	fetch	[m]
C_2	:	constant, approximately $4 \cdot 10^{-6}$	[-]
u_{10}	:	wind speed at 10 m	[m/s]
g	:	gravitational acceleration	[m/s^2]
d	:	average water depth	[m]

Using equation 5.7 the maximum possible wind set-up in the basin is found to be 0.60 m[42]. Hence, the design SWL at the basin side of the storm surge barrier amounts to 5.80 mODN. Using Brettschneider's equations (equations 2.9 and 2.10) the significant wave height of

[42] U_{10} = 34 m/s, F = 25 km and an average water depth of 10.

waves generated within the basin itself is 2.50 m and the corresponding peak period amounts to 6 s.

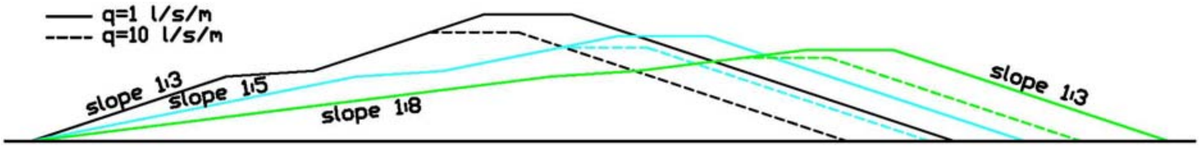


Figure 59: relation crest height and overtopping discharge.

Unlike the existing levees, from which no data are available on both the quality of the grass cover and the clay used, an allowable average overtopping discharge of 10 l/s/m is assumed for the design of the storm surge barrier. According to table 44 this overtopping discharge is an upper limit. However, tests on a real dike in 2007 in The Netherlands have shown that for the tested dike 30 l/m/s was acceptable on the rear slope consisting of a good quality grass cover on top of good quality clay [Eurotop, 2007]. Therefore an average overtopping discharge of 10 l/s/m can still be regarded as conservative, while at the same time a significant reduction in crest height is achieved with respect to an average overtopping discharge of 1 l/s/m, see figure 59.

Hazard type and reason	Mean discharge (q) [l/s/m]
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50-200
No damage to crest and rear face of grass covered embankment of clay	1-10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

Taken from the Overtopping manual [EurOtop, 2007].

Table 44: limits for overtopping with respect to damage to the crest and rear slope.

The relation between the average overtopping discharge and the required crest height above design SWL is given by the following empirical relation [EurOtop, 2007].

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \cdot \frac{R_C}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \qquad [5.8]$$

$$\text{with a maximum of } \frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \cdot \frac{R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right) \quad \text{, for } \xi_{m-1,0} < 5$$

Where:

q	:	average overtopping discharge	[m^3/s/m]
g	:	gravitational acceleration	[m/s^2]
H_{m0}	:	significant wave height	[m]
$\tan\alpha$	:	slope	[-]
$\xi_{m-1,0}$	:	breaker parameter	[-]
R_C	:	crest height above design SWL	[m]
γ_b	:	influence factor for a berm	[-]
γ_f	:	influence factor for surface roughness on a slope	[-]
γ_β	:	influence factor for oblique wave attack	[-]
γ_v	:	influence factor for a vertical wall on top of a levee	[-]

Manipulation of equation 5.8 results in an empirical relation for the required crest level above design SWL, see equation 5.9.

$$R_C = -\frac{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}{4.3} \cdot \ln\left[\frac{q \cdot \sqrt{\tan\alpha}}{0.067 \cdot \sqrt{g \cdot H_{m0}^3} \cdot \gamma_b \cdot \xi_{m-1,0}}\right]$$

$$\text{with a maximum of } R_C = -\frac{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}{2.3} \cdot \ln\left[\frac{q}{0.2 \cdot \sqrt{g \cdot H_{m0}^3}}\right], \text{ for } \xi_{m-1,0} < 5 \qquad [5.9]$$

The following assumptions are made with respect to the preliminary cross-section of a permanent barrier across the Wash estuary:

- a vertical wall on top of the crest of the storm surge barrier is not considered, therefore $\gamma_v = 0$;
- the angle of the incoming waves under design conditions varies between 0° and 45°. Since waves with an angle of incidence (β) of 0° result in the highest run-up and as a consequence the largest average overtopping discharge, $\beta = 0°$ is used in the preliminary design;
- a possible berm will be situated at design SWL, because the berm is then the most effective, see equation 5.10 and figure 60. From theory it is known that the berm width should be equal to or smaller than ¼·L_0. In this specific case berm width at the seaward side should be smaller than 70 m.

The relation between the influence factor for a berm and the position of the berm relative to SWL is given in the following equation [EurOtop, 2007].

$$\gamma_b = 1 - r_b \cdot (1 - r_{db}) \text{ for}: 0.6 \le \gamma_b \le 1.0 \qquad [5.10]$$

And

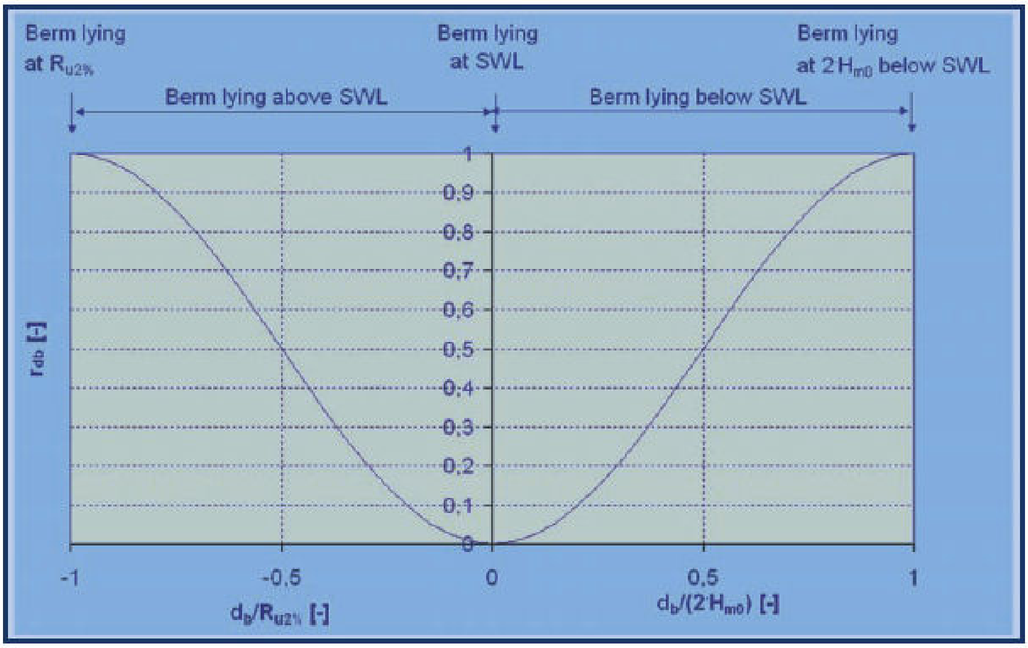
$$r_b = \frac{B}{L_{berm}} \qquad [5.11]$$

$$r_{db} = 0.5 - 0.5 \cdot \cos\left(\pi \cdot \frac{d_b}{R_{u,2\%}}\right) \text{ for berm above still water line}$$

$$r_{db} = 0.5 - 0.5 \cdot \cos\left(\pi \cdot \frac{d_b}{2 \cdot H_{mo}}\right) \text{ for berm below still water line} \qquad [5.12]$$

Where:

γ_b	:	influence factor for a berm	[-]
r_b	:	normalised width of the berm	[-]
B	:	horizontal width of the berm	[m]
L_{berm}	:	characteristic berm length	[m]
r_{db}	:	normalized difference between SWL and the middle of the berm	[-]
d_b	:	vertical difference between middle of berm and SWL	[m]
H_{mo}	:	significant wave height	[m]



Taken from the Overtopping manual [EurOtop, 2007].

Figure 60: relation between berm position and r_{db}.

Initial computations have shown that at the basin side of the permanent barrier a slope of 1:3 can be selected, as the required crest height at the North Sea side of the barrier is much larger than the crest height needed at the basin side under design conditions. For the same reason an inner berm has no purpose and only increases the volume of the cross-section. A crest width of 10 m is selected to accommodate a service road and space for utilities related to the tidal power plant.

In figure 61 the relation between berm width and the required crest height above design SWL at the North Sea side of the permanent barrier is presented for several slopes and an influence factor for surface roughness $\gamma_f = 1.00$. The horizontal line represents the required crest level based on the design conditions within the basin behind the storm surge barrier. As expected the required crest height above SWL decreases with increasing berm width. The effect of an increasing berm width is largest for slopes 1:4 and 1:5, while for a slope of 1:3 a berm is only effective for a width between 10 to 25 m.

The required crest height above SWL turns out to be very large and does not seem very economical because the required crest height at the North Sea side is, even for very mild slopes, much larger than that at the basin side. Since the outside design SWL is already higher than that within the basin, the difference in required crest height related to ordnance level is even larger. Decreasing the difference between both crest levels results in a more economic design. This can be accomplished by using a double layer of quarry stone on the outside slope, which will reduce the required crest height by a factor 0.5, since $\gamma_f = 0.50$ for this type of elements [Verhagen, 2009].

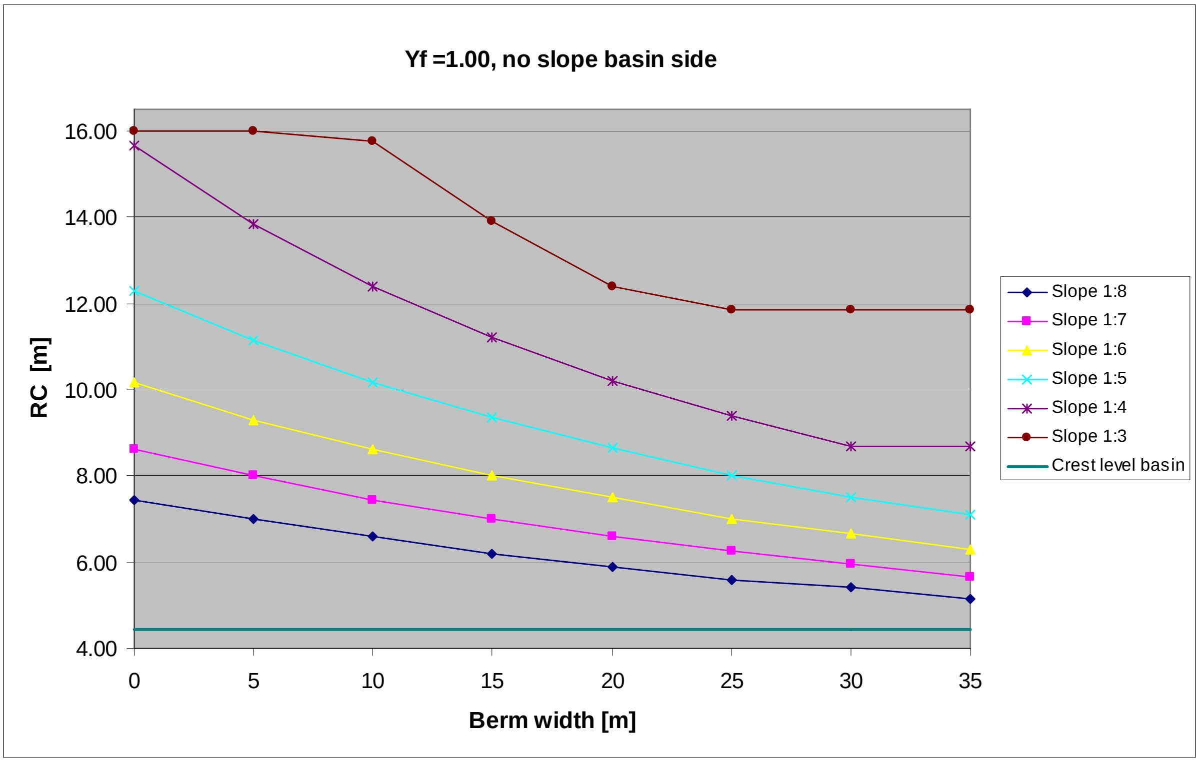


Figure 61: crest height above SWL as a function of the outer berm width, surface roughness factor is 1.00.

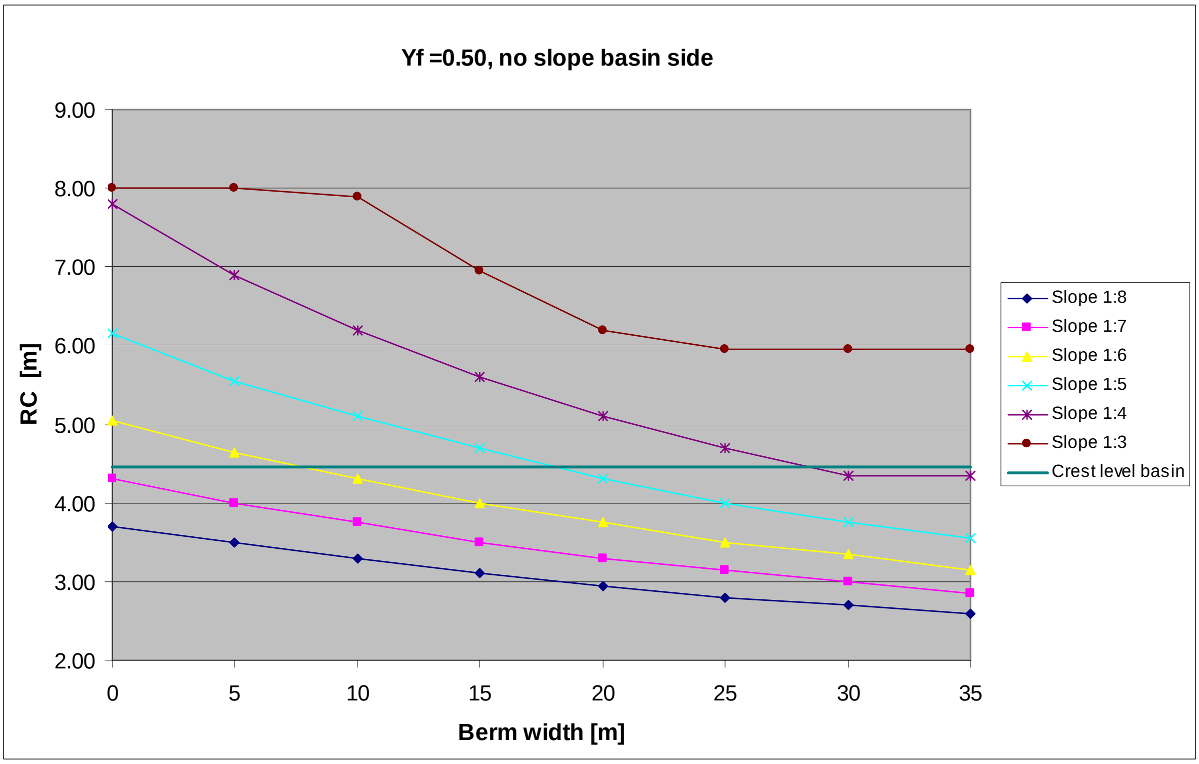


Figure 62: crest height above SWL as a function of the outer berm width, surface roughness factor is 0.50.

In figure 62 the relation between required crest level above SWL and the outer berm width is presented for several slopes, now using an influence factor for surface roughness $\gamma_f = 0.50$. Again the horizontal line represents the required crest level at the basin side of the barrier ($\gamma_f = 1.00$). The overall differences between figure 61 and 62 are the halving of the required crest height above design SWL and the smaller difference between the required crest height relative to ordnance level between the North Sea side and the basin side of the barrier, further the result is fairly similar. Again for the very mild slopes the effect of a berm seems not to be very large, see also figure 63.

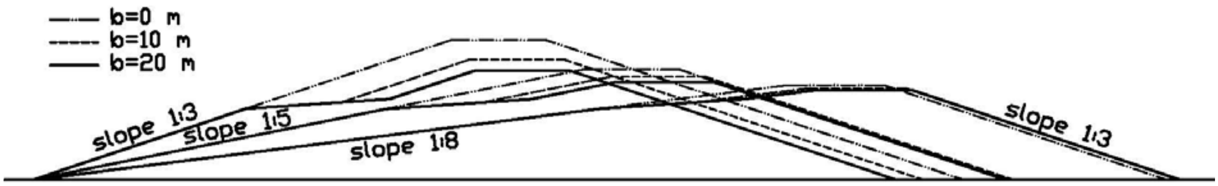


Figure 63: relation crest height and berm width.

Based on the 1:3 slope it seems that the optimum outer berm width lies somewhere between 10 m and 20 m. However for the other slopes regarded, this interval is not so clear. Since the volume increases quadratic with the total height of the barrier, the relation between the volume of the cross-section of the barrier and the berm width has been determined for several slopes, see figure 64.

The following assumptions are made computing the volume of the typical cross-section of the permanent barrier:

- settlement of the subsoil is computed, using Koppejan's equation, see equation 5.2, the values used for both the primary and secondary compression coefficients[43] correspond to weak silty clayey sand, according to table 1 of the National Annex to EN 1997-2 Eurocode 7: Geotechnical Design. Furthermore 100% consolidation is assumed. The layer of subsoil regarded has a thickness of 10 m, the settlement is computed at a depth of 5 m below the seabed;
- 5% settlement of the new material is taken into account, as the core is assumed to consist of sand;
- in the analysis the foundation level is taken at -10 mODN.

Except for a 1:3 slope the trend lines in figure 64 all fit the computed values rather well. The reason for the bad fit in case of a 1:3 slope, is that a berm has no effect on the required crest height above SWL until a width of 10 m is reached. The same holds for a berm width larger than 20 m. However the volume of the cross-section increases with increasing berm width, so if the crest level is not reduced the total required volume increases.

The other trend lines have a minimum at a berm width of either 15 m (slope 1:8 to 1:6) or 20 m (slope 1:5 & 1:4). Overall the shape of the trend lines for slopes 1:8 to 1:5 is very similar, but the trend line for a 1:8 slope has a steeper tail. The reason for this is the fact that, as a result of the mild slope, the required crest height with respect to ordnance level at the North Sea side is a few centimetres lower than that at the basin side of the permanent barrier. Hence the difference in crest level is added to the required crest height above SWL, as computed for the North Sea side of the barrier, resulting in an additional increase of the required volume.

[43] Primary compression coefficient: $C_p = 450$; secundary compression coefficient: $C_s = 0$.

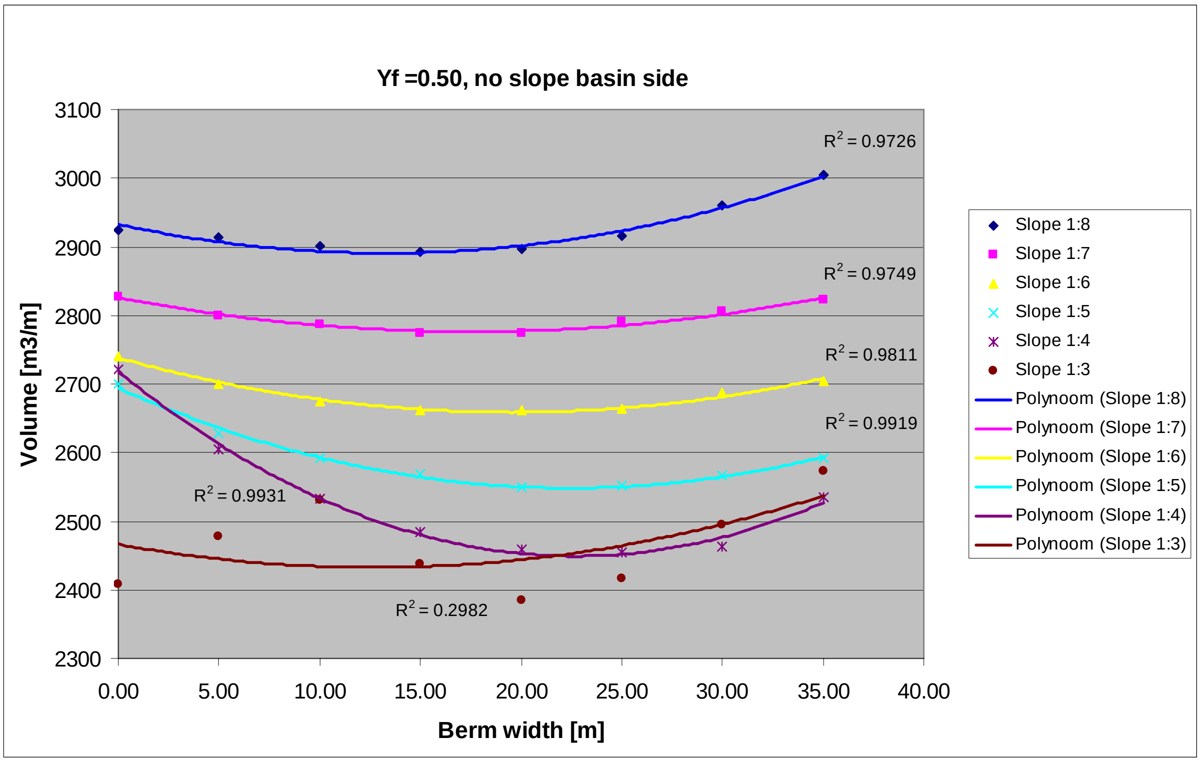


Figure 64: volume as a function of the outer berm width, surface roughness factor is 0.50.

From figure 64 it can be concluded that despite the fact that a 1:4 slope requires a higher crest level (12.50 mODN) than a 1:5 slope (11.70 mODN), the required volume is smaller. So it seems logical to select a 1:4 slope for the outer berm. However after reviewing other closure dams the outer slopes are in the range 1:6 to 1:5, therefore a 1:5 slope is selected as the outer slope in the preliminary design. Most probably stability reasons form the basis for not using a 1:4 slope on the outer berm, this is confirmed by a rule of thumb concerning slope stability, which states that for significant wave heights exceeding 3.00 m the outer slope should be milder than or equal too 1:5 [Bekker et al,1998]. So either way a 1:4 slope turns out to be to steep.

Hence the preliminary profile of the dam body consists of an outer slope of 1:5, an inner slope of 1:3 and an outer berm at design SWL with a width of 20.00 m. Crest level is situated at 11.70 mODN, and the crest width is 10.00 m, see figure 65. The outside slope is covered with a double layer of quarry-stone while the inside slope is covered with asphalt until 9.00 mODN[44]. The upper part of the inner slope is covered with a grass cover on clay. Both the toe structures and bottom protection are not depicted.

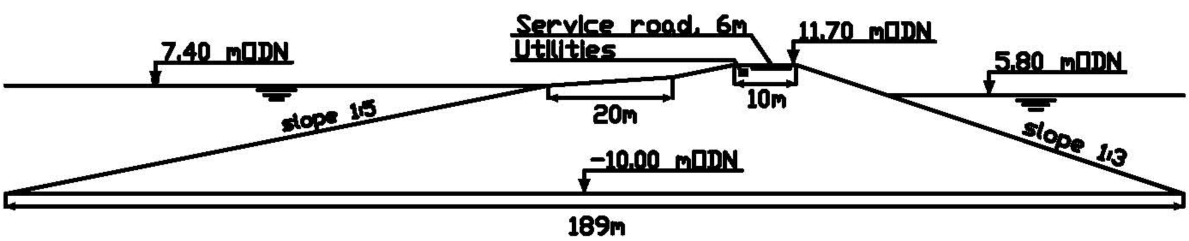


Figure 65: preliminary cross-section permanent barrier (not to scale).

44 Length slope protection $0.25 \cdot R_{u2\%,smooth}$ below SWL to $0.50 \cdot R_{u2\%,smooth}$ above SWL; $R_{u2\%,smooth} = 6.30$ m.

Since no ballpark figure per m^3 material is available it is assumed that the costs/m^3 material are two times larger than that of the land based section[45], resulting in a ballpark figure of £ 60/m^3. Since the total volume of the permanent barrier is $60.95 \cdot 10^6$ m^3 (alternative 2-3, see table 41), the construction costs are estimated to be £ $3.66 \cdot 10^9$. This very crude cost estimation does not include costs regarding the navigation locks, bottom protection and mitigating measures.

5.4 Summary

In table 45 an overview is presented of the estimated costs per alternative. From the table can be concluded that raising the current flood defences (including mitigating the expected effects of climate change) turns out to be the cheapest solution. Hence, closure of the Wash estuary is not necessary for providing sufficient flood protection of the hinterland in the future.

However, since the main purpose of this thesis is establishing the technical and economical feasibility of a storm surge barrier combined with a tidal power plant, alternative 2b will be selected for further elaboration to a final conceptual design. Although the construction costs of all alternatives are determined very crudely it is deemed safe to conclude that alternative 2a will be more expensive than alternative 2b. Furthermore an embankment dam combines the dual function better (providing flood protection and harnessing tidal power).

Alternative	Description	Estimated costs [10^9 £]
Zero alternative	adapt current flood protection for climate change	1.04
Alternative 1	raise current flood protection level too SoP 1:500	1.13
Alternative 2a	construct combination movable & permanent barrier	34.69
Alternative 2b	construct permanent barrier	3.66

Table 45: cost comparison of the alternatives.

The fact that alternative 2a is much more expensive than a permanent barrier, including the cost of a tidal power plant (£ 3824M, see section 7.3), clearly indicates that the construction of a movable, Eastern Scheldt type of barrier, is not a suitable solution at all for the closure of the Wash estuary.

5.5 Navigation locks

Since it is preferable to keep commercial and recreational traffic as much separated as possible *and* because the pleasure craft tend to sail close to the coastline, leaving the Wash estuary near Hunstanton, the decision is made to construct separate lock facilities. A navigation lock in deep water for the commercial vessels, including the fishing fleet, situated at the western boundary of the Lynn Deeps. And a navigation lock in the shallower water off the coast near Hunstanton for the recreational traffic.

For both lock systems the highest lock level is taken equal to mean high water spring, see table 11 in chapter 2, plus an additional 1.30 m to compensate for the expected effects of climate change. Lowest lock level is taken equal to mean low water spring (see table 11). Top

[45] Construction under water will result in larger losses of material, floating equipment is more expensive than rolling equipment, armour is required and working conditions at sea will result in more delays.

of floor of the structures depends on the draught of the design ships, while top of structure is taken equal to the crest height of the embankment dam, +11.70 mODN. Based on the 1:500 storm surge level on the North Sea, a design significant wave height of 3.00 m in the outer harbour and a freeboard of 1.00 m top of structure should be +11.40 mODN, this is rounded upward to equal the height of the embankment dam. The design significant wave height does not equal the significant wave height on the North Sea (5.50 m). The reason for this is that the lock complexes will not be in operation under severe storm conditions and because under these conditions some overtopping discharge is not considered to be a problem.

At the basin side top of structure, +9.30 mODN, is based on the 1:200 spring tidal level (see table 13 in chapter 2), a relative sea level rise of 1.30 m, a wind set-up of 0.60 m, a freeboard of 1.00 m and a design significant wave height of 2.50 m.

5.5.1 Commercial vessels

Checking with the port authorities of the commercial ports present within the Wash estuary resulted in the following maximum ship dimensions that are able to use the berths, see table 46.

	Boston	**King's Lynn**	**Sutton Bridge**	**Wisbech**
LOA [m]	120	> 100	120	83
B [m]	13.6	?	17	?
D [m]	6	6	6	5.2

LOA = length overall; B = beam and D = draught.

Table 46: maximum ship dimensions that can access the ports in the Wash estuary.

Furthermore it is assumed that the local fishing vessels have an length overall of 15-25 m, a beam of 6-7 m and draught of 3.5 m [Ligteringen, 2009]. From these figures the dimensions of the design ship for the commercial navigation locks are deduced, resulting in:

LOA: 120 m;
B: 17 m;
D: 6 m.

Per annum on average commercial 1800 ship movements, excluding fishery related movements, take place within the estuary. This corresponds to on average 2-3 ships per calendar day. Due to lack of data it is assumed that in the future this number will remain the same, therefore the lock chamber dimensions will be based on 1 design ship per lock cycle.
Inner chamber dimensions [Molenaar et al, 2011]:

$$L_{chamber} = 1.1 \cdot \mathrm{LOA} = 1.1 \cdot 120 \approx 135\,\mathrm{m}$$

$$B_{chamber} = 1.25 \cdot \mathrm{B} = 1.25 \cdot 17 \approx 22\,\mathrm{m}$$

$$D_{chamber} = 1.15 \cdot \mathrm{D} + 0.5 = 1.15 \cdot 6.0 + 0.5 \approx 7.50\,\mathrm{m}\ \text{(below lowest water level)}$$

Hence top of floor is situated at -9.50 mODN.

5.5.2 Recreational vessels

The dimensions of the sailing ships and motor vessels as presented in table 47 are based on Ligteringen [Ligteringen, 2009], the dimensions include 90% of the vessels in the British recreational fleet.

	Sailing ships	Motor vessels
LOA [m]	10-15	10-15
B [m]	3-4	4-5
D [m]	1.5-2	1.5-2

LOA = length overall; B = beam and D = draught.
Source: Ligteringen, 2009.

Table 47: maximum dimensions of recreational crafts.

With the use of table 47 the design ship for the recreational navigation lock are established, being: LOA 15 m;
B 5 m;
D 2 m.

Because in spring and summer there is much recreational traffic in the Wash estuary, but no figures are available, it is assumed that the inner chamber dimensions will be large enough to accommodate 6 design vessels per lock cycle. Hence the inner chamber dimensions become:

$$L_{chamber} = 2 \cdot \mathrm{LOA} + 1.1 \cdot \mathrm{LOA} = 2 \cdot 15 + 1.1 \cdot 15 = 46.50\,\mathrm{m}$$

$$B_{chamber} = \mathrm{B} + 1.25 \cdot \mathrm{B} = 5 + 1.25 \cdot 5 \approx 12.00\,\mathrm{m}$$

$$D_{chamber} = 1.15 \cdot \mathrm{D} + 0.5 = 1.15 \cdot 2.0 + 0.5 \approx 3.0\,\mathrm{m} \text{ (below lowest water level)}$$

Therefore top of floor is located at -5.00 mODN.

5.5.3 Lock gates

The following four gate types are frequently used for navigation locks:
- mitre gates;
- vertical lift gate;
- rolling/sliding gate;
- single leaf gate.

In this section the most suitable gate type is selected for both the commercial and recreational navigation locks situated in the proposed Wash barrier.

The rolling/sliding gate is ruled out because the navigation locks are intended to be designed as caissons. Hence, the width wise space requirement is too large. The single leaf gate is ruled out in advance because this type of gate is only suited for lock chamber widths up to 10 m.

Advantages of mitre gates are:
- no air draught limitation;
- relative light gate structure, so less installed power and smaller equipment compared to a vertical lift gate;
- large spans, result in lighter lock head structures compared to a vertical lift gate.

Disadvantages are:
- two sets of doors are required as a result of the required two-way water retention;
- without hydraulic pistons the gates should not be opened under a head difference;
- an additional set of doors is required in case the navigation lock must remain operational during maintenance;

- vulnerable to ship impact, even in opened position;
- relative large space requirement for the lock head.

Advantages of a lift gate are:

- small space requirement for the lock head;
- gate can be opened under a head difference;
- one gate is able to retain water in two directions;
- much less vulnerable for ship impact, and not vulnerable in opened position.

Disadvantages are:

- air draught limitation;
- large superstructure required (lift towers, counter weights);
- in case of larger spans the lock head becomes larger and more heavy, especially compared too mitre gates as one gate only has to span half of the total span;
- compared too mitre gates a large amount of power is required to operate the gates, even when counter weights are used;
- need an additional gate in case the lock has to remain operational during maintenance.

In table 48 an overview is given of the main factors governing the choice regarding the type of gates for the navigation locks in the Wash barrier. From the table can be concluded that mitre gates are best suited for both navigation locks.

Characteristic	Mitre gates	Vertical lift gate
Air draught limitation	++	--
Space requirement lock head	+/-	++
Visual impact on seascape	++	--
Suited for medium to large spans [1)]	+	++
Total	**++**	**+/-**

++ = very good; + = good; +/- = fair; - = poor; -- = bad
[1)] Medium span 10-16 m, large span 16-24 m [Molenaar et al, 2011].

Table 48: comparison between mitre gates and a vertical lift gate.

5.5.4 Preliminary design

As was mentioned earlier the navigation locks are to be designed in such a way that they can be towed to the location where they are to be sunk on an underwater sill. Rules of thumb are used to determine the rough dimensions of the preliminary design. The lock chamber walls are thicker than the outer walls because mooring rings, ladders in recesses, etc. have to be included into the walls.

With respect to the manoeuvrability of the caissons during towing a length/width ratio of 3:1 is sufficient. However a length/width ratio of 3.8:1 is more favourable for navigation. The recreational lock has a ratio of 3.8:1, while the commercial lock has ratio of 3.3:1 (it is assumed that lock consists of two caissons of equal length). Temporary wooden bulkheads at the end side of the caissons are used as sealing during navigation.

In figures 66 and 67 cross-sections of respectively the commercial and recreational navigation lock are presented. Longitudinal cross-sections and top views are included in appendix 11. Both lock complexes feature a harbour of refuge at the North Sea side of the barrier, see figure 92 in chapter 7.

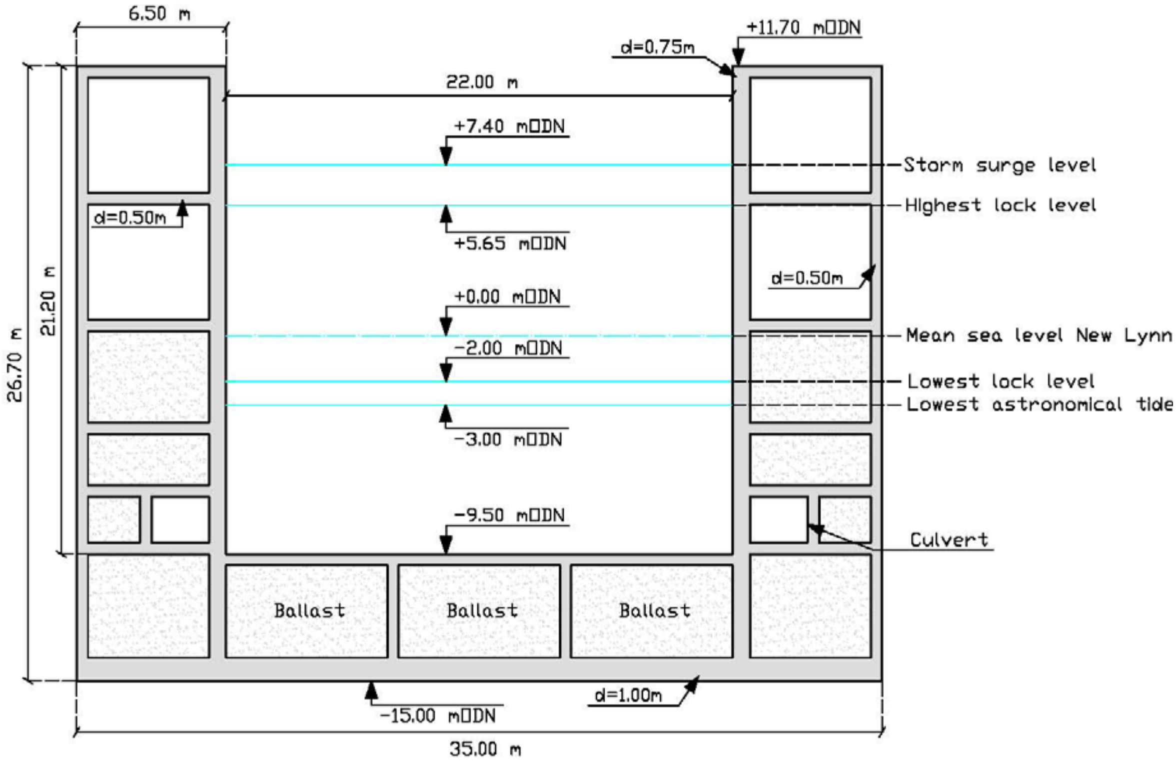


Figure 66: cross-section lock chamber commercial navigation lock (not to scale).

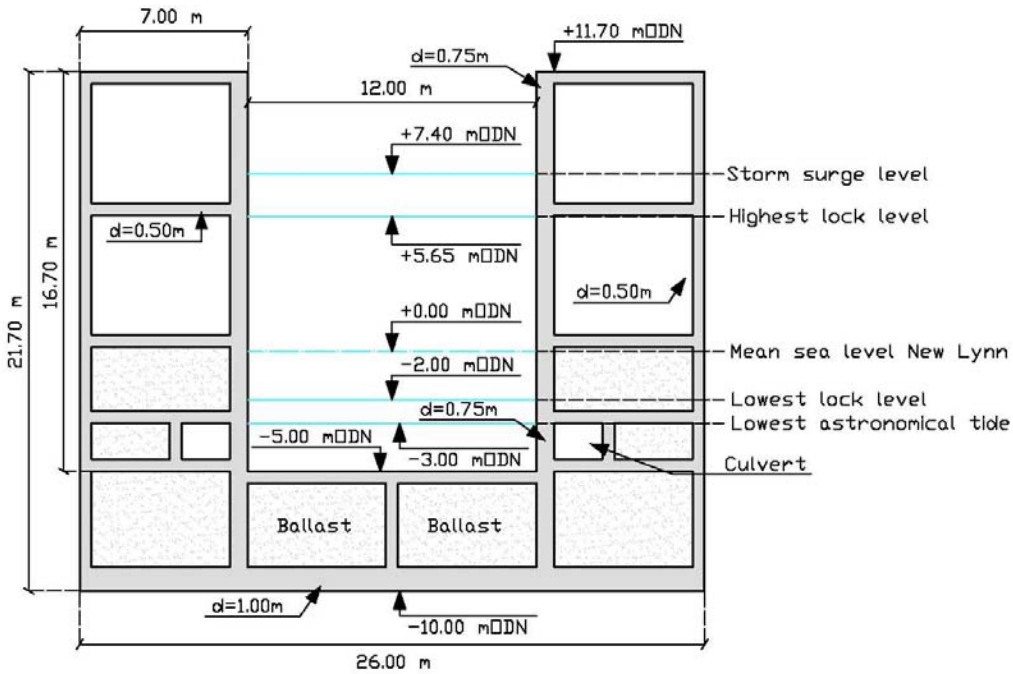
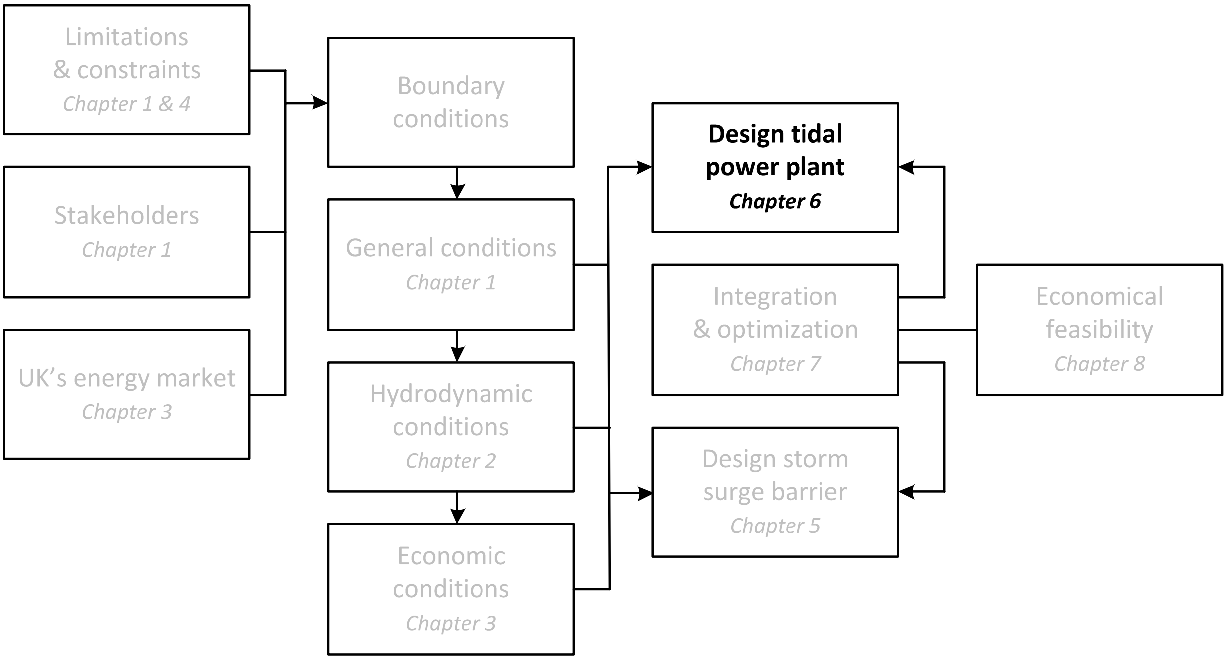


Figure 67: cross-section lock chamber recreational navigation lock (not to scale).


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

6 TIDAL POWER PLANT

The usage of the available energy harnessed within the tidal motion is not a new concept, in essence a modern tidal range power plant is a large scale version of a tidal mill. In the United Kingdom records exists of tidal mills going back in time as far as the 6th century (Along the Irish coast) and maybe even to Roman times[46].

Archaeological findings along the embankments of the River Fleet[47] in London are possibly of a Roman tidal mill and are dated back to the early Roman period in the United Kingdom. This would mean that in England tidal power was already being used as early as the period 43 AD to 200 AD [Spain, 2002].

The earliest recordings of tidal mills in England are found in the Domesday book (1086 AD), that mentions several tidal mills, the earliest being located in Dover harbour. Also mentioned were the tidal mill in Eling and the tidal mills on the River Lea at Three Mills Island, the latter became the largest in England by the 18th century. At that time there were 78 tidal mills in the London area alone[48].

Figure 68: House Mill and the Miller's House at Three Mills at low Tide, London's Docklands. (Courtesy: *Gordon Joly, 2006*)

Figure 69: Thorrington Tide Mill. (Courtesy: *Roger W Haworth, 2006*)

Data from Meighs [1970, 1979] and Minchinton [1979] state that at one time there were 750 tide mills operating along the shores of the Atlantic Ocean, of which approximately 300 in North America, 200 on the British Isles and 100 in France. During the second half of the 20th century the use of tidal mills declined dramatically. Rex Wailes[49] discovered in 1938 that of the 23 still existing tidal mills in England only 10 were still working. At present day in England there are only five tidal mills left and of two more tidal mills only elements survived, see the list below. The tidal mill in Eling is the only one still working in the United Kingdom, that of Woodbridge is still in working order.

[46] 43 AD to 410 AD; source: www.bbc.co.uk/history/british/timeline/romanbritain_timeline_noflash.shtml.
[47] The largest subterranean river in London.
[48] Source: http://en.wikipedia.org/wiki/Tide_mill.
[49] Source: http://en.wikipedia.org/wiki/Tide_mill.

Present day (remains of) tidal mills that have survived in England[50]:

- Thorrington Tide Mill (1831), Thorrington, Essex;
- Three Mills Tide Mill (<1086), Bromley by Bow, London;
- Eling Tide Mill (<1086), Hampshire;
- Woodbridge Tide Mill (1170), Woodbridge, Suffolk;
- Carew Castle Tide Mill (1801), Pembrokeshire, Wales;
- Tide Mills (1761), Newhaven, East Sussex (sluice only);
- Pembroke Tide Mill (1542), Pembrokeshire, Wales (mill pond only).

The previous paragraphs show that the UK has already a long history of using tidal power, so why not continue that tradition in this era were sustainable development and the replacement of fossil fuels by renewable energy sources are key guiding principles for political decision making.

In this chapter the technical feasibility of a tidal power scheme in the Wash estuary will be determined. After describing the different forms of tidal power, the most suitable turbine is selected for the Wash estuary tidal power plant. Then the same is done in order to determine the most suited sluice gates. Next the different schemes are described, as are the possible modes of operation. Finally it will be determined which type of scheme has the most potential within the Wash estuary and a preliminary design is presented.

6.1 Tidal power

Two types of tidal power can be distinguished, namely tidal stream power and tidal range power.

6.1.1 Tidal stream power

Tidal streams are relatively fast[51] volumes of water caused by the tidal motion and occur usually in shallow seas where a natural constriction or human interventions force the water to speed up. Tidal stream power schemes harness energy either in a similar way as wind turbines or as a tidal fence, which resembles a tidal barrage and closes of an entire channel. In contrast to conventional tidal barrages, which need sluices to set up the water level within the basin, the tidal fence uses the tidal current to directly drive the tidal turbines. In both cases tidal currents need a minimum flow velocity of 2.0 too 2.5 m/s in order to be able to propel the turbines.

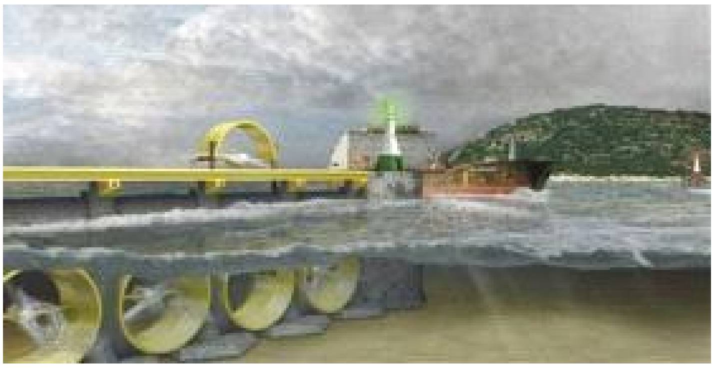


Figure 70: on the left a tidal stream turbine (Courtesy: *OpenHydro*), on the right an artist impression of the Severn tidal fence (Courtesy: *unknown internet source*).

[50] Source: http://en.wikipedia.org/wiki/Tide_mill.
[51] Fast with respect to tide induced ocean currents.

However for the Wash estuary tidal stream power is not an option, first of all the flow velocity is too low. The largest tidal current velocities within the estuary are measured in the main channels during spring tide and are in the order of 1.20 m/s [The Wash SMP2, appendix C, 2010]. Secondly the main purpose for building a barrage in the estuary is flood protection. As tidal stream power is either captured by means of single turbines, that may be positioned in groups to form a so-called marine current farm, or by means of a tidal fence, no decent flood protection is provided.

6.1.2 Tidal range power

Tidal range power makes use of the potential energy present as a result of the head difference over a barrage during the tidal cycle. As a rule of thumb a tidal range power plant is economically feasible when the available mean tidal range is 5 m [Clark, 2007]. In the Wash estuary the mean tidal range is 4.70 metres. Therefore it may be possible to design an economically feasible tidal power plant, as a large part of the estuary is very shallow. An additional advantage of the barrage needed to harness the tidal energy is the possibility to reduce the flood risk in the hinterland. In case the UK Government participates in the project this will result in revenues, see chapter 8.

In the remainder of this chapter the focus will be on the elements and possible layout of a tidal range power plant.

6.2 Turbines

The type of turbine chosen and the corresponding discharge capacity are very important factors in the design of a tidal power plant. According to Clark [2007] the electromechanical equipment generally accounts for 45 to 55% of the direct costs of development and indirectly affects a further 30 to 35% of the direct costs, because the dimensions of the powerhouse are largely dependent on the size of the turbines and also the number and size of the sluices are dictated by the turbines discharge capacity. Hence the power generation capacity of a tidal power plant as a whole depends on the type of turbine chosen and their number.

The character of the tidal environment imposes specific requirements on the turbines, that are listed below [Clark, 2007]:

- the available head is low and continuously changing;
- a low head requires a large discharge;
- cyclical operation results in larger stresses on the turbines;
- generally turbine dimensions must be kept as small as possible, with respect to the construction costs. However in deep water it is better to keep the number of turbines small;
- due to the large quantity of water available, overall efficiency of the turbine is of less importance;
- be able to withstand rough conditions resulting from pressure fluctuations due to waves and the fact that the turbines seldom operate in the optimum hydraulic range;
- the turbines are placed in a highly corrosive environment.

The only turbines suitable to operate under these conditions are very low-head axial-flow turbines with a high specific-speed[52]. The four basic types are; vertical Kaplan turbines, tubular turbines, straight flow turbines and bulb turbines [Clark, 2007]. Figure 71 shows the overall dimensions for these turbine types. It is apparent that the tubular turbine and the vertical Kaplan turbine require a larger depth of foundation and therefore do not fulfil the requirement of compactness, as mentioned above. Therefore only the straight flow turbine and bulb turbine will be considered in more detail.

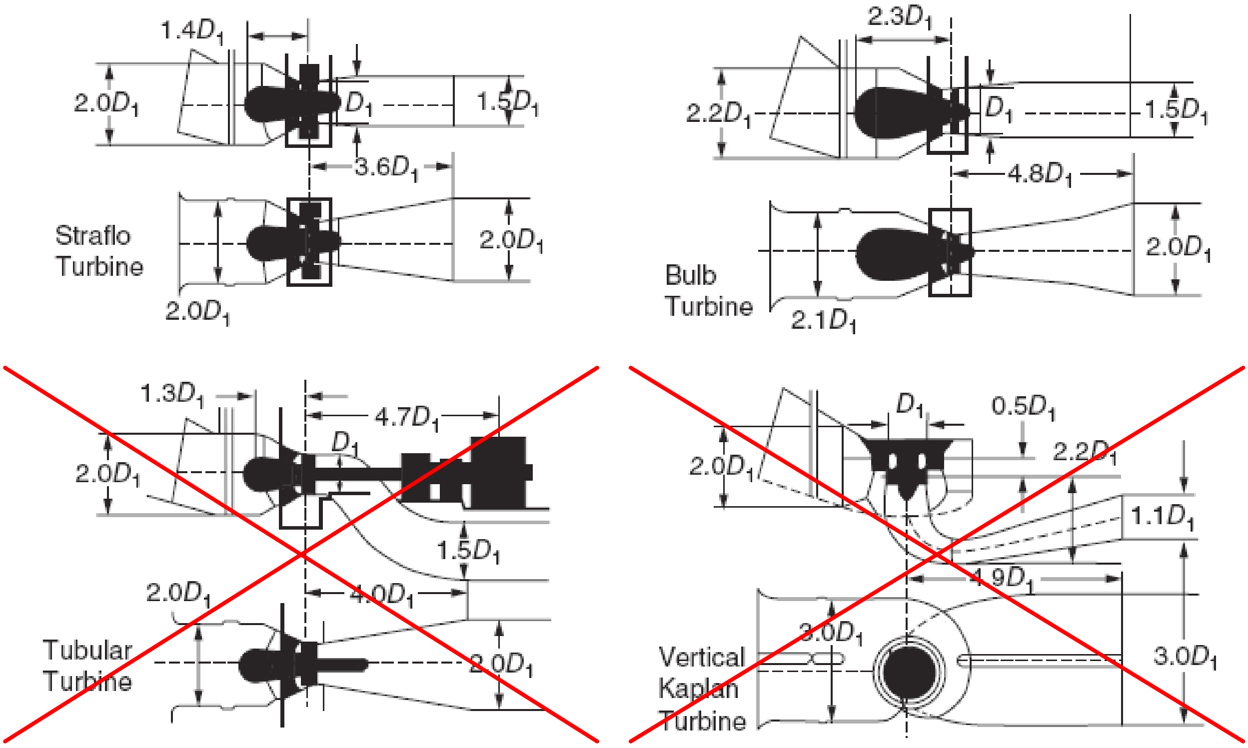


Figure 71: overall dimensions for low-head turbine designs (Courtesy: *Clark, 2007*).

6.2.1 Straight flow turbine

The straight flow turbine or Straflow turbine is a turbine type were the generator and the runner blades form an integral unit without a driveshaft. The generator rotor poles are located in a rim that is fixed peripherally to the runner blades, see figure 72.

Advantages:

- the absence of a bulb results in a shorter, less expensive structure (turbine housing and powerhouse) and also results in less hydraulic losses;
- there is no restriction in the water passage;
- the generator is more easy accessible for maintenance;
- needs less maintenance as a result of only a small number of moving parts.

[52] Specific speed = that speed in rpm at which a turbine would operate if the runner diameter is reduced to a size that would generate 1 kW under 1 m head difference.

Disadvantages:

- some trouble with water tightness of the seal between rotor and generator;
- the turbine has fixed runner blades and has therefore a smaller operational range, which results in a loss of a portion of the potential energy;
- is not suitable for operation in both directions and pumping;
- needs a separate downstream gate in order to control starting and stopping the turbine for both the generating cycle and the reverse sluicing cycle;
- the gate must be able to be closed during maximum flow in case of an emergency;
- less fish friendly than a bulb turbine;
- there is not much experience and expertise in its design and use.

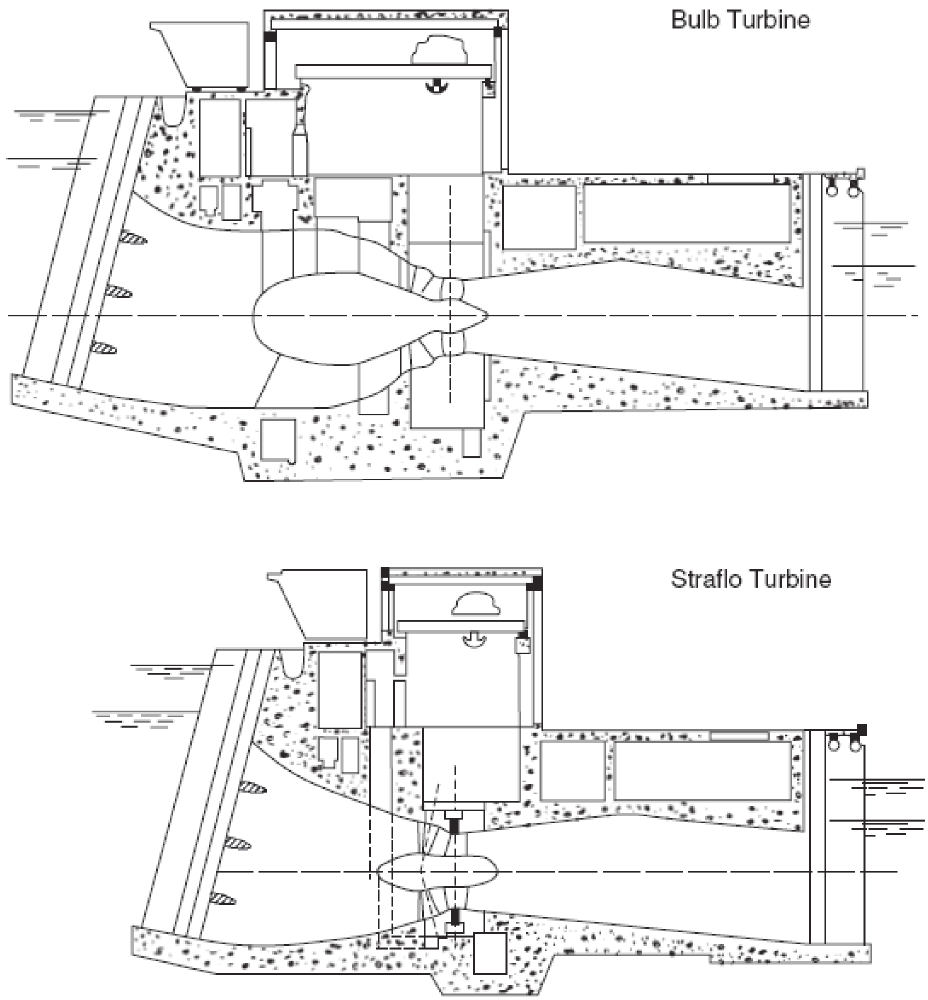


Figure 72: comparison of straflo and bulb turbine powerhouses for the same output and head. (Courtesy: *Braikevitch, 1970*).

6.2.2 Bulb turbine

This turbine type consists of a large steel bulb connected to stay vanes, variable-pitch runner vanes on a horizontal shaft and adjustable guide vanes. The generator is mounted inside the steel bulb that is placed within the straight water passage. The bulb is located at the upstream side of the turbine runner and is suspended via the stay vanes, see also figure 72.

Advantages:

- double regulation is provided by the adjustable guide vanes and runner vanes, thus improving the overall efficiency (larger operational range). The turbines also can be used for sluicing;
- improved hydraulic efficiency, larger discharge capacity and larger power output for the same runner diameter than other designs;
- adaptable to operate in different modes (as an orifice in both directions, as a turbine and as a pump);
- relative fish friendly as a result of the relatively slow rotational speed;
- has the advantage of experience and expertise in its design and use.

Disadvantages:

- rotational speed has to be kept low to avoid cavitation damage, resulting in a larger generator, which in turn results in a larger bulb. However for economic and hydraulic reasons the bulb should be as small as possible;
- the turbine intake must be carefully designed in order to prevent unacceptable fluctuations in generator output;
- the larger generator results in an increase of the length of the convergent section in the water way in order to avoid higher energy losses due to a sharper gradient in the flow direction along the bulb;
- is more expensive due to the adjustable runner and guide vanes *and* larger structure (powerhouse and turbine housing).

6.2.3 Conclusion

Regarding the cost aspect, which is a very important driver in making a choice between the turbine types, the straight flow turbine scores well because the dimensions of both the structural elements, e.g. the powerhouse, and the dam body will be smaller compared to the requirements needed for housing a bulb turbine. Furthermore the bulb generator is double regulated what makes the turbines more expensive, and the runner diameter needed is larger as a result of the large bulb required to house the large generator needed to keep the rotational speed of the turbine sufficiently low. As the bulb turbine has a larger runner diameter, the depth of foundation will be deeper compared to that of a straight flow turbine, resulting in greater costs.

Another important driver is the operating flexibility of the scheme. With respect to tidal power schemes, where the head difference varies continuously, regulation of the flow through the turbine is necessary to maintain enough efficiency within the range of operating conditions. The bulb turbine is double regulated which enables the turbine to be used for generating electricity in both directions. Also this turbine type can be used for pumping, which is often used during neap tide in order to increase the head difference to a suitable level for electricity generation [Duivendijk, 2007]. The straight flow turbine has fixed runner blades and as a result has a smaller operational range, also the turbine is not suited for double-effect operation. Furthermore the straight flow turbine is not suited for pumping.

Weighing factor	Straight flow turbine	Bulb turbine
Costs	++	--
Operational range	+/-	+
Experience with turbine	--	++
Double operation & pumping	--	++
Fish friendliness	+	++
Total	-	++

++ = very good; + = good; +/- = fair; - = poor; -- = bad

Table 49: comparison between straight flow and bulb turbine.

Other advantages of a bulb turbine are the facts that there is experience and expertise in designing and operating this type of turbine and that the turbines can be used for sluicing, which means that no sluices have to be installed, thus saving costs. Also the bulb turbine is more fish friendly due to its slower rotational speed. Hence the most suitable turbine appears to be a bulb turbine, see table 49. This turbine is therefore chosen.

6.3 Sluices

Another important element of a tidal power plant are the sluices as they are regulating water levels between the basin and the outside water. Although the installation of bulb turbines does not necessarily require sluices, they will be needed in case the energy generation is to be maximized, see section 6.4.

There are many types of sluice gates, but due to the large concentration of suspended sediment in the water column within the Wash estuary it is to be expected that strong sedimentation will occur during slack water periods as the coarse grains instantaneously react to changes in flow velocity. Due to the skewness of the tide both high water and low water slack can occur before high and low water respectively. Hence sedimentation occurs before the sluice gates have to be closed or opened, depending on the mode of operation see section 6.4. As result all submerged sluice gates, such as a bottom-hinged flap gates, drum gates and submergible radial gates are ruled out. The same holds with respect to rolling or trolley gates operating in horizontal direction. But these last two gates are mainly ruled out due to their large space requirement.

In the Wash estuary a semi-diurnal tide is present, which has a period of 12.42 hrs. Hence annually the sluices have to be opened and closed 8760/12.42 = 705 times in case of a single operation tidal power plant and double that amount in case of a double operation scheme. Therefore the gates have to be capable of rapid, frequent operation and be as free as possible from maintenance and operating problems [Clark, 2007]. Also the gates must be capable of being operated against head differences(in case of power failure at either low or high water), which rules out mitre gates and narrows the choice down to tainter gates or vertical lift gates. Another important requirement is that the power required for gate operation should be kept to a minimum [Clark, 2007].

Figure 73: Haringvliet sluices (Courtesy: *Grontmij*).

6.3.1 Tainter gate

A tainter gate consists of a skin plate shaped as a segment of a cylinder mounted on horizontal and vertical stiffeners connected to radial arms that rotate around trunnions anchored to the piers, see figure 73. All forces are guided via the arms to these trunnions.

Advantages:
- simple, relative light weight design, providing high stiffness;
- large flexibility with respect to dimensions [Duivendijk, 2007];
- no need for gate slots, which is a great advantage [Duivendijk, 2007];
- low power requirement for opening and closing, as water flow helps to open and close the gate;
- relative low height of the structure, since no overhead structure is needed.

Disadvantages:
- long and heavily reinforced downstream piers;
- difficult to design if the outside or basin water levels are to high (high positioning of the trunnions) [US Army Corps of Engineers];
- the maximum width/height ratio is 20-25/16-20 in view of cost [Duivendijk, 2007].

6.3.2 Vertical lift gate

Like a tainter gate, a vertical lift gate is essentially a stiffened plate structure that guides the forces through horizontal and vertical stiffeners to the piers or lifting tower, see figure 74.

Advantages:
- simple shape that is easy to fabricate;
- large span is possible;
- only short piers are required.

Disadvantages:
- need for a large overhead structure;
- sensitive for sediments as the gate must be mounted on rollers to permit movement under a water load. This requires gate slots to guide the rollers;
- the total weight is suspended from the hoisting installation, thus resulting in a high power requirement during opening and closing, even if counter weights are used;
- large friction forces and moving loads.

Figure 74: Eastern Scheldt storm surge barrier (Courtesy: *Reisgids Nederland*).

6.3.3 Conclusion

Important requirements with respect to the sluice gates, besides costs, are defined by Clark [Clarke, 2007] as:
- as little operational problems as possible;
- minimizing the power requirement for gate operation.

Type of gate	Number of gates in survey	Fault frequency per 10 years	Fault frequency % per gate per year
Radial gates	362	235	6.5
Sector gates	107	125	10.0
Vertical-lift gates, roller type	590	770	13.1
Vertical-lift gates, sliding type	2418	73	0.3
Needles	944	11	0.1
Stoplogs	433	44	1.0

Figure 75: fault frequency per 10 years by type of gate, after Lagerholm 1966. (Courtesy: *PIANC InCom WG 26*)

The previous sections clearly indicate that the vertical lift gate (roller type) scores worse on both requirements than the tainter gate, see also figure 75. Because at present no cost information is available, it is assumed that the costs of the large piers (tainter gate) cancel out against the costs of the vertical lift towers. Hence tainter gates turn out to be the most suitable, see table 50.

Weighing factor	Tainter gate	Vertical lift gate
Costs	+/-	+/-
Operational reliability	++	-
Stability of the overall structure	-	+
Visibility	+	-
Total	++	-

++ = very good; + = good; +/- = fair; - = poor; -- = bad

Table 50: comparison between sluice gates.

6.4 Power schemes and modes of operation

In this section the different power schemes and their modes of operation will be discussed. In order to be able to determine which scheme is most suitable for the Wash estuary the (dis)advantages of each scheme are stated.

The following power schemes can be distinguished:

- single basin, single-effect mode of operation;
- single basin, double-effect mode of operation;
- double basin system.

In this section only the first two power schemes will be discussed in more detail. The major benefits of the latter scheme, being able to produce energy during the whole solar day and therefore also during peak demand, is very likely to be outweighed by the disadvantage of the additional costs to be made to construct the extra levees needed within the basin and the additional costs for the more sophisticated generating equipment required. Furthermore, for the double basin system the energy yield will be lower compared to a single basin scheme using either of the two basins [Clark, 2007]. Also Bernshtein [1996] endorses this view point.

6.4.1 Single basin, single-effect mode of operation

Single-effect mode of operation means that the turbines used for the generation of energy only work in one direction, subsequently a distinction can be made between generating energy when emptying a basin (ebb generation), or when filling a basin (flood generation). In addition the head difference between basin level and outside water level can, for both schemes, be increased by pumping.

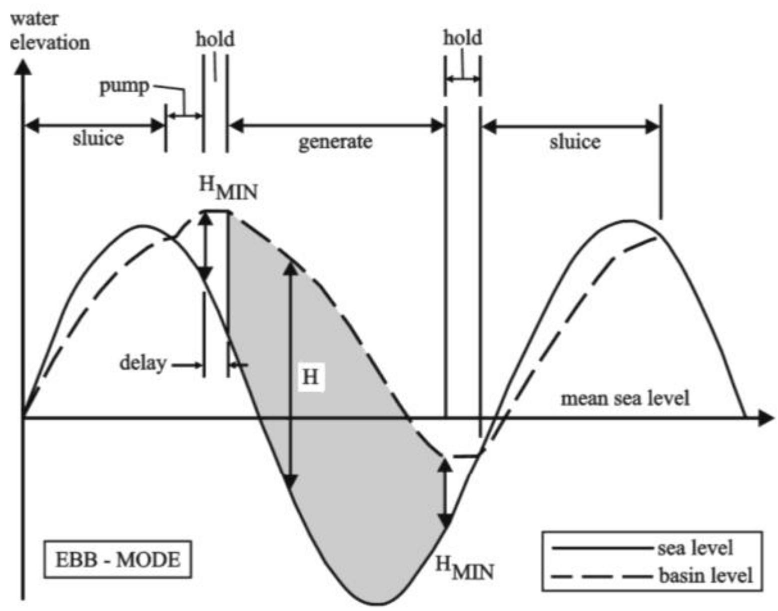


Figure 76: water level versus time during ebb generation (Courtesy: *Wyre tidal energy*).

<u>*Ebb generation*</u>

The cycle of an ebb generation scheme, including pumping, is depicted in figure 76. The dashed line represents the water level variation in the basin during one cycle, while the solid line represents the outside water level variation. During the rising tide the sluices are opened and the basin is filled, at high tide the sluices are closed. Now a waiting period starts during which the outside water level starts to fall. When the head difference is sufficient (1.5 m to half the tidal range) the turbines start to generate energy until the head difference between outside level and basin level reaches the minimum under which they can operate (1.0 m to 1.2 m). At this time the energy production is stopped and after a short waiting period the sluices are opened again to allow the rising tide to fill the basin again. Energy is generated during approximately 50% of the tidal cycle.

Depending on the type of turbine installed it is also possible to overfill the basin after high water. In this case the turbines are used to pump water into the basin during the waiting time after the sluices are closed at high water. Hence water pumped into the basin under a low head, will result in a higher energy yield as a result of the greater head difference during turbine operation. According to theory this results in a net energy gain.

Advantages:

- average water level in the basin increases as a result of the higher low water level, which has a positive effect on shipping;
- generates more energy than a flood generation scheme, due to the slow initial head reduction;
- significant energy gain in case of additional pumping;
- less head variation compared too double effect operation;
- cheaper turbines compared to double effect operation.

Disadvantages:

- average water level in the basin increases. This may cause negative effects on the environment and river discharge capacity;
- discontinuous energy production.

Flood generation

The cycle of a flood generation scheme is approximately the opposite of the ebb generation cycle, see the figure 77. As the falling tide reaches low water the sluices are closed and after a waiting period that gives the rising tide time to create a large enough head difference the turbines start to generate energy. This continuous until the head difference between the outside water level and the rising basin level becomes too small for the turbines to operate. After a waiting period in which the basin level and outside water level become equal again, the sluices are opened so that the basin can empty itself during the remaining period of the falling tide.

As a direct result of the basin geometry it is to be expected that the energy yield from a flood generating scheme is smaller than for an equivalent ebb generating scheme. Because during low water levels a large part of the basin's area consists of mudflats and sand banks that have fallen dry. Hence the water level is expected to rise faster at the beginning of flood generation, thus decreasing the available head difference, than that it will fall during the early stages of ebb generation. Furthermore the volume that passes the turbines will be smaller than for an ebb generation scheme. A flood generation scheme is capable of generating energy during approximately half the tidal cycle.

In case the turbines installed are also suited for pumping, the basin level can be further lowered during the waiting period after closing the sluices at low water. Again the same reasoning holds as for the ebb generation scheme. Only here the water is pumped out under a low head and the energy gain comes from the higher head difference during the filling of the basin. But as the head difference in the early stages of turbining reduces more rapidly it is to be expected that the efficiency of additional pumping is lower for a flood generation scheme than for an ebb generation scheme.

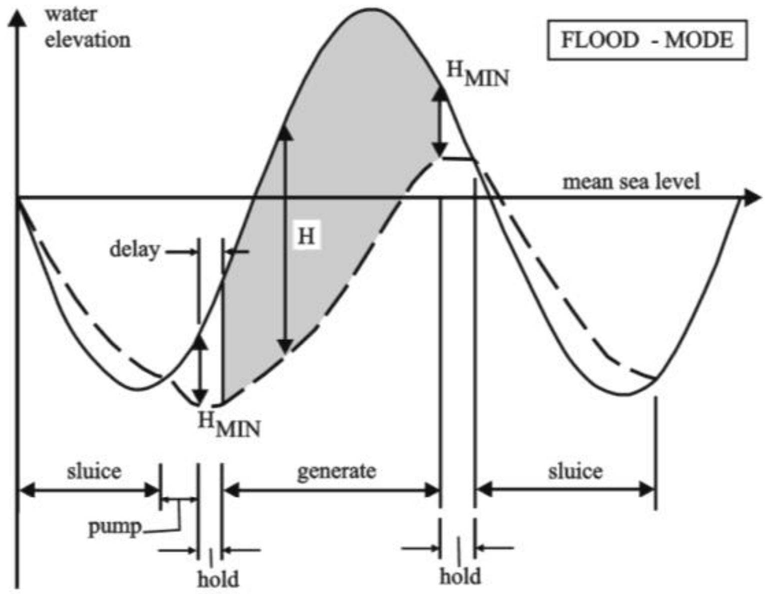


Figure 77: water level versus time during flood generation (Courtesy: *Wyre tidal energy*).

Advantages:

- the average water level in the basin decreases since the low water level stays the same and the high water level is reduced. In this way the river discharge capacity is guaranteed and also the ensures the drying of the sand and mudflats;
- less head variation compared too double effect operation;
- cheaper turbines compared to double effect operation.

Disadvantages:

- results in a decrease of the basin level, even more in case of additional pumping, this will have a negative effect on shipping and possibly on the environment as maybe not all intertidal flats flood again;
- produces less energy than an ebb generation scheme and a double-effect scheme;
- discontinuous energy production;
- more rapid head reduction and also the river discharge into the estuary results in a small contribution to the head reduction.

6.4.2 Single basin, double-effect mode of operation

Double-effect mode of operation means that energy is generated during approximately 85% of the tidal cycle. This is possible because this scheme combines both flood and ebb generation, see figure 78. Due to the fact that both ebb and flood generation are combined the head differences are smaller because ebb generation results in a higher low water level, while flood generation results in a lower high water level. As a result the water level variation in the basin resembles the natural variation the most, but on the other hand larger and more expensive turbines are required.

According to Prandle an ebb generation scheme harnesses approximately 27% of the energy potential, while a double-effect mode scheme harnesses 37% [Clark, 2007]. Duivendijk [2007] states that a double-effect mode scheme results in a profit of 2-10% relative to single-effect mode schemes with pumping, this is including the effect of different energy costs during peak hours and off-peak hours. In both cases only the energy yield is taken into account and not the additional costs of more expensive double regulated turbines. The advantage of these turbines is that they also are suitable for pumping, have a larger operating range and that the turbines are suited for sluicing. This means that no sluices have to be installed, what results in a cost reduction. However in order to maximize the energy yield, sluices are required as they are able to raise or lower the basin level more quickly.

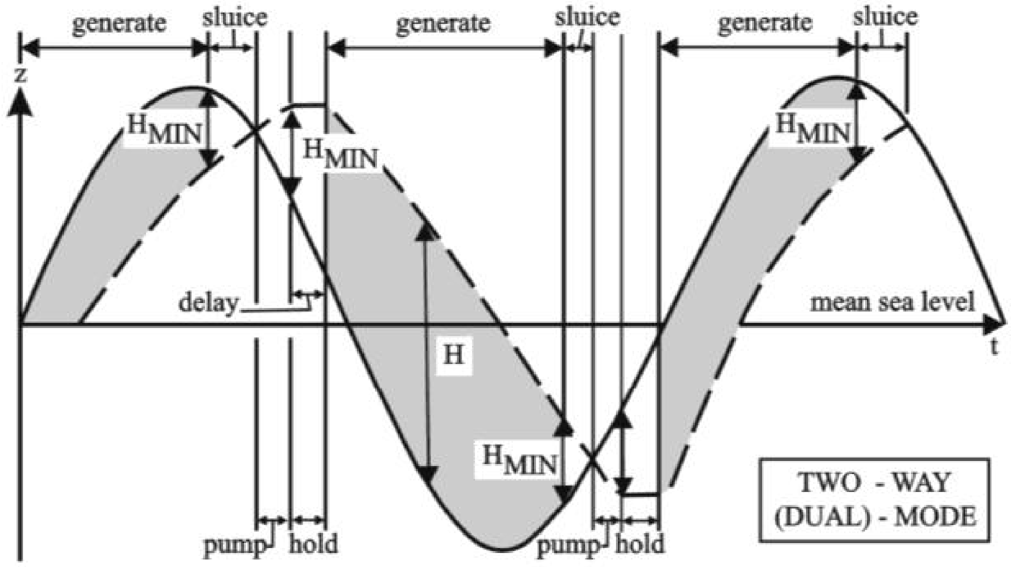


Figure 78: water level versus time during double-effect operation (Courtesy: *Wyre tidal energy*).

Advantages:

- larger energy production than both ebb and flood generation [Clark, 2007 & Duivendijk 2007];
- more or less continuous energy production during the day;
- water level variation within the basin resembles the natural tidal variation the most of all schemes;
- no sluices are needed.

Disadvantages;

- double regulated turbines are needed, resulting in more expensive turbines;
- a larger runner diameter is needed because of the smaller available head difference and thus more expensive turbines;
- as a result of the barrage ships have to negotiate an extra obstacle, while the tidal window is still in present.

6.4.2 Conclusion

Based on the (dis)advantages of the three possible schemes stated in the previous sections, a comparison is made taking into account the most important aspects concerning the construction of a tidal power plant within the Wash estuary. In table 51 the result of this comparison is presented.

Weighing factor	Ebb generation	Flood generation	2-way operation
Cost	++	++	+
Energy yield	+	--	++
Environmental aspects	--	+	++
Ship traffic	+	--	-
Total	**++**	**-**	**++**

++ = very good; + = good; +/- = fair; - = poor; -- = bad

Table 51: comparison between tidal range power schemes.

At this stage no information concerning cost levels of the different schemes is available. Therefore it is assumed that the construction cost of both an ebb generation scheme and a flood generation scheme will be similar, while a two-way operation scheme will be more expensive due to the higher turbine cost. As mentioned before an ebb generation scheme generates more energy than a flood generation scheme, especially since the Wash estuary is very shallow and includes approximately 29.770 ha of intertidal flats [Wash Estuary Strategie Group, 2004]. A two-way generation scheme harnesses approximately 10% more energy than an ebb generation scheme.

The expected negative impact on the environment is largest for an ebb generation scheme and smallest for 2-way operation as the water level variation for this configuration resembles the natural tide the most. The ebb generation scheme is the only scheme that is beneficiary with respect to ship traffic, since the average water depth is raised.

As can be seen in table 51 both an ebb generation scheme and a double-effect scheme have an equal score. According to Duivendijk [2007] experience gained with the La Rance scheme learned that single-sided operation is preferable to double-sided operation, while from an environmental point of view it is the other way around. Hence it comes down to a trade off between costs and environmental impacts. Therefore in the next section a preliminary design will be made for both an ebb generation scheme and a two-way generation scheme in order to establish which scheme is most suited in case of the Wash estuary.

6.5 Preliminary design tidal power plant

In this section it will be determined whether, in case of the Wash estuary, an ebb generation scheme or a two-way generation scheme is most suited at the selected site of the storm surge barrier (see chapter 5). The governing parameters ultimately leading to the preliminary design of a tidal power plant are presented in this section, while in appendix 12 all related parameters are included.

Table 52 presents an overview of either existing or planned large tidal power plants (TPP's) in the world, all schemes have double regulated bulb turbines. Except for the TPP in La Rance, which is a two-way operation scheme, all TPP's are ebb generation schemes.

TPP	Year	R_{mean} [m]	P_{inst} [MW]	D [m]	A_b [km^2]	L [km]	Cost 1)	Cost 2012 2) [£]
France								
La Rance3)	1966	8	240	5.35	22.5	0.75	₣ 620M	477M
*United Kingdom*4)								
Severn	n.a.	14	8640	9.0	480	16	£23,300M5)	25,200M
Solway Firth	n.a.	5.6	5580	9.0	860	30	£7,480M6)	18,436M
Mersey7)	n.a.	6.5	700	8.0	61	1.9	£3,200M	3,200M
South Korea 8)								
Sihwa	n.a.	5.6	250	7.5	43	12.7	$ 350M9)	248M
Garolim	n.a.	4.7	480	8.0	45.5	2	$1,000M10)	766M
Inchon	n.a.	5.3	1000	7.5	106	20	$2,500M11)	1,772M

1) Sources: http://nl.coinmill.com/FRF_calculator.html, exchange rate on January 1st 2002: € 1.00 equals ₣ 6.55957 and www.wisselkoersen.nl , exchange rate on February 8th 2012: € 1.00 equals £ 0.83 and $ 1.00 equals £ 0.63.
2) Price level 2012, using equation 5.3 and a real interest rate of 4%.
3) Source: http://en.wikipedia.org/wiki/Rance_Tidal_Power_Station.
4) Source: http://www.reuk.co.uk/Severn-Barrage-Tidal-Power.htm and DECC, 2010.
5) 2010 price level.
6) 1989 price level.
7) Source: http://www.merseytidalpower.co.uk/
8) Source: Lee, 2006.
9) The closure dam was already constructed in 1994, it is not clear whether or not, the construction cost of the dam are included in the amount presented. However Clarke mentions $250M as the construction cost for the tidal power plant [Clarke, 2007], therefore it seems safe to assume that the amount presented here includes also the construction cost of the tidal barrier. Price level 2009.
10) 2007 price level.
11) 2009 price level.

Table 52: existing and planned large tidal power plants.

6.5.1 Turbines

The runner diameter and generated power of a turbine depend on the rated head[53]. When the head over the TPP exceeds the rated head the guide vanes are gradually closed. Thus reducing the discharge, while keeping the generator at rated capacity[54]. In case the head drops below the rated head the capacity of the generator reduces and as a consequence the generated power decreases.

According to Song and van Walsum the rated head is accurately estimated using equation 6.1 [Song and van Walsum, 2006]:

$$H_r = C_{TPP} \cdot R_{mean} \qquad [6.1]$$

[53] Rated head = lowest head for which the turbine is capable of driving the generator at its rated capacity. Hence, the turbine guide vanes are opened to their maximum.
[54] Rated capacity = maximum power that the generator is allowed to produce.

Were:

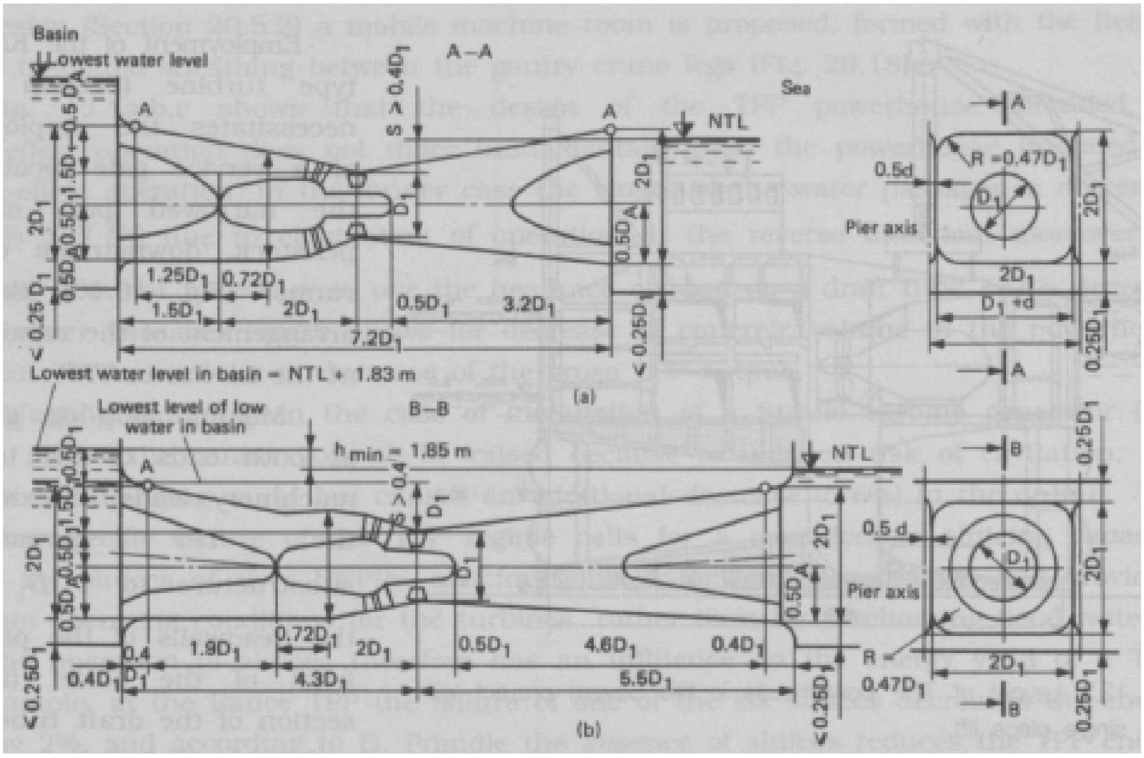
H_r	:	rated head	[m]
C_{TPP}	:	factor expressing the mode of operation of a TPP: - single effect mode of operation: $C_{TPP} = 0.66$ - double effect mode of operation: $C_{TPP} = 0.50$	[-]
R_{mean}	:	mean tidal range	[m]

In case of the Wash estuary the mean tidal range is 4.70 m. Hence, the rated head for both an ebb generation scheme and a two-way generation scheme is 3.10 m and 2.35 m respectively. In reality the head difference over the barrier is likely to vary between 65% and 125% of the rated head[55]. However it is not economical to design the TPP based on the mean spring tidal range, because the corresponding head difference will only occur during 20%[56] of the total time during one year. This would lead to the installation of too many turbines.

Runner diameter

The runner diameter is an important parameter in the design of a TPP because:

- the electromechanical equipment generally accounts for 45% to 55% of the direct costs of the TPP. Hence, the number of turbines needed, significantly influences the economy of the TPP scheme;
- it has a large influence on the civil engineering costs, as the dimensions of the turbine governs the dimensions of the power house, see figure 79;
- the turbine discharge capacity determines the number and size of the sluices.



Taken from Bernshtein, 1996.

Figure 79: overall dimensions of the power house setting in case of a bulb unit.
(a) single effect operation (b) double effect operation.

[55] Source: Indian Institute of Technology Kharagpur.

[56] Spring tide occurs each 14.765 days, hence 24.72 times a year. Assuming a duration of 3 days, during which the tidal range is equal to or larger than the mean spring tidal range, results in 24.72·3/365 = 0.20.

Historically the effort to reduce unit costs has lead to an increase in turbine size. According to manufactures of turbines, a diameter of 9 to 10 m is considered to be a reasonable extension of existing knowledge and technology [Clarke, 2007]. Generally larger turbines tend to have a higher turbine and generator efficiency. Therefore it seems logical to install a limited number or turbines with a large runner diameter. It should be kept in mind however, that the required submergence in order to avoid cavitation should be available without the need for excavation.

Due to the fact that the main channel in the Wash estuary is deep and very wide, the selection of a large runner diameter is economical. With increasing runner diameter the power generated also increases (larger discharge), resulting in decreasing cost per kW. Furthermore the number of required turbine caissons decreases. This is beneficial due to the fact that the longitudinal axis of these caissons is orientated perpendicular to the barrier line, while the sluice caissons and sluiced caissons have a longitudinal axis parallel to the barrier line. Based on preliminary calculations, the required number of turbines will be so large that the TPP's flexibility is not at risk.

The relation between the runner diameter of the turbine (D_1) and the required Bottom of Structure (BoS), based on figure 79, is presented in table 12.1 in appendix 12. In case of an ebb generation scheme the BoS is located $2.25 \cdot D_1$ beneath Mean Low Water Spring (-2.00 mODN, see table 11 in section 2.1.3), while in case of a two-way generation scheme this is $2.50 \cdot D_1$ below Mean Low Water Spring. Based on the bottom profile at the location of the most suitable barrier line, see figure 58, a runner diameter of 8.0 m is selected for both schemes.

<u>*Generated power*</u>
During every one year not all tidal cycles can be used to generate energy. The main reasons for not using a tidal cycle are; maintenance, (electro) mechanic failure and severe storm conditions. According to Clarke every year 3-5% of the tidal cycles are lost for the generation of energy [Clarke, 2007]. So on average 96% of the annual tidal cycles is used to generate energy, this amounts to $0.96 \cdot 705.50 = 677.30$ tidal cycles per annum. The power generated per year can be estimated by means of equations 6.2 and 6.3 [Duivendijk, 2007].

$$P = \eta \cdot \rho \cdot g \cdot H_{av} \cdot Q_{av} \qquad [6.2]$$

Were:

P	:	power generated per tidal cycle	[W]
η	:	plant efficiency	[-]
ρ	:	volumetric density of water	[kg/m^3]
g	:	gravitational acceleration	[m/s^2]
H_{av}	:	average head per tidal cycle	[m]
Q_{av}	:	average discharge per tidal cycle	[m^3/s]

And

$$Q_{av} = \frac{V_{av}}{\varepsilon \cdot T_{tide}} \qquad [6.3]$$

Were:

Q_{av}	:	average discharge per tidal cycle	[m^3/s]
V_{av}	:	average volume of water per tidal cycle	[m^3]
ε	:	fraction of tidal cycle during which TPP is operational	[-]
T_{tide}	:	duration of the tidal cycle, M_2-tide: $T_{tide} = 44700$ s	[s]

In figures 80 and 81 a schematic representation of the operation of the TPP during a tidal cycle is presented for both an ebb generation scheme and a two-way generation scheme. The area hatched with red represents the period during which energy is generated from the tide.

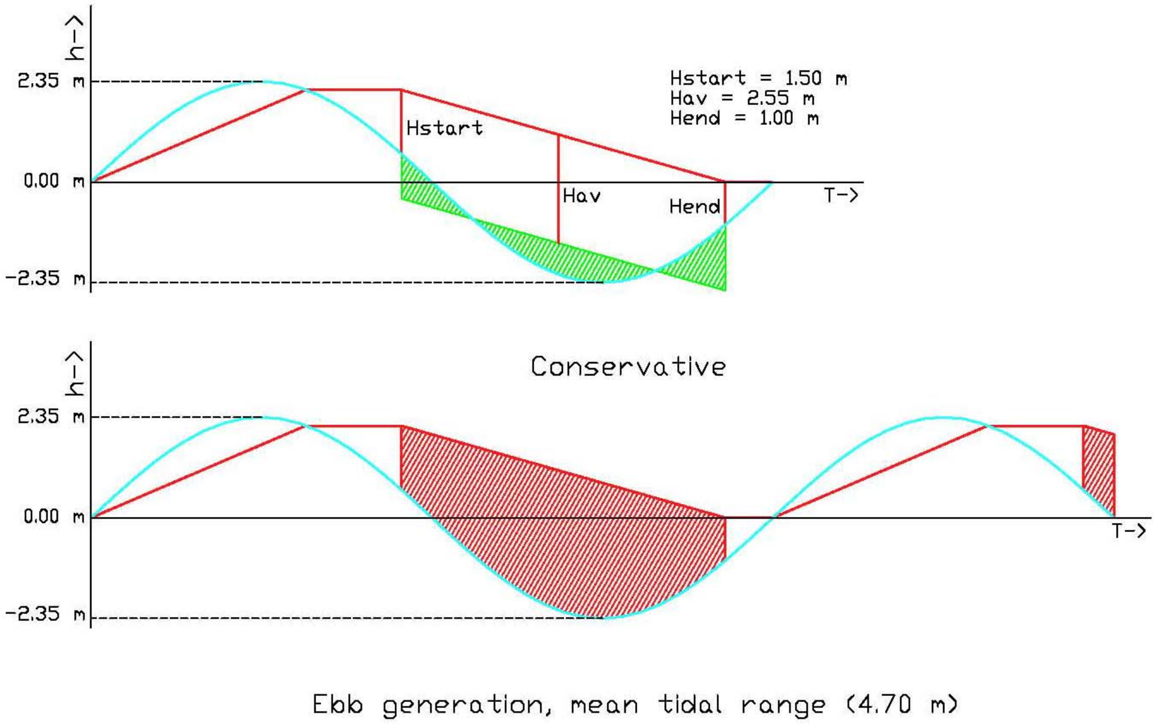


Figure 80: schematic representation of TPP operation in case of an ebb generation scheme (not to scale).

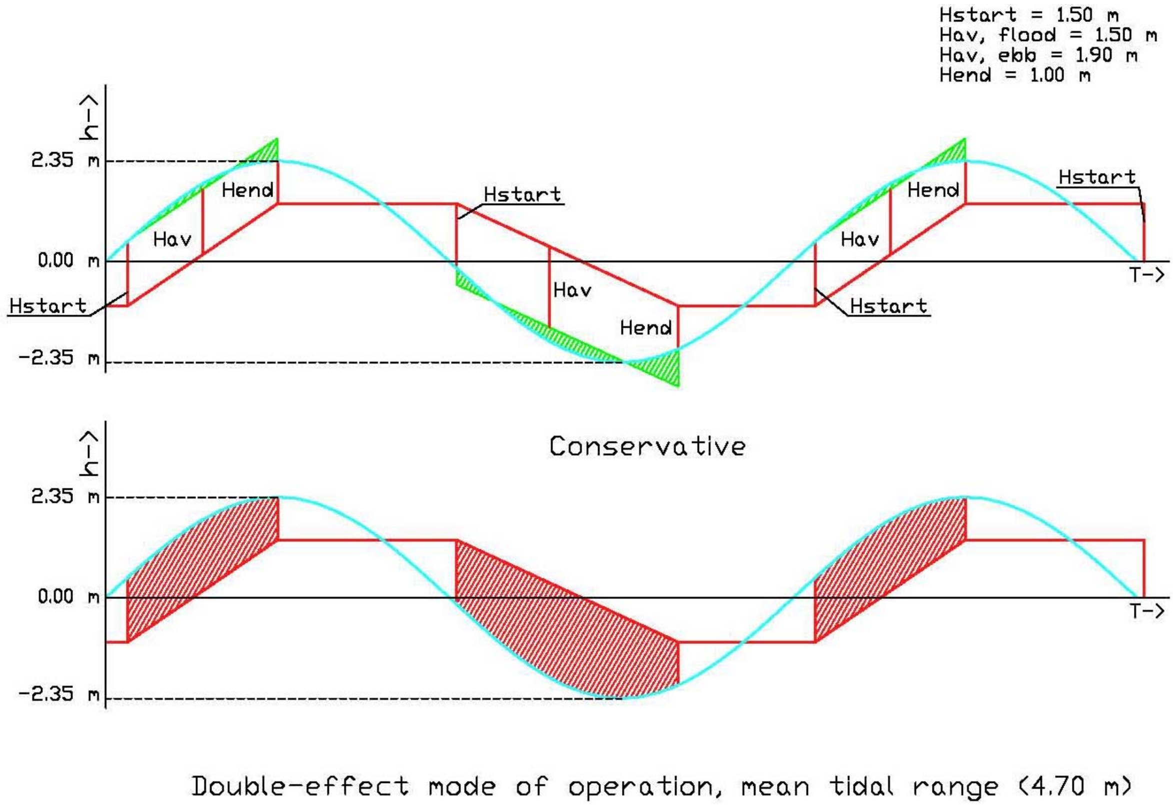


Figure 81: schematic representation of TPP operation in case of a two-way generation scheme (not to scale).

According to literature the fraction of the tidal cycle during which a TPP is operational is 0.50 in case of an ebb generation scheme and 0.85 in case of an two-way scheme [Clarke, 2007 and Bernshtein, 1996]. From figures 80 and 81 it is determined that in case of an ebb generation scheme the operational time frame of the TPP corresponds indeed to 50% of the tidal cycle, but in case of a two-way generation scheme the operational time frame is only 55% of the tidal cycle[57].

In table 53 an overview is given of all basin parameters required to estimate the annual energy yield and the generated power per tidal cycle, for more detailed information the reader is referred to appendix 12.

Basin parameter	Unit	Ebb generation	2-way operation	
			Ebb mode	Flood mode
Average head per tidal cycle	[m]	2.55	1.90	1.50
Average water level variation	[m]	2.15	2.40	2.40
Average basin area	[m^2]	$355 \cdot 10^6$	$355 \cdot 10^6$	$355 \cdot 10^6$
Average volume of water per tidal cycle	[m^3]	$763 \cdot 10^6$	$852 \cdot 10^6$	$852 \cdot 10^6$
fraction of tidal cycle during which TPP is operational	[-]	0.50	0.33	0.22
Average discharge per tidal cycle	[m^3/s]	$34.15 \cdot 10^3$	$57.76 \cdot 10^3$	$86.64 \cdot 10^3$

Table 53: overview basin parameters.

The generated power per tidal cycle, as computed using equations 6.2 and 6.3, is presented in table 54. From the table can be concluded that the power generated per tidal cycle in case of a two-way generation scheme is approximately two times as large as for an ebb generation scheme. However, a two-way scheme requires a much larger number of turbines, while on the other hand no sluices will be required. Therefore a fair comparison is only possible based on the estimated costs, see section 6.6.4.

Type of scheme	Ebb generation [MW]	Flood generation [MW]	Total [MW]
Ebb generation	700	n.a.	700
2-way generation	828	980 (552) [1)]	1808 (1380) [1)]

[1)] As a result of the assumption made with respect to the average basin area the computed value is unrealistic. It is assumed that the power generated during flood generation mode equals 2/3 of the power generated during ebb generation mode. These values are shown between brackets.

Table 54: estimation of the power generated per tidal cycle.

Note that the power generated during flood generation mode is 154 MW larger than during ebb generation mode, in case of the two-way generation scheme. This is a direct result of the assumption made with respect to the average basin area (see appendix 12). In reality this will be the other way around because during flood generation the highest head occurs when the basin level is lowest and as a consequence a large part of the basin area consists of sand and mudflats that have fallen dry. Hence the water level in the basin will rise faster during the beginning of flood generation than near the end of the operation. During ebb generation mode

[57] During 33% of the tidal cycle, the tidal power plant is in ebb generation mode. And during 22% of the duration of the tidal cycle the tidal power plant is in flood generation mode.

the opposite occurs, the basin's water level will fall relatively slow during the early stages, when the largest head differences are present, compared to the end of operation when the sand and mud flats fall dry again. Therefore in case of a two-way generation scheme the power generated during ebb generation mode will be assumed governing with respect to the design of the tidal power plant.

The estimated amount of power generated per year can be computed using equation 6.4.

$$P_{annum} = N_{tide} \cdot P \qquad [6.4]$$

Were:

P_{annum}	:	power generated per year	[GW]
P	:	power generated per tidal cycle	[GW]
N_{tide}	:	number of energy generating tides per year	[-]

In case of an ebb generation scheme the annually generated power amounts to 474 GW, for a two-way generation scheme this is 935 GW.

In table 55 an overview is given of all turbine parameters as computed in the appendix 12. The in the appendix performed check on cavitation, shows that no cavitation problems are to be expected.

Turbine parameter	Symbol	Unit	Ebb generation	2-way operation
Rated head	H_r	[m]	3.10	2.35
Runner diameter	D	[m]	8.0	8.0
Plant efficiency	η	[-]	0.80	0.75
Rated discharge	Q_r	[m^3/s]	328	267
Rated power per turbine	P_{rated}	[MW]	9.70	6.00
Design discharge	Q_d	[m^3/s]	298	240
Power per turbine	P_d	[MW]	7.24	4.35
Specific speed	n_q	[rpm]	456.30	522.04
Operating speed	n	[rpm]	58.82	60.00
Annual energy yield	E_{annum}	[GWh]	2945	3320
Annually generated power	P_{annum}	[GW]	474	935
Installed power	P_{inst}	[MW]	940	1140
Throat area	A_t	[m^2]	50	50
Flow velocity	u_d	[m/s]	6.5	5.3
Number of turbines	n_t	[-]	97	190

Table 55: summary turbine parameters.

In figures 82 and 83 a cross-section and frontal view of the turbine caisson for an ebb-generation scheme are depicted respectively. The overall dimensions of the power house are based on figure 79 *and* the power house design of existing and planned tidal power plants all over the world. The figures should be interpreted as artist impressions and not passed as a fully-fledged design.

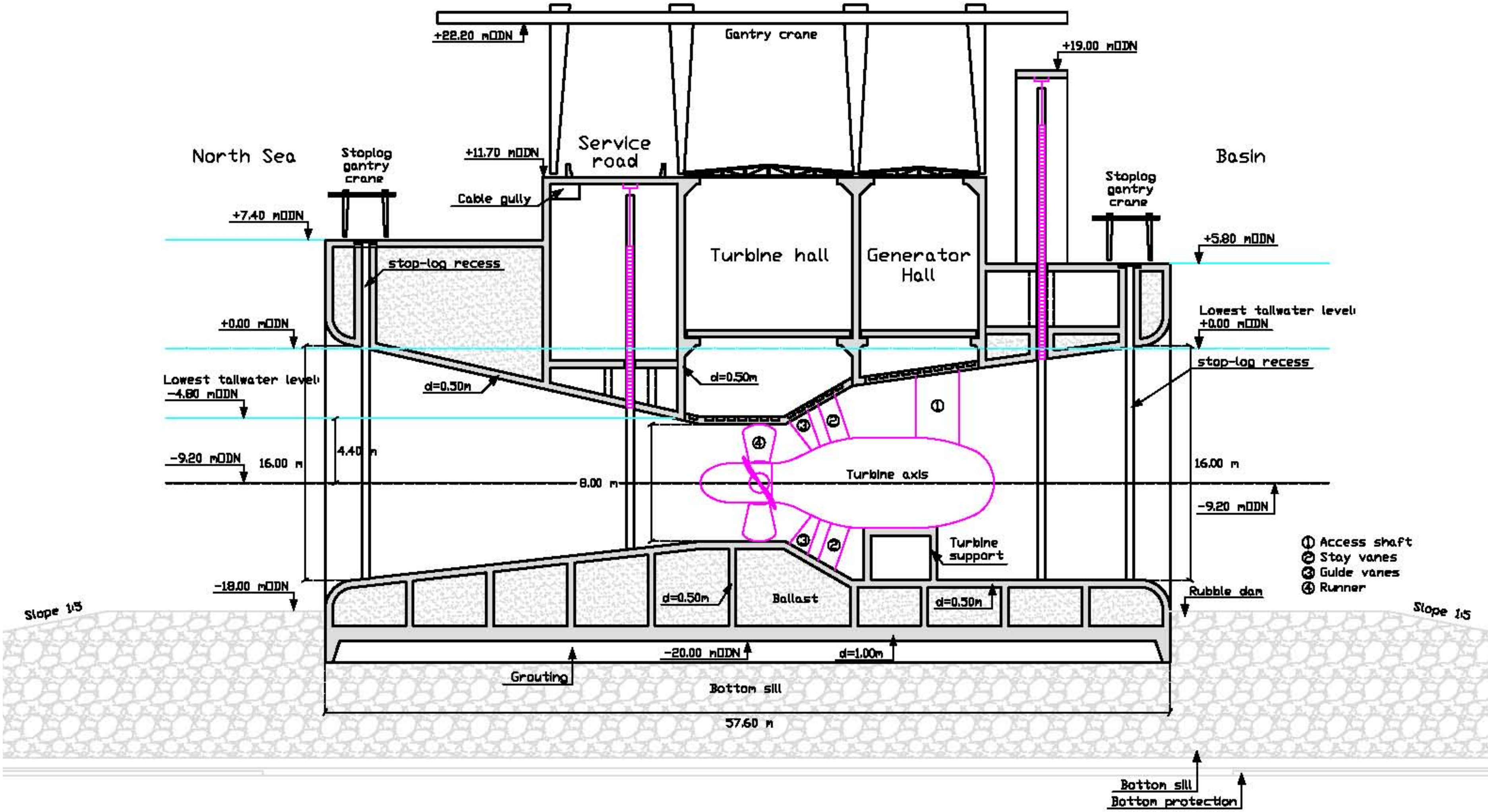


Figure 82: cross-section turbine caisson (not to scale).

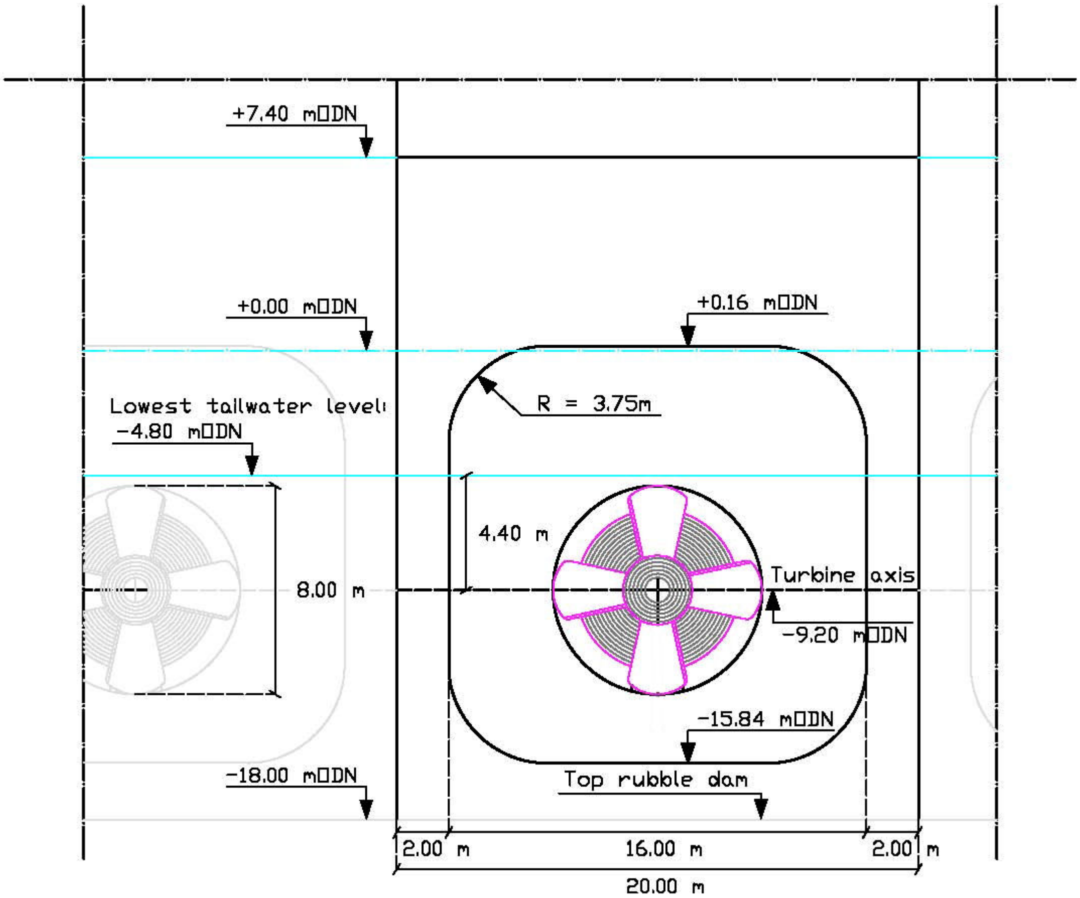


Figure 83: front view turbine caisson (not to scale).

6.5.2 Sluices

In case an ebb generation scheme is selected, sluices are needed in addition to the turbines in order to be able to fill the basin again during rising tide. Since these sluices contribute to the total cost of the TPP and therefore the cost/kWh, the number of sluices required is determined in this section.

In section 6.3.3 it was already concluded that tainter gates are most suited with respect to this specific project. The gate dimensions as presented in table 56 are based on the dimensions of existing tainter gates applied by the U.S. Army Corps of Engineers [U.S.A.C.E, 2000; appendix D]. A cross-section of the gate configuration is presented in figure 84, while a front view is depicted in figure 85. Again both figures should be interpreted as an artist impression and not passed as a fully-fledged design.

The mean tidal range within the Wash estuary is 4.70 m. Therefore the maximum water level reached during a tidal cycle is 2.35 m above Mean Sea Level (0.00 mODN). Hence during rising water the average tidal level at the North Sea is approximately 1.15 mODN. This results in an average water level of 3.15 m above the sluice's floor level. Since it is assumed that a basin storage approach is valid, the average water level at the basin side will not differ much from that at the North Sea. On average a difference of 0.25 m is assumed to be present. Hence the average water level on the basin side of the sluice gate is 2.90 m above floor level.

Dimension of sluice gate	Unit	Single effect operation
Width	[m]	20.00
Height	[m]	10.00
Gate radius	[m]	12.00
Floor level	[mODN]	-2.00
Dimension of sluice caisson	**Unit**	**Single effect operation**
Height trunnion above floor level	[m]	8.00
Pier thickness	[m]	5.00
Length sluice caisson	[m]	80.00
Width sluice caisson	[m]	26.00
Top of structure (gate)	[mODN]	+8.00
Bottom of Structure	[mODN]	-10.00

Table 56: overall dimensions of a sluice gate and sluice caisson.

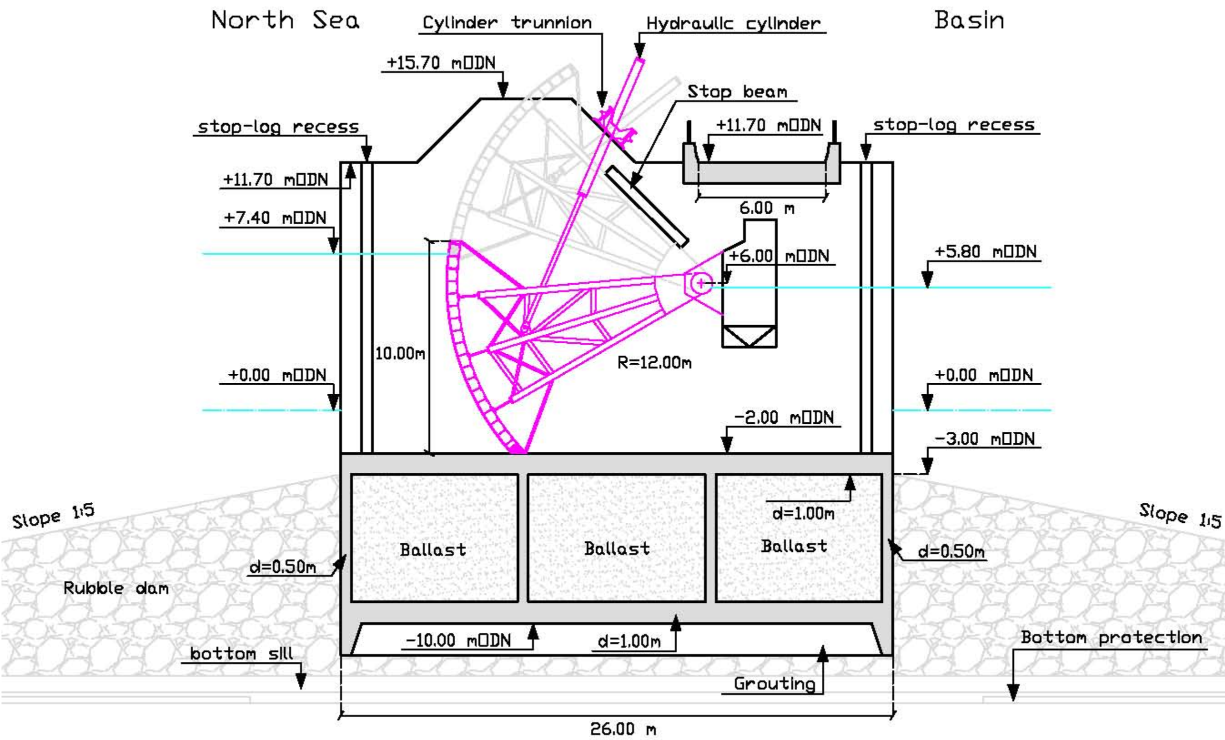


Figure 84: cross-section sluice gate configuration (not to scale).

The sluice opening may be regarded as a broad-crested submerged weir the average discharge through one single sluice opening can be computed using equation 6.5 [Nortier and de Koning, 1991].

$$Q_{d,sl} = m \cdot B \cdot h_b \cdot \sqrt{2 \cdot g \cdot (h_{NS} - h_b)} \qquad [6.5]$$

Were:

$Q_{d,sl}$	:	design discharge through sluice	[m^3/s]
m	:	discharge coefficient	[-]
B	:	width of the sluice opening	[m]
g	:	gravitational acceleration	[m/s^2]
h_b	:	average water level in the basin	[m above sluice floor]
h_{NS}	:	average water level on the North Sea	[m above sluice floor]

The discharge coefficient (*m*) for the gate configuration depicted in figure 84 turned out to be 0.80, see appendix 13 for the derivation. Hence, the design discharge through one sluice opening, computed with equation 6.5, is:

$$Q_{d,sl} = 0.80 \cdot 20 \cdot 2.90 \cdot \sqrt{2 \cdot 9.81 \cdot (3.15 - 2.90)} \approx 103 \text{ m}^3/\text{s}$$

During idle discharge[58] the turbine's sluicing capacity is approximately 70% of the discharge capacity during generation mode [Bernshtein, 1996]. With an design discharge of 298 m^3/s per turbine (see appendix 12, section 12.11), the idle discharge per turbine is 0.7·298 = 209 m^3/s.

Because the average discharge per mean tidal cycle, $34.15 \cdot 10^3$ m^3/s, must be allowed into the basin during rising water by both the turbines and sluices, the number of sluices required can be determined by means of equation 6.6.

$$n_{sl} = \frac{Q_{av} - n_t \cdot Q_{d,idle}}{Q_{d,sl}} \qquad [6.6]$$

Were:

n_{sl}	:	number of sluices	[-]
Q_{av}	:	average discharge per tidal cycle	[m^3/s]
n_t	:	number of turbines	[-]
$Q_{d,idle}$	:	average idle discharge through turbine	[m^3/s]
$Q_{d,sl}$	:	average discharge through sluice	[m^3/s]

The number of sluices required when the tidal power plant is an ebb generation scheme, becomes:

$$n_{sl} = \frac{34{,}150 - 97 \cdot 209}{103} = 134.73 \Rightarrow 135$$

[58] Idle discharge = discharge through the turbines when in orifice mode.

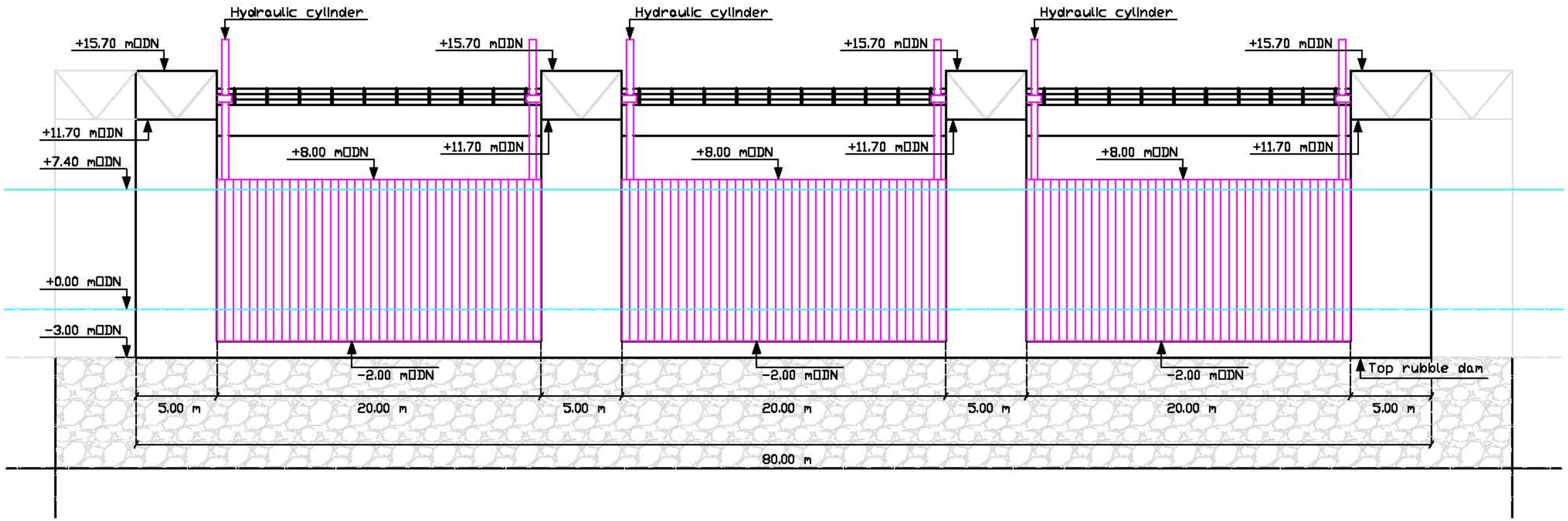


Figure 85: front view sluice caisson (not to scale).

6.6 Cost estimation

In this section first the cost of the electromechanical equipment is estimated, after which is determined which scheme is most suited in the Wash estuary. In chapter 8 the total cost of the tidal power plant, including the barrier are estimated using both the construction cost of similar tidal power plants as stated in table 52 and based on the bills of quantities included in appendix 14

6.6.1 Electromechanical equipment

The cost of the turbine, generator and electronic peripheral equipment is estimated by means of equation 6.7. This equation originally is the result of a regression analysis performed by Swane[59] on the cost of electromechanical equipment installed in tidal power plants in China and South Korea. Mooyaart[60] adapted the equation to account for the higher labour cost in The Netherlands. The equation presented below is the equation derived by Mooyaart adapted to the 2012 price level. The indexing with a real interest rate of 4% and the conversion to Pound Sterling[61] has resulted in slightly different factors.

$$C_t = 7.62 \cdot 10^6 + 1.64 \cdot 10^5 \cdot n_t \cdot H_r^{0.18} \cdot D^2 \qquad [6.7]$$

Were:

C_t	:	cost electromechanical equipment	[£]
n_t	:	number of turbines	[-]
H_r	:	rated head	[m]
D	:	turbine diameter	[m]

Hence in case of an ebb generation scheme the estimated cost of the electromechanical equipment amount to:

$$C_t = 7.62 \cdot 10^6 + 1.64 \cdot 10^5 \cdot 97 \cdot 3.10^{0.18} \cdot 8^2 = 1.25 \cdot 10^9 \text{ £}$$

The cost of the electromechanical equipment in case of a two-way generation scheme is estimated to be:

$$C_t = 7.62 \cdot 10^6 + 1.64 \cdot 10^5 \cdot 190 \cdot 2.35^{0.18} \cdot 8^2 = 2.33 \cdot 10^9 \text{ £}$$

6.6.2 Sluice and turbine gates

The cost of the sluice and turbine gates are based on a cost estimation of a vertical lift gate made by the UKAEA[62] in the 1980's, used to assess the economic feasibility of tidal power plants in the UK. According to the UKAEA the cost are proportional to the gate's area [Burrows R., 2008, appendix 3]. It is assumed that the cost of a vertical lift gate and a tainter gate are comparable (see section 6.3.3). Equation 6.8 is used to compute the cost per gate type for the Wash estuary tidal power plant, the results are presented in table 57.

[59] Tidal Power Plant in Saemangeum, MSc-thesis at Delft University of Technology by Hugo Swane, May 2007.
[60] De energiepolder, MSc-thesis at Delft University of Technology by Leslie Mooyaart, June 2009.
[61] Source: www.wisselkoersen.nl, exchange rate on February 8th 2012: € 1.00 equals £ 0.83.
[62] UKAEA = UK Atomic Energy Authority.

$$C_{gate} = \frac{A_{gate}}{A_{ref}} \cdot C_{ref} \qquad [6.8]$$

Were:

C_{gate}	:	cost sluice or turbine gate	[£]
A_{gate}	:	area new gate	[m^2]
A_{ref}	:	area existing gate	[m^2]
C_{ref}	:	cost existing gate	[£]

	Gate type	Gate area [m^2]	Cost, 1980 [10^6 £]	Cost, 2012 [1)] [10^6 £]
Reference gate	vertical lift	12·12	0.33	1.16
Turbine gate	vertical lift, sea side	12·12	-	1.16
	vertical lift, basin side	16·14	-	1.80
Sluice gate	tainter gate	10.75·20	-	1.73
	vertical lift, small caisson	15·6	-	0.73
	vertical lift, medium caisson	15·7	-	0.85
	vertical lift, large caisson	15·11	-	1.33

[1)] indexation with a real interest rate of 4%.

Table 57: cost sluice and turbine gates.

Each sluice has one tainter gate and each turbine has two vertical lift gates to close off the draft tube, therefore the total cost for the gates can be computed using equation 6.9 for both types of schemes.

$$C_{gates} = n_t \cdot C_{gate,t} + n_{sl} \cdot C_{gate,sl} \qquad [6.9]$$

Were:

C_{gates}	:	total cost tainter and vertical lift gates	[£]
n_t	:	number of turbines	[-]
$C_{gate,t}$	:	cost vertical lift gate	[£]
n_{sl}	:	number of sluice	[-]
$C_{gate,sl}$	:	cost tainter gate	[£]

Hence, for an ebb generation plant the combined cost of the sluice and turbine gates amount to: $C_{gates} = 97 \cdot 1.16 \cdot 10^6 + 97 \cdot 1.80 \cdot 10^6 + 135 \cdot 1.73 \cdot 10^6 = 520.67 \cdot 10^6$ £. And in case of an two-way generation scheme: $C_{gates} = 190 \cdot 1.16 \cdot 10^6 + 190 \cdot 1.80 \cdot 10^6 = 562.40 \cdot 10^6$ £.

6.6.3 Revenues from generated energy

In section 3.4 it was already stated that the levelised energy cost must lie within a price range of 8-11 p/kWh. Hence, it is assumed that the revenues from the generated energy amount to 9.5 p/kWh, being the mean of the price range. The estimated annual revenues of both the ebb generation scheme and the two-way generation scheme are presented in table 58.

Type of scheme	Annual energy yield [kWh]	Annual revenues [10^6 £]
Ebb generation	2945·10^6	279.78
2-way generation	3320·10^6	315.40

Table 58: estimation of the annual revenues from a tidal power plant.

6.6.4 Cost comparison

In table 59 the summation of costs and benefits is presented for both the ebb generation scheme and the two-way generation scheme. From the table can be concluded that an two-generation scheme is a factor 1.7 more expensive than an ebb generation scheme, not taking into account the cost of the caissons, navigation locks and embankment dam. As the two-way generation scheme requires much more turbine caissons (which are positioned perpendicular to the barrier line), in reality the difference in cost between both schemes will be even larger.

Cost factor	Cost ebb generation scheme [10^6 £]	Cost two-way generation scheme [10^6 £]
Turbines	1250.00	2330.00
Sluice and turbine gates	520.67	562.40
Revenues from energy	-279.78 +	-315.40 +
Total	**1491**	**2577**

Table 59: cost comparison between schemes.

6.6.5 Multi-criteria assessment

With respect to a tidal power plant within the Wash estuary the following aspects are most important in determining the most suitable scheme:
- cost;
- energy yield;
- environmental impact;
- ship traffic.

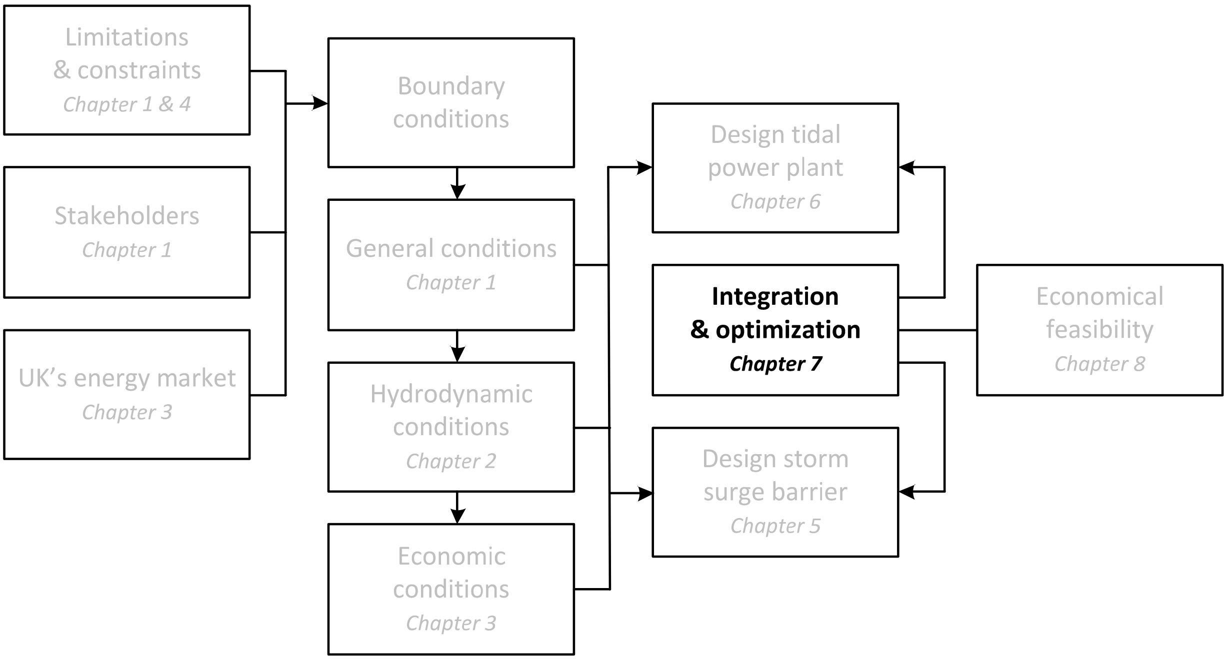
In table 60 the results of the multi-criteria assessment is presented. From the results can be concluded that the ebb generation scheme has the best score. However this is mainly based on the rough cost estimation presented in section 6.6.4, also the environmental impact and impact on ship traffic is based on qualitative arguments that are not supported by hard figures. Therefore the decision to design an ebb generation scheme is somewhat arbitrary.

Weighing factor	Ebb generation	2-way operation
Cost	++	--
Energy yield	+	++
Environmental impact	-	+
Ship traffic	+	-
Total	**++**	**+/-**

++ = very good; + = good; +/- = fair; - = poor; -- = very poor

Table 60: comparison between tidal range power schemes.

The expected negative impact on the environment is largest for an ebb generation scheme and smallest for 2-way operation, as the water level variation for this configuration resembles the natural tide the most. The ebb generation scheme is the only scheme that is beneficiary with respect to ship traffic, since the average water depth within the basin is raised significantly.


Limitations
& constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary
conditions
General conditions
Chapter 1
Hydrodynamic
conditions
Chapter 2
Economic
conditions
Chapter 3
Design tidal
power plant
Chapter 6
Integration
& optimization
Chapter 7
Design storm
surge barrier
Chapter 5
Economical
feasibility
Chapter 8

7 INTEGRATION AND OPTIMIZATION

In this chapter both the preliminary designs of the storm surge barrier and tidal power plant will be integrated. Furthermore optimization of the design of the storm surge barrier crossing the deep channels within the estuary will be considered in more detail. To optimize the number of sluices and turbines of the tidal power plant a storage basin approach will be followed to determine the influence of the structure on the water levels within the basin.

First the design of the sections of the combined structure crossing the deep channels within the Wash estuary will be optimized. Next the results of the storage basin approach will be discussed, as are the consequences for the tidal power plant. Finally the final conceptual design will be presented.

7.1 Optimization storm surge barrier

On the tidal flats constructing an embankment dam is not a problem, in case of the deep tidal channels the flow velocity is expected to cause some problems. For the closure of the Boston and Lynn Deeps there are three options available:
- a sand closure;
- a stone closure;
- a caisson closure.

As a result of the large tidal prism the flow velocities in the closure gap are expected to be much larger than 2.00-2.50 m/s, thus rendering a sand closure impossible as the mentioned flow velocities form an upper limit for such closure [RWS, 1992]. Another possibility is a stone closure. However this type is ruled out because of the large amount of material required in comparison to the underwater sill needed in case of a caisson closure. Other advantages of a caisson closure are:

- the structures footprint is reduced, thus reducing the environmental impact within the Wash estuary;
- with caissons the duration of the closure works is reduced;
- high flow velocities in the final closure gap as a result of the large tidal prism makes gradual closure difficult. With sluiced caissons, see figures 86 and 87, a large part of the water movement remains possible and the estuary can be closed of suddenly during the turning of the tide.

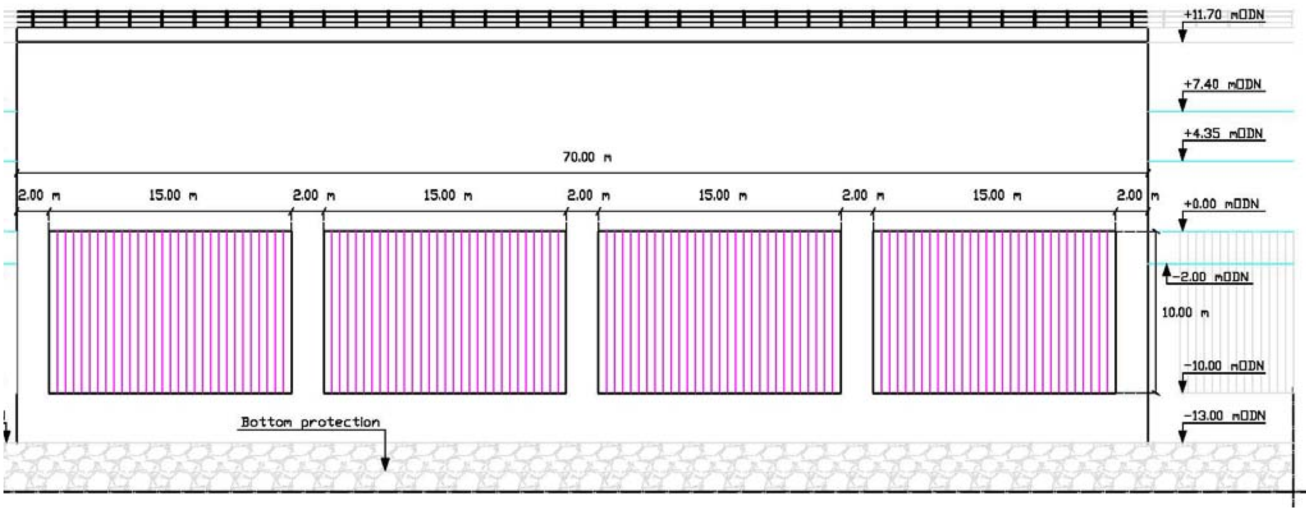


Figure 86: front view sluiced caisson Lynn Deeps (not to scale).

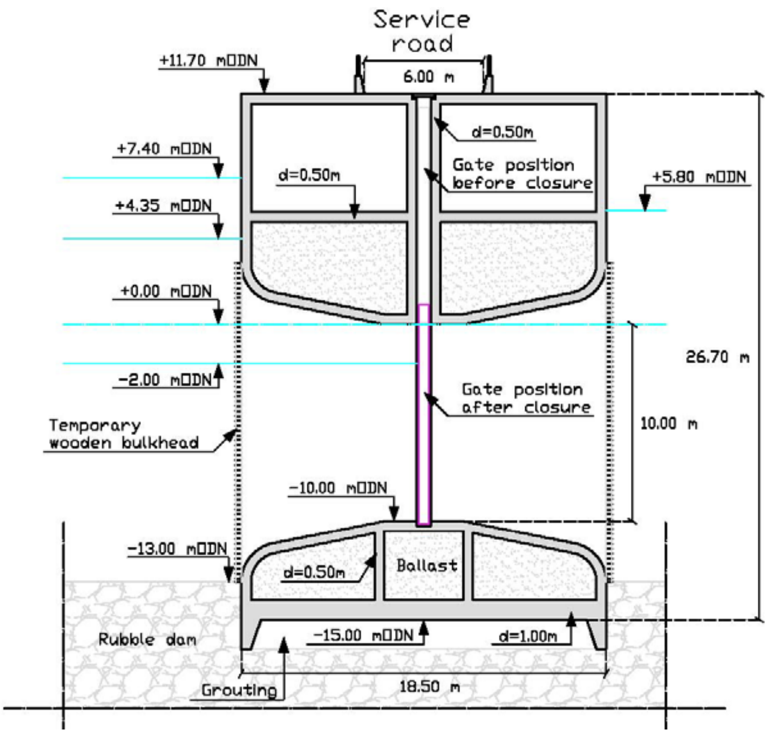


Figure 87: front view sluiced caisson Lynn Deeps (not to scale).

Taking into account the above considerations, the preliminary design that was derived in chapter 5 will have a caisson core. The question arises: "Would it not more economic to construct larger caissons[63] instead of building an embankment dam around the smaller caissons[64]". In order to be able to answer this question, for both an embankment dam with a caisson core *and* a caisson dam with stretches of embankment dam crossing the tidal flats a bill of quantities is produced, see appendix 14. The construction costs are determined using the unit prices as presented in table 61.

Description	Unit price [£/m^3]	Unit price [10^3 £/m^1]	Unit price [10^6 £]
Embankment dam, dry	15		
Embankment dam, wet	30		
Revetment	25		
Ballast material	20		
Rock	65		
Reinforced concrete	335		
Sluice caisson		167.60	
Turbine caisson		1170.83	
Abutment caisson		91.63	
Small sluiced caisson		107.56	
Medium sluiced caisson		117.47	
Large sluiced caisson		146.65	
Commercial lock complex			54.05
Recreational lock complex			25.11

Table 61: unit prices used to estimate the total construction costs.

[63] Top of Structure: +11.70 mODN.
[64] Top of Structure: +7.40 mODN.

In case of the embankment dam the volume of the caissons is subtracted from the total volume per cross-section. In both cases the costs of the sluiced caissons, turbine caissons, sluice caissons and navigation locks are determined separately and added to the cost of the respective cross-section. In table 62 both the estimated construction costs and the total volume of material required are presented. From the table can be concluded that constructing a caisson dam requires 12.97M m^3 less material and reduces the construction costs by £ 380M. Therefore the caisson dam is to be constructed, see figure 88 for the preliminary design.

Section	Embankment dam		Caisson dam	
	Volume [10^6 m^3]	Costs [10^6 £]	Volume [10^6 m^3]	Costs [10^6 £]
Connecting stretches	17.72	465	17.72	465
Boston Deeps	1.94	400	1.51	387
Lynn Deeps	18.30	3853	5.76	3486
Total	**37.96**	**4011**	**24.99**	**3631**

Table 62: comparison volume and costs embankment dam vs. caisson dam

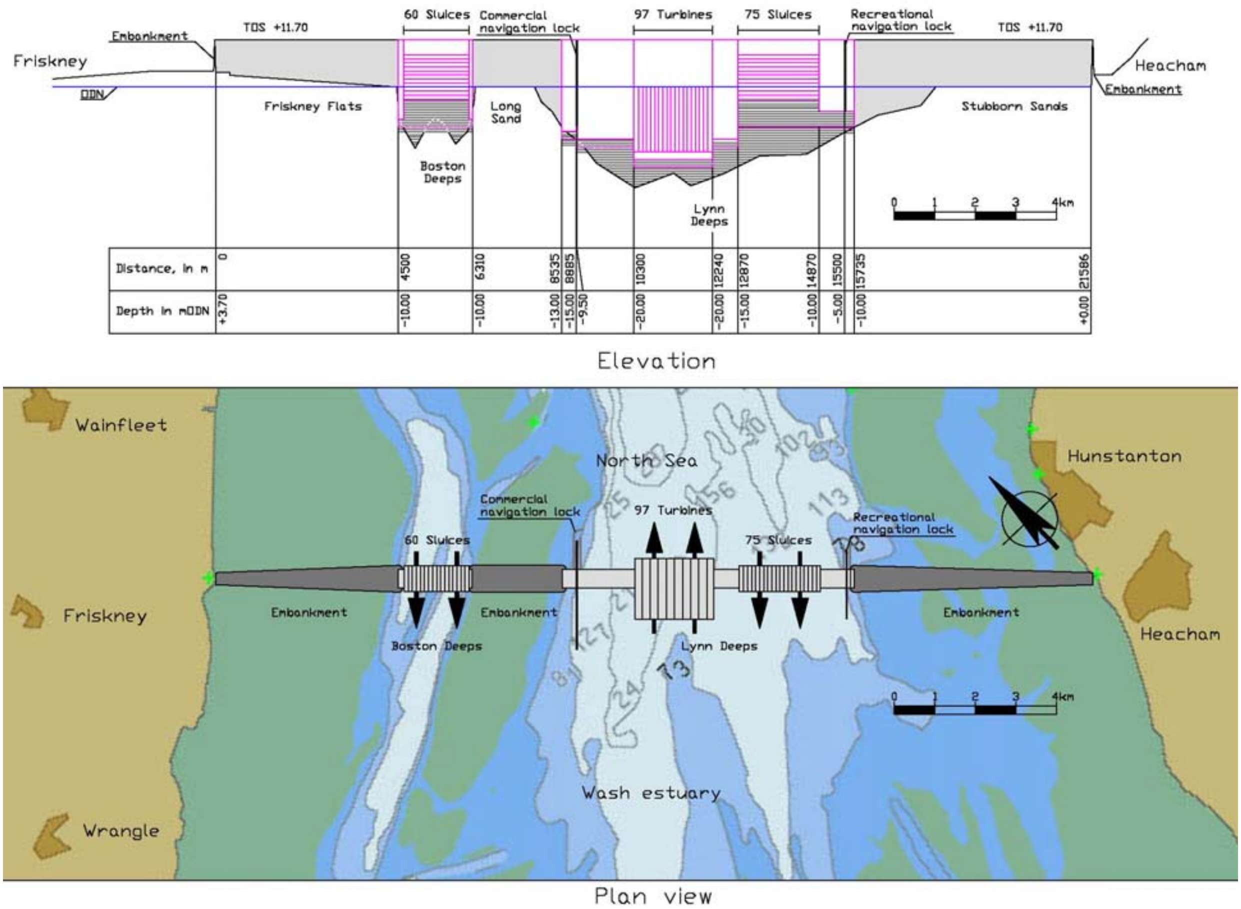


Figure 88: preliminary design of the combined storm surge barrier and tidal power plant in the Wash estuary.

7.2 Optimization tidal power plant

Constructing a combined storm surge barrier and tidal power plant within the Wash estuary is bound to influence the amplitude of the water level variation behind the barrier. In this section the St. Vernant or shallow water equations will be used to determine the effects of the barrier on the tidal amplitude within the Wash basin. This is relevant with respect to the optimisation of the sluicing capacity of the tidal power plant. The purpose of the performed analysis is to see what sluicing capacity is required for the amplitude of the basin water level to resemble the amplitude of the water level variation at the North Sea as much as possible.

The theoretical basis for analysing and performing computations with respect to the behaviour of long waves in shallow water are the continuity equation (equation 7.1) and the equation of motion (equation 7.2), together also referred to as the St. Venant equations or shallow water equations. These equations represent a coupled system of differential equations describing the relation between water level and discharge as function of time and distance.

$$B \cdot \frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{7.1}$$

And

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A_s}\right) + g \cdot A_s \cdot \frac{\partial h}{\partial x} + c_f \cdot \frac{|Q| \cdot Q}{A_s \cdot R} = 0 \tag{7.2}$$

Were:

B	:	storage width	[m]
h	:	water level	[m]
t	:	time	[s]
Q	:	discharge	[m^3/s]
x	:	distance	[m]
A_s	:	current-carrying cross-section	[m^2]
c_f	:	friction coefficient	[-]
R	:	hydraulic radius	[m]

As is shown in appendix 15 the basin can be regarded as a storage basin, while the connection through the barrier only has a transport function. Therefore a storage basin approach may be applied for the analysis of the problem. Thus simplifying the computations, as the resistance dominates over inertia in the connection, the latter may be ignored. Hence after eliminating the discharge from equations 7.1 and 7.2, the governing equation becomes:

$$h_{NS}(t) - h_b(t) = \tau \cdot \frac{dh_b}{dt} \tag{7.3}$$

Were:

h_{NS}	:	water level on North Sea	[m]
h_b	:	basin water level	[m]
t	:	time	[s]
τ	:	relaxation time	[s]

A MATLAB routine is used to compute the variation of the basin water level in time, the reader is referred to appendix 15 with respect to the assumptions made.

In the approach followed the number of turbines remains fixed at 97, as was previously determined in chapter 6. The reason for this is that the number of turbines is based on the average head during the mean tidal cycle, where it was implicitly assumed that the average water level at the basin side will differ on average 0.25 m from that at the North Sea[65]. The analysis described in appendix 15 showed that this assumption turned out to be quite accurate.

First the reduction of the amplitude as a result of the presence of a barrier is determined for the preliminary design as depicted in figure 88. The amplitude ratio between the basin water level and the outside level is computed to be 0.64. In other words, as a consequence of the presence of a storm surge barrier, the amplitude of the basin water level is approximately 0.42 m lower than the amplitude of the outside water level. In figure 89 the blue line represents the amplitude variation of the outside water level, while the green line represents the amplitude variation of the water level within the basin. Obviously the average water level used in the determination of the installed power of the tidal power plant was too optimistic.

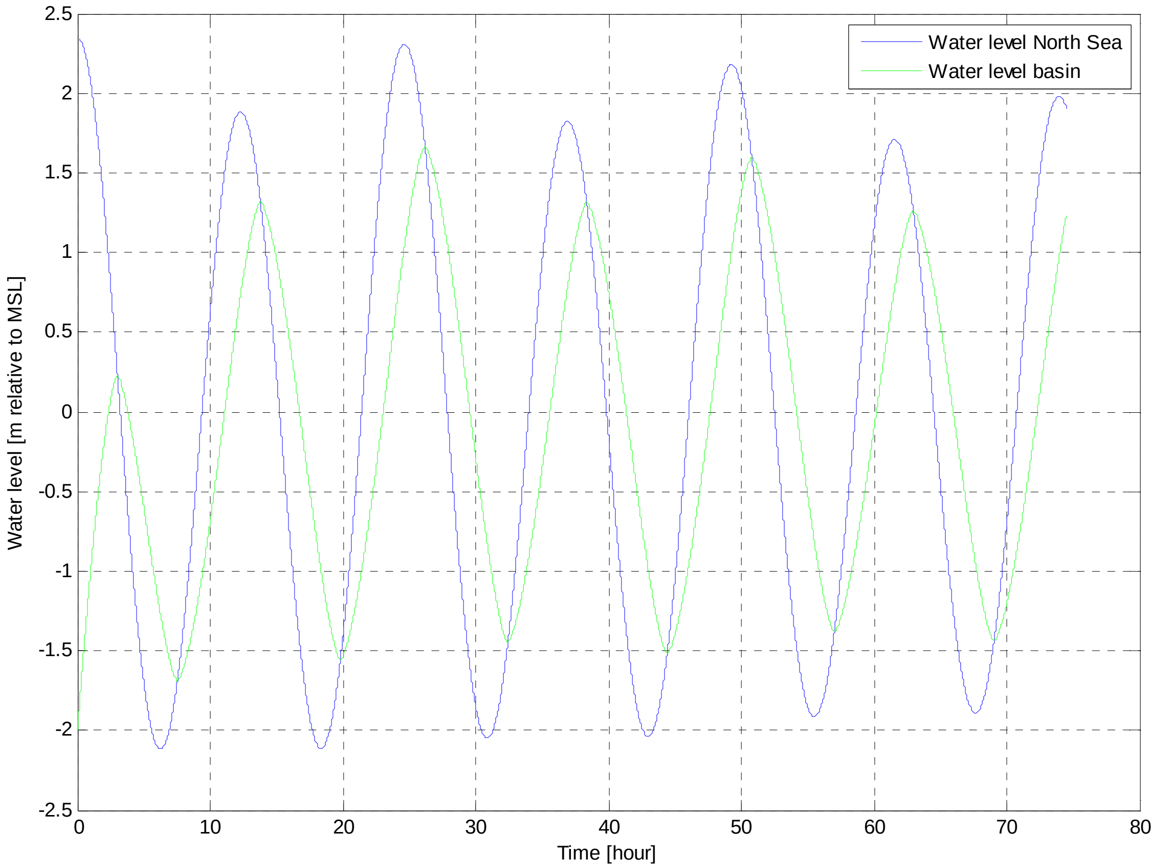


Figure 89: impact of the preliminary design on the amplitude of the water level within the basin.

Because the reduction of the amplitude at the inside is rather large, the current-carrying cross-section is raised in steps of 174 m^2, being the current-carrying cross-section of one sluice caisson, to determine an optimum value. The results are presented in figure 90.

The optimum current-carrying cross-section is found to be 18,622 m^2, corresponding to an additional 34 sluice caissons compared too the preliminary design. The computed amplitude ratio is 0.83. Hence the reduction of the amplitude of the outside water level as a result of the

[65] On average a difference of 0.25 m is assumed to be present during the filling of the basin.

presence of a storm surge barrier is 0.20 m. However placing an additional 34 sluice caissons requires an extra length of 2720 m, which is not available along the barrier line without large dredging works being required.

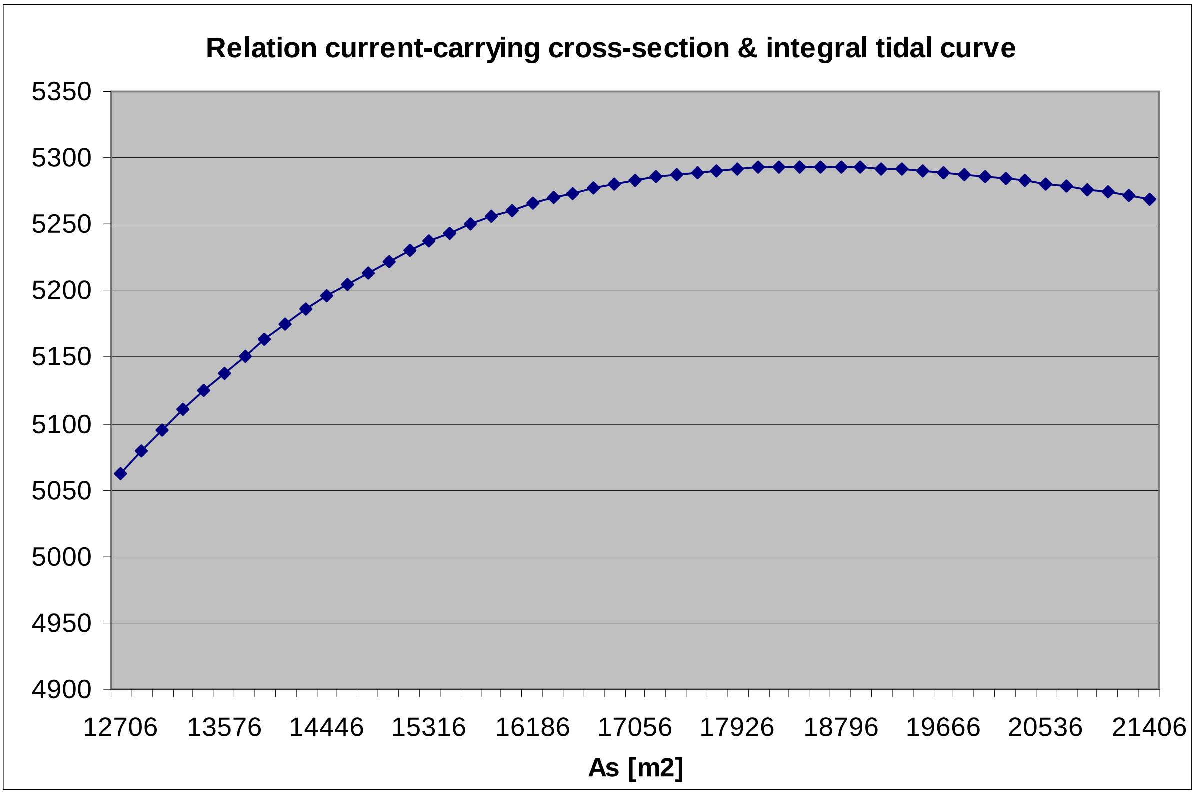


Figure 90: relation current-carrying cross-section and the area under the tidal curve.

Analysis learned that the available space along the barrier line allows for the placement of 30 additional sluice caissons, with a total length of 2400 m. The minimum distance between the sluices and both the commercial and recreational lock complexes is at least 140 metres at either side of the navigation locks, thus assuring that the flow conditions induced by the presence of the intake sluices does not interfere with navigation.

For this configuration the computed amplitude ratio is 0.81. Hence, the amplitude of the water level is reduced 0.22 m as a result of the presence of the storm surge barrier. This corresponds well to the 0.25 m head that was assumed during the preliminary design of the tidal power plant. Therefore the number of turbines in the final conceptual design remains the same as was established during the preliminary design. The variation in time of both the outside and inside water level is shown in figure 91 on the next page. The final conceptual design is presented in the next section.

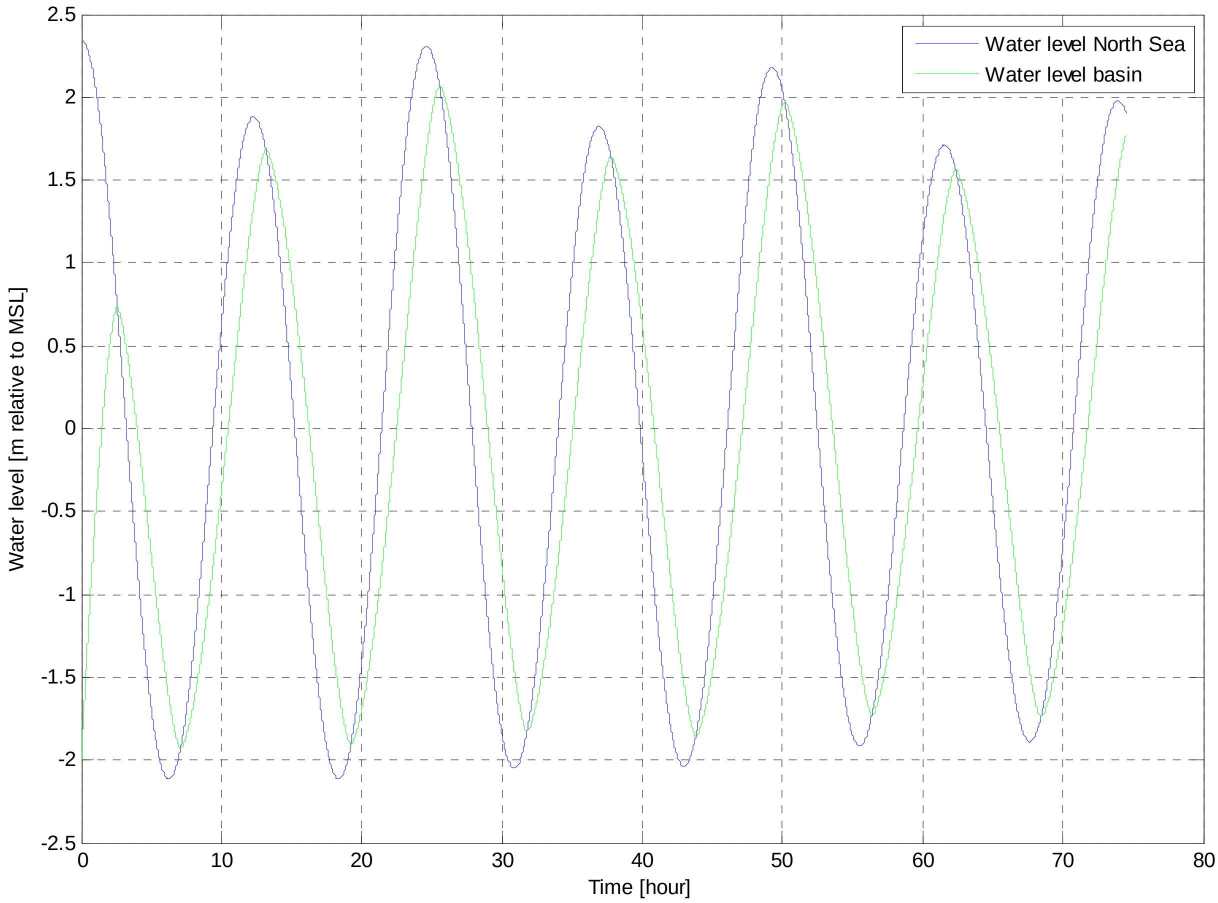


Figure 91: impact of the final conceptual design on the amplitude of the water level within the basin.

7.3 Final conceptual design

The final conceptual design is presented in figure 92 and consists of 97 turbines, 225 sluices, several interlinking caissons and both a commercial and a recreational lock complex. The total construction costs amount to £ 3824M. Although this is £ 193M higher than the construction costs of the preliminary design, the amount of annually generated energy is higher[66]. A bill of quantities of the final conceptual design is included in appendix 14.3.

Computing the break-even energy price (BEP) based on the total investment costs before taxes[67] and including the revenues from generating energy from the tide and reducing the flood risk of the hinterland bordering the Wash estuary, shows that the final conceptual design has a higher economic potential, see table 63.

	Preliminary design	Final conceptual design
BEP incl. flood risk	16.8 p/kWh	14.4 p/kWh
BEP excl. flood risk	21.1 p/kWh	17.8 p/kWh

Table 63: comparison break-even energy prices.

[66] Energy generated with the configuration of the preliminary design 2370 GWh per annum, energy generated with the configuration of the final conceptual design 2945 GWh per annum. The annually generated energy is estimated using equation 2.8 in section 2.1.3.
[67] The reader is referred to chapter 8 for more details regarding the computation of the break-even energy price.

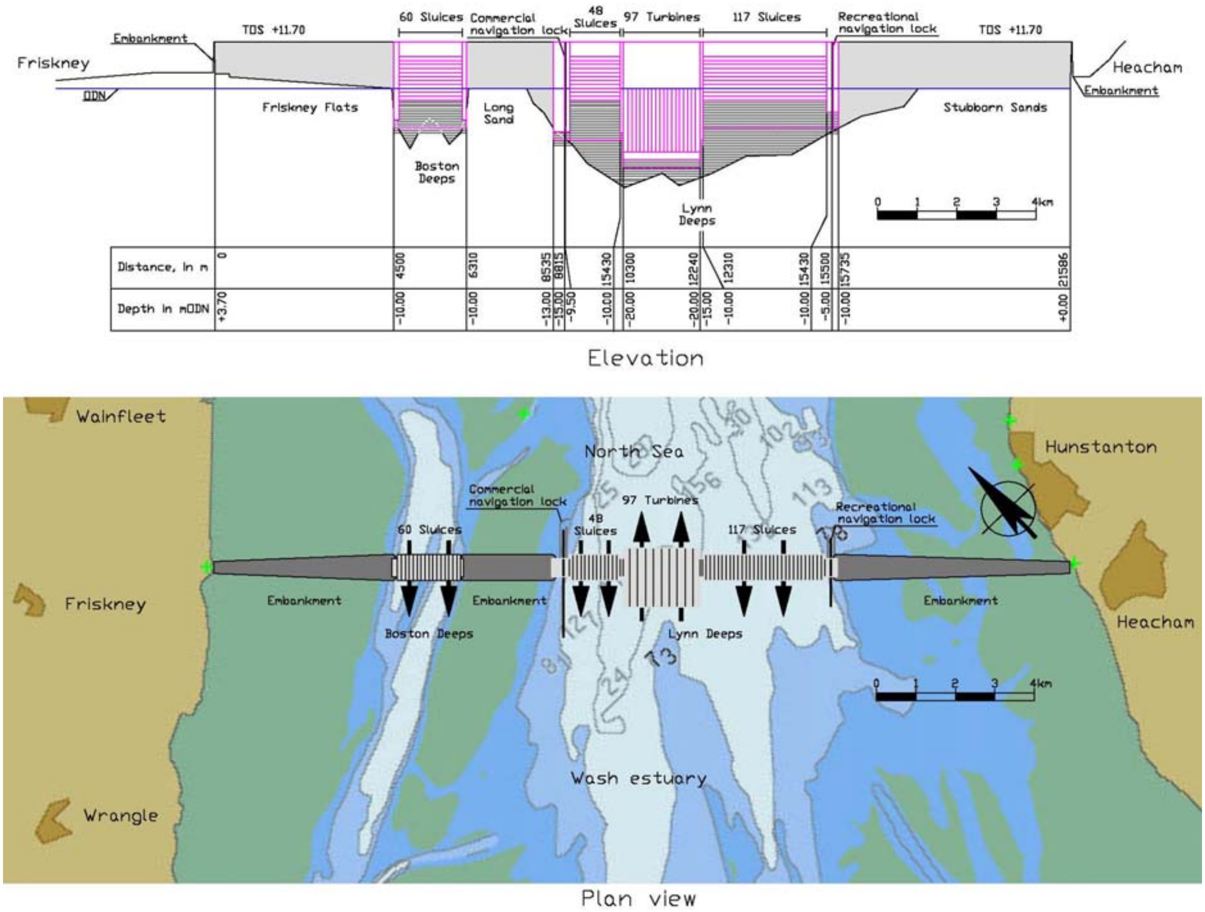
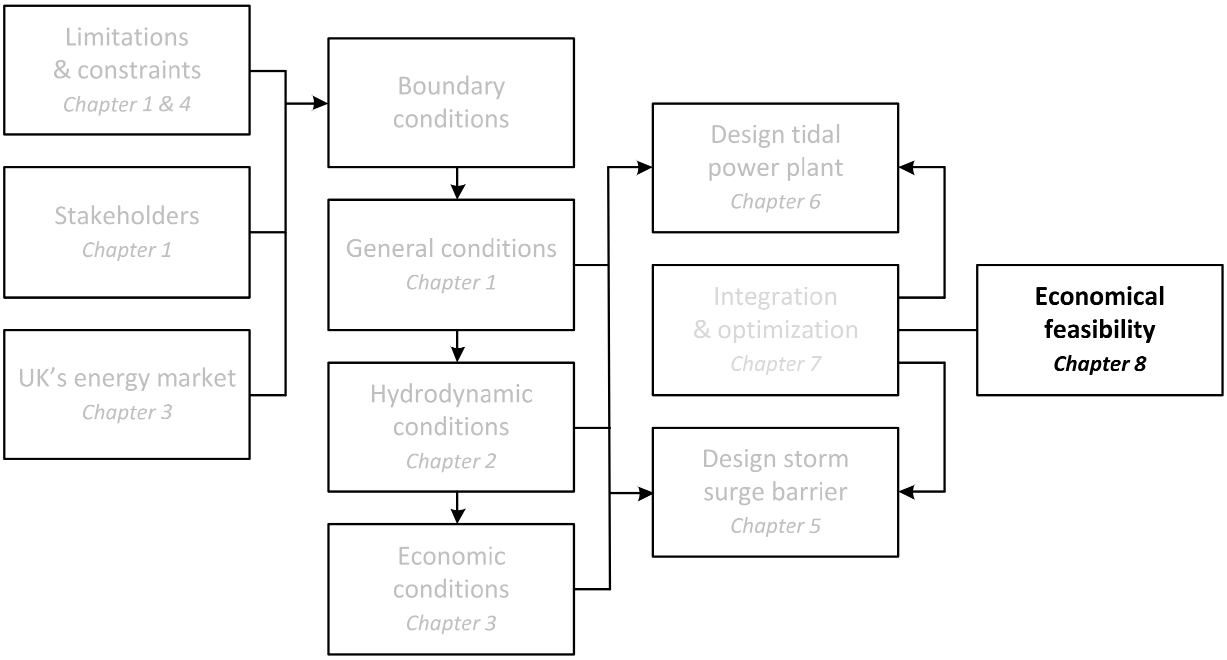


Figure 92: final conceptual design of the Wash estuary storm surge barrier and tidal power plant.

In appendices 16.1 and 16.2 the figures are included of the caissons used to cross the Boston and Lynn Deeps respectively.


Limitations & constraints
Chapter 1 & 4
Stakeholders
Chapter 1
UK's energy market
Chapter 3
Boundary conditions
General conditions
Chapter 1
Hydrodynamic conditions
Chapter 2
Economic conditions
Chapter 3
Design tidal power plant
Chapter 6
Integration & optimization
Chapter 7
Design storm surge barrier
Chapter 5
Economical feasibility
Chapter 8

8 ECONOMIC FEASIBILITY

In this chapter the economic feasibility of the construction of a combined storm surge barrier and a tidal power plant within the Wash estuary will be determined based on the comparison of the discounted values of all expenses and revenues. First the used assumptions and preconditions will be stated, after which the results of the performed economic analysis will be presented. Finally a conclusion regarding the economic feasibility of the project will be formulated.

8.1 Preconditions and assumptions

In table 64 an overview of the main preconditions regarding the performed economic analysis is given, for further details the reader is referred to appendix 17.

Parameter	Unit	Value
Design lifetime	[yr]	120
Construction time	[yr]	5
Cost range	[p/kWh]	8-11
Real interest rate	[%]	6
Annual energy yield	[GWh]	2945
Transmission losses	[%]	3
Maintenance & operation costs	[%] 1)	1
Costs civil work	[%] 2)	55
Costs electromechanical equipment	[%] 2)	45
Current SoP	[1/yr]	1:50/1:200
Future SoP	[1/yr]	1:500
Flood prone area, of which:	[ha]	353,900
rural area	[ha]	*348,600*
minor city	[ha]	*2,100*
major city	[ha]	*3,200*

1) Percentage of the total construction costs (direct costs only).
2) Percentage of the total investment costs.

Table 64: parameters used in the economic analysis.

With respect to the economic appraisal of the project the following assumptions were made:

- costs of the transmission lines are not included in the performed appraisal, see section 3.4;
- no residual value of the structure at the end of the design lifetime is taken into account;
- V.A.T. (20%[68]) is not included in the present value computations.

[68] Source: http://www.hmrc.gov.uk/vat/forms-rates/rates/rate-increase.htm

8.2 Economic appraisal

The expenses of the project consist of the total investment costs, the costs of the required refurbishments and the maintenance and operation costs during the design lifetime of 120 years. The revenues resulting from the generation of energy and enhancing the current flood protection level start in the 6th year. In this section only the main conclusions of the performed economic appraisal are presented, for more detailed information the reader is referred to appendix 17.

8.2.1 Discounted value expenses

In order to be able to acquire a rough estimation of the construction costs for a tidal power plant in the Wash estuary, two methods will be used. The first method consists of the same approach as was followed before with respect to the storm surge barrier. While the second method is based on a bill of quantities (see chapter 7).

Method 1:

Based on the data represented in table 65 characteristic values are determined for each of the large tidal power plants, using equation 8.1. The characteristic values found should be in the order of 3300 £/kW[69].

$$C_{new} = I_{ref} \cdot P_{new} \qquad [8.1]$$

Were:

C_{new}	:	cost new tidal power plant design	[£]
I_{ref}	:	characteristic values reference design	[£/kW]
P_{new}	:	installed power new tidal power plant	[kW]

TPP	Cost 2012 [10^6 £]	P_{inst} [MW]	Turbines [pieces]	R_{mean} [m]	L [10^3 m]	D [m]	I_{ref} [£/kW]
France							
La Rance	477	240	24	8	0.75	5.35	1988
United Kingdom							
Severn	25,200	8640	214	14	16	9.0	2917
Solway Firth	18,436	5580	200	5.6	30	9.0	3304
Mersey	3,200	700	28	6.5	1.9	8.0	4571
South Korea							
Sihwa	248	250	10	5.6	12.7	7.5	992
Garolim	766	480	20	4.7	2	8.0	1596
Inchon	1,772	1000	44	5.3	20	7.5	1772

Table 65: characteristic values for several reference tidal power plants.

Although the characteristic values of the Severn and Solway Firth TPP are in the order of 3300 £/kW, it should be concluded from table 65 that site specific characteristics seem to have a large influence on the construction costs. With respect to the TPP's in South Korea the main factor contributing to the deviation of the expected characteristic value will be the labour costs, which are much lower than in Europe. The same will more or less hold for the La

[69] Source: L.F. Mooyaart. The value mentioned corresponds to the characteristic value used in The Netherlands for the appraisal of run-off river plants.

Rance TPP as the labour costs in the 1960's were much lower than the present day labour costs. In case of the Mersey TPP the difference is most likely the consequence of the larger ratio between the length of the tidal power plant section over the total barrier length.

Using a characteristic value of 3300£/kW, the total investment costs, before taxes and without the project contingency, for the Wash TPP will be approximately £ $3.10 \cdot 10^9$, corresponding with total construction costs of £ $2.20 \cdot 10^9$. The break-even energy price, including revenues from the reduction of the flood risk, in this case is computed to be 6.35 p/kWh, which is even lower than the 2010 UK energy price (without taxes) for coal fired power plants (7.58 p/kWh[70]). Without the revenues from increasing the SoP the break-even energy price is computed to be 9.80 p/kWh. Both figures seem highly unlikely, especially because the 2010 Parsons and Brinkerhoff study concluded that for the Severn TPP, which is deemed to be the most feasible UK TPP, the cost range is 15.50-39.00 p/kWh.

In chapter 5 it has been established that the Wash TPP barrier length is 21 km. Hence, both the Severn and Solway Firth TPP's are dimensionally similar to the Wash TPP. On the other hand, for the Wash TPP 97 turbines are required, which is in the order of half the required number of turbines for the Severn and Solway Firth TPP's. Therefore the total investment costs should be in the order of $6 \cdot 10^9$ £ to $15 \cdot 10^9$ £ [71]. It can be concluded that method 1 is not appropriate for estimating the construction costs.

<u>*Method 2:*</u>
In chapter 7 the construction costs were estimated based on a bill of quantities, resulting in the total construction costs being £ 3,824M. The total investment costs, before taxes and without the project contingency, result in £ $6.88 \cdot 10^9$ (see appendix 17.1). The results of this method are within the range that was to be expected based on the cost estimations made for the Severn and Solway Firth tidal power plants.

The characteristic value is computed to be 7300 £/kW. The rather high characteristic value may explained by the fact that tidal power plants are deemed feasible in case the mean tidal range is equal to or larger than 5 metres [Clarke, 2007]. Since the mean tidal range in the Wash estuary is only 4.70 m it is to be expected that the characteristic value will be higher than 3300 £/kW. Furthermore a long barrier is required to be able to generate energy from the tidal motion. Although the characteristic value of the Wash barrier and tidal power plant is almost twice that of the value for run-off river plants in the Netherlands, method 2 will be used as an indication of the construction costs.

The discounted value of the expenses amounts to £ 6704M.

[70] Source: http//www.iea.org/stats/index.asp. International Energy Agency publication on Energy Prices and Taxes, 2010.
[71] Using the Severn TPP as reference, the estimated costs of the Wash TPP become:
$25200 \cdot 10^6 \cdot (21 \cdot 97) / (16 \cdot 214) = 15 \cdot 10^9$ £.
In case of the Solway Firth TPP, the estimated costs are: $18436 \cdot 10^6 \cdot (21 \cdot 97) / (30 \cdot 200) = 6 \cdot 10^9$ £.

8.2.2 Discounted value of the revenues from energy

In the 6th year the tidal power plant is able to generate energy, using the head difference over the Wash barrier. The discounted value of the revenues is computed using different energy prices, all within the cost range as was established earlier in chapter 3. See appendix 17 for further details on the performed computation.

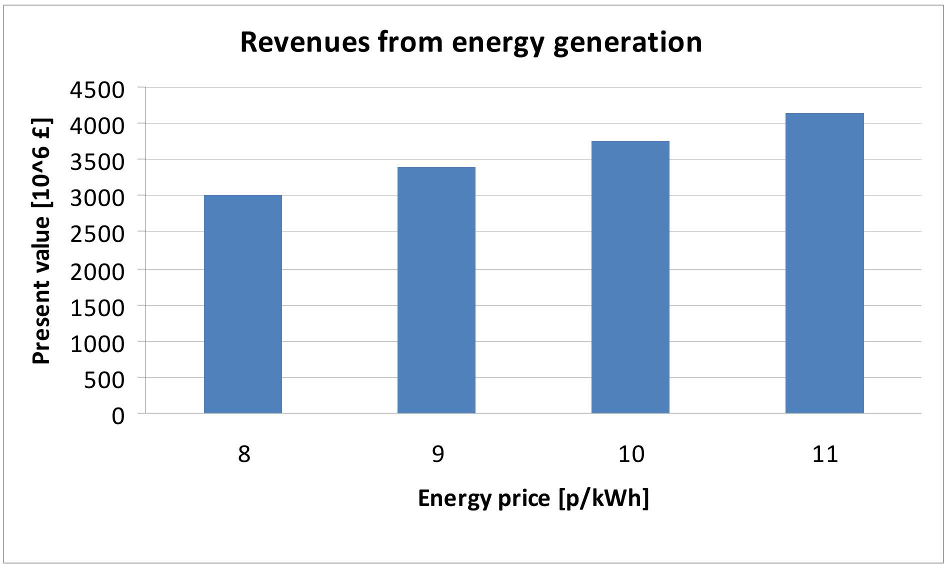


Figure 93: discounted value revenues from energy generation.

8.2.3 Discounted value of the revenues from enhanced flood protection

When the UK Government participates in the project, raising the Standard of Protection from 1:200 too 1:500 will result in revenues as a consequence of the reduced flood risk for the hinterland bordering the Wash estuary. In case the project is financed by a private investor only, this added value does not generate additional revenues and therefore does not contribute to the profitability of the project.

The economic value of the expected damages and losses is determined using the *estimated annual average damages* figures as drawn up by Halcrow in the National Appraisal of Assets at Risk from Flooding and Coastal erosion, that was commissioned by the Department for Environment, Food and Rural Affairs in 2001 [DEFRA, 2001]. The estimated annual average damages are defined as the flood risk (probability of failure *times* damages), see table 66 for the values of the damages and losses per hectare that are applied in the present value computations (see appendix 17 for further details).

Category	Mean value AAD [£/ha]	Damages [£/ha]
Rural area	250	50,000
Minor city	700	140,000
Major city	3000	600,000

Table 66: applied value of the annual damages.

Table 67 presents the computed annual flood risk for both present day and future situations. From the table can be concluded that the present value of the revenues from enhancing the level of flood protection amounts to £ 1293M.

	Failure probability [1/yr]	Total value of damages [10^6 £]	Annual flood risk [10^6 £/yr]
Present day	1:50 / 1:200	19,644	137.38
Future	1:500	19,644	39.29

Table 67: discounted present day and future flood risk.

8.2.4 Net Present Value

The Net Present Value (NPV) represents the current value of an investment by means of comparing the discounted cash flows of expenses and revenues, see equation 8.2. As long as the NPV is larger than or equal too zero an investment is considered to be feasible.

$$NPV = \left(\sum_{n=1}^{N} \frac{C_E}{(1+r)^n} + \sum_{n=1}^{N} \frac{(P_{pd} \cdot D) - (P_{fu} \cdot D)}{(1+r)^n} \right) - \left(I_0 + \sum_{n=1}^{N} \frac{C_{ex}}{(1+r)^n} \right) \qquad [8.2]$$

Were:

NPV	:	net present value of the investment	[£]
C_E	:	Monetary value of the generated energy in year n	[£]
P_{pd}	:	present day failure probability	[1/yr]
P_{fu}	:	future failure probability	[1/yr]
D	:	total damage	[£]
I_0	:	initial investment costs	[£]
C_{ex}	:	monetary value of the expenses in year n	[£]
r	:	real interest rate	[-]
n	:	number of years from investment year (n = 0)	[-]
N	:	design life time storm surge barrier, 120 yr	[-]

The NPV's corresponding to several energy prices are presented in table 68. From the table can be concluded that the investment is economically not feasible, based on the present day preconditions and the assumptions made in the performed analysis.

Energy price [p/kWh]	Net Present Value public-private cooperation [10^6 £]	Net Present Value private cooperation [10^6 £]
8.00	-2398	-3691
9.00	-2021	-3314
10.00	-1644	-2938
11.00	-1268	-2561

Table 68: NPV corresponding to several energy prices.

8.2.5 Break-Even Point

The Break-Even Point (BEP) corresponds to NPV of zero, the energy price corresponding to the break-even point can be computed as follows:

$$BEP = \frac{PV_{ex} - PV_{SoP}}{PV_{rev,E,1p.kWh}} \qquad [8.3]$$

Were:

BEP	:	energy price corresponding to the break-even point	[£/kWh]
PV_{ex}	:	summation of the discounted values of the expenses	[£]
$PV_{rev,SoP}$	:	summation of the discounted values of the revenues from raising the SoP	[£]
$PV_{rev,E,\ 1\ p/kWh}$	:	summation of the discounted values of the energy revenues for a energy price of 1p/kWh	[£]

The energy price at the BEP amounts 14.4 p/kWh in case the revenues from enhanced flood protection are included, and 17.8 p/kWh when they are excluded.

8.3 Conclusion

Since the Wash barrier project is a private initiative revenues have to be generated in order for the project to be profitable, this is where a tidal power plant entered the picture. Hence in order for the project to be economical feasible the revenues generated via the extraction of energy from the tide must at least cover the total expenses of construction, operation and maintenance during the structure's economical lifetime and preferably be higher. The Parson's and Brinkerhoff study, see chapter 3, showed that in order to be competitive with other low carbon energy schemes the energy price must lie within a range of 8-11 p/kWh. Figure 94 clearly indicates that the BEP lies at approximately 18 p/kWh. Therefore a fully privately funded Wash barrier is not considered to be financially feasible. The project may compete with offshore wind farms, but compared to an offshore wind farm the environmental impact of a barrier is disproportionally large.

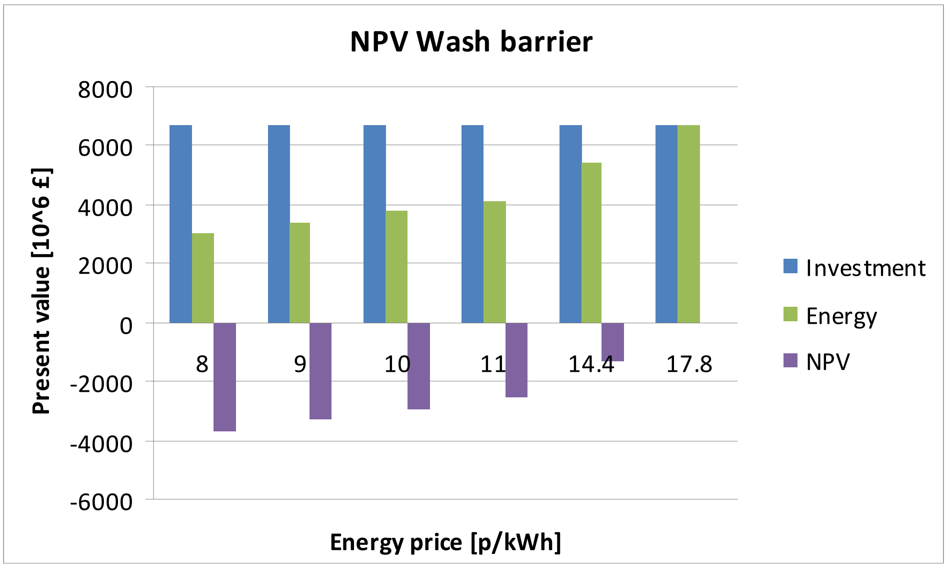


Figure 94: net present value Wash barrier excluding revenues from flood risk reduction.

In case the UK Government decides to participate in the project, the revenues from enhancing the flood protection level have to be included. Figure 95 shows that even with these revenues included, the energy price at the BEP is 14.4 p/kWh. Nevertheless a reduction of 3.4 p/kWh, the BEP energy price is still too high to compete with other low carbon energy generation technologies. Although the competition with offshore wind farms is improved significantly, the project is still not considered economical feasible.

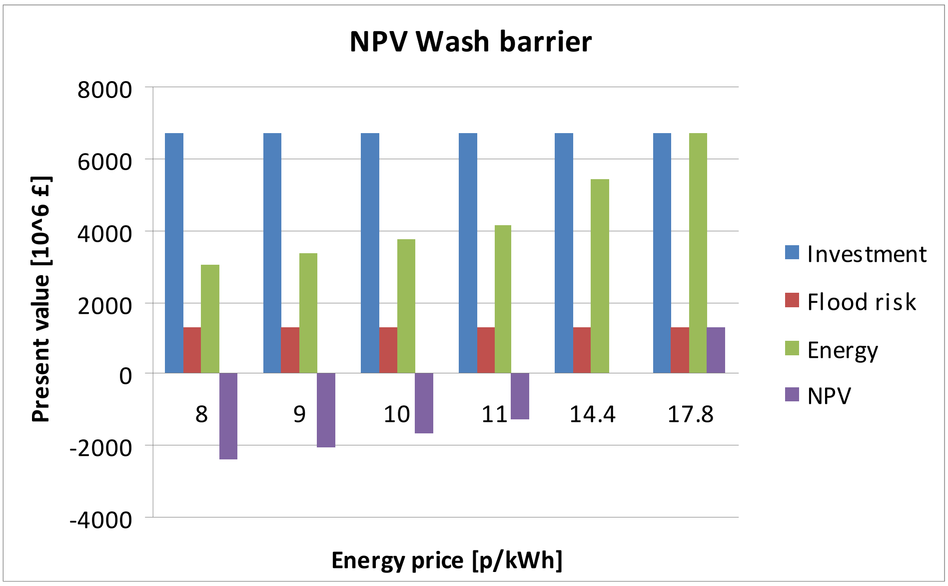


Figure 95: discounted value revenues from energy generation.

In the performed analysis possible financial incentives from both the UK Government and European Committee, such as, carbon pricing, the buy-out price and Feed-in-Tariffs are not included in the analysis. The reason for this is the fact that the revenues largely depend on market operation, which is a complex and continuously changing system. The mapping and quantifying all the influences falls outside the scope of this thesis. It is however recommended to include these effects in a future study.

Combining the structure with other functions, such as the levying of tolls on a possible future road connection[72] from Hunstanton to Skegness crossing the estuary on top of the structure, renting out space for utilities, placing wind turbines on the structure, etc., may result in additional revenues. Thus lowering the energy price at the break-even point. However within the framework of this thesis this possibility was not studied.

[72] The present day situation does not result in any demand for direct a road or rail connection between Skegness and Hunstanton, see the Terms of Reference in chapter 4.

9 CONCLUSIONS AND RECOMMENDATIONS

Over the last century several flood disasters and near flood disasters have occurred in the Fenlands bordering the Wash estuary. The purpose of this thesis is to establish whether it is possible and attractive to combine the closure of the estuary with the construction of a tidal power plant. Therefore the technical and economical feasibility of such a combined structure is assessed. In the remainder of this chapter first the main conclusions are presented, after which the recommendations for further research are stated.

9.1 Conclusions

Conclusions regarding the technical feasibility of the tidal power plant are:

1. Technically it is possible to construct a tidal power plant within the Wash estuary. The tidal power plant consists of 225 sluices and 97 turbines, has a installed power of 940 MW and is estimated to have a annual energy yield of 2945 GWh. The turbine diameter is 8.00 m.
2. Based on a comparison of costs, operational flexibility, fish friendliness and hands-on experience the bulb turbine is selected over the straight flow turbine.
3. With respect to the gates of the sluices, tainter gates are preferred over roller type vertical lift gates, mainly because of the higher operational reliability and lower impact on the seascape.
4. Although a single basin two-way generation scheme is predicted to generate more energy and does effect the natural tide the least, and therefore has the smallest impact on the fragile ecology within the basin, a single basin ebb generation scheme proved to be more cost effective and was therefore selected.

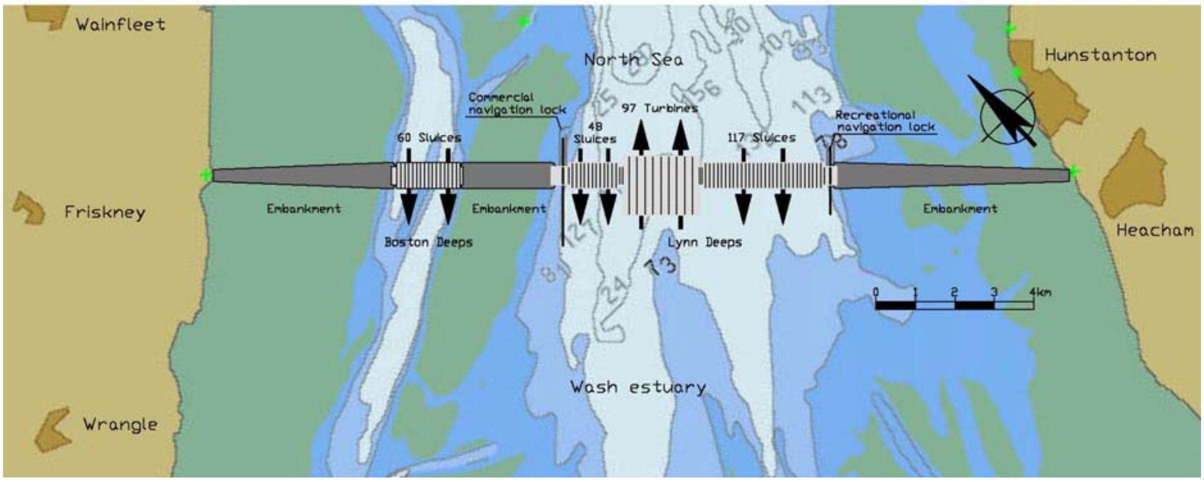


Figure 96: bird-eye view of the Wash estuary storm surge barrier and tidal power plant.

Conclusions regarding the technical feasibility of the storm surge barrier are:

1. Constructing a storm surge barrier across the mouth of the Wash estuary is technically possible. However raising the existing embankments results in the same flood protection level for less costs. The main reasons for this are the facts that the Fenlands are not densely populated, the main land use in the hinterland is agricultural, the presence of vast salt marshes and intertidal flats in front of the embankments dissipate most incoming wave energy. As a result

of which the embankments are relatively low compared to the crest height required by the storm surge barrier.

2. The combined costs of permanent barrier and the tidal power plant are much lower than the costs of constructing an Eastern Scheldt type barrier. Therefore a combination of a permanent and movable barrier is not suited as a storm surge barrier in the Wash estuary.
3. On the shallow parts of the estuary an embankment dam is projected, while in the Boston and Lynn Deeps caissons will be used. The layout of the storm surge barrier is presented in figure 96.
4. As a consequence of the expected large flow velocities in the final closure gap, due to the large tidal prism, a caisson closure using sluiced caissons is opted for, crossing the Boston and Lynn Deeps. A stone closure is ruled out because of the large amount of material required and due to the larger footprint of the structure, implying a larger environmental impact.
5. The barrier line is located at some distance of the mouth of the estuary due to the presence of a deep underwater canyon located near the mouth of the estuary.
6. The design foresees in two navigation lock complexes, near the western border of the Lynn Deeps a commercial navigation lock with a capacity of one design vessel is projected. While at the eastern border a recreational navigation lock[73] with a capacity of six design ships is planned.

Conclusions regarding the economical feasibility of the combined structure:

1. In order to be competitive with other low carbon energy sources the cost of the electricity generated by a tidal power plant in the Wash estuary should lie within a price range of 8-11 p/kWh. This range is derived from the results of a study by Parsons and Brinkerhoff in 2010.
2. The total investment costs before taxes amount to £ 6877M.
3. For the economic appraisal a break-even energy price is determined for the Wash barrier project, based on the Net Present Value of the investment[74]. When the project is funded by an private investment group, the revenues from raising the Standard of Protection may not be included as these revenues do not contribute to the profitability of the project. The break-even energy price is computed to be 18 p/kWh. When the UK Government decides to participate in the project the break-even energy price is computed to be 14 p/kWh, as now the discounted value of the reduced damages and losses may be included in the Net Present Value. As these prices are both larger than 11 p/kWh the project is considered to be not economical feasible.
4. In case the UK Government participates in the project the present value of the revenues resulting from enhancing the flood protection level makes up for approximately 24% of the total present value regarding the revenues.

[73] The Wash Tidal Barrier Corporation plc has prescribed the construction of a recreational lock, see chapter 4. However the commercial navigation lock is characterized by a very low usage (2-3 vessels a day), therefore it seems more logical to abandon the idea of constructing a separate recreational navigation lock and combine all traffic in the commercial navigation lock.
[74] The Net Present Value represents the current value of an investment by means of comparing the discounted cash flows of expenses and revenues. As long as the Net Present Value is larger than zero an investment is considered to be feasible.

9.2 Recommendations

During the process of assessing the technical and economical feasibility of the proposed storm surge barrier and tidal power plant, a number of assumptions had to be made with respect to environmental, technical and economical aspects. Some due to the lack of data, others as a result of the restricted available time frame. In order to improve and complete the feasibility study the following recommendations are stated:

1. The impact of the storm surge barrier on the morphology within the basin and along the adjacent Norfolk and Lincolnshire coastlines should be assessed. As is the effect on the Standard of Protection along these adjacent coastlines, this may be done by developing a 3D flow model.
2. An environmental impact study is required to study the consequences of the project and determine what mitigating measures are required. Also a solution for enabling the Common Seals to migrate from their hunting grounds in the North Sea to their breeding and haul out area within the Wash estuary has to be determined. This may lead to fundamental changes in the design.
3. With respect to designing the foundation of the structure, which contributes considerably to the construction costs of the barrier, a geotechnical survey is recommended to gain insight into the composition of the sea bed at the barrier line.
4. In order to improve the design criteria, measurement of river discharges; water levels; wave heights and basin geometry at several locations within the estuary is recommended in order to be able to develop a hydraulic model. This model may be used to determine the relation between water level and wetted area, to assess the effects of the barrier on the water levels within the basin and to check the design criteria.
5. The detailed design of the electromechanical equipment is recommended.
6. The structural design of the different types of caissons, turbines, gates and navigation locks is recommended.
7. It is recommended to review the capacity of the recreational navigation lock complex, as no exact figures on the amount of vessels entering and leaving the estuary where available.
8. In the performed analysis possible financial incentives from both the UK Government and European Committee, such as, carbon pricing, the buy-out price and Feed-in-Tariffs are not included. The reason for this is the fact that the revenues largely depend on market operation, which is a complex and continuously changing system. The mapping and quantifying of all the influences is recommended, as to include these effects in a future study.

GLOSSARY

Anglian glacial period	:	name used on the British Isles for the 2nd major glacial period that in Northern Europe is called Elsterian.
Aphelion	:	point in the earth's orbit around the sun were it is furthest from the sun.
Apogee	:	point in the moon's orbit around the earth were it is furthest from the earth.
Break Even Point	:	value for which the coverage contribution per period equals the fixed costs over that period.
Buy-out price	:	penalty that has to be paid by an electricity supplier that has not acquired enough Renewables Obligation Certificates proportionate to the amount of electricity sold.
Car dyke	:	Roman build catchwater drain. This early form of flood protection is approximately 122 km long, running along the edge of the Fenlands from Waterbeach to Lincoln.
Carstone	:	type of sandstone that is orange when weathered and greenish-brown otherwise.
Declination	:	the angle between the equatorial plane and both the earth-moon and sun-earth lines.
DECC	:	Department of Energy and Climate Change.
DEFRA	:	Department of Environment, Food and Rural Affairs.
Devenian glacial period	:	name used on the British Isles for the 4th major glacial period that in Northern Europe is called Weichselian.
Discounted value	:	see Present Value.
Diurnal tide	:	tidal wave with a periodicity that is close to the duration of a solar tide.
Ebb tide	:	falling tide, the period between high water and the succeeding low water.
Ebb dominance	:	maximum ebb velocities are higher than the maximum flood velocities.
Equator	:	the intersection of earth's surface with the equatorial plane.
Equatorial plane	:	a plane perpendicular through the axis of rotation of the earth that also contains earth's centre of mass.
Feed-in-Tariffs	:	fixed prices per kWh, to be paid by electricity suppliers to the owners of a low-carbon electricity plant (<= 5 MW) that export energy to the electricity network. The prices are nor linked to the wholesale market prices.
Fen	:	local name for an individual area of marshland or former marshland.
Fen edges	:	are upland surrounding the Fenlands.
Fen islands	:	areas of higher land which were never covered by the growing peat and remained dry when the surrounding Fenlands were flooded.
Fenlands	:	low lying former marshlands surrounding the Wash estuary.
Fetch	:	the distance to the upwind coastline.
Flood dominance	:	maximum flood velocities are higher than the maximum ebb velocities.

Flood tide	:	rising tide, the period between low water and the succeeding high water.
Foss dyke	:	Roman build catchwater drain that runs from Lincoln to Torksey on the river Trent.
Haven, The	:	tidal arm of the Wash estuary, near the town of Boston.
High-carbon energy sources	:	energy sources that emit much carbon dioxide into the atmosphere, such as traditional coal and gas fired power plants.
High water	:	the highest level reached by the water during the tidal cycle.
High-water slack	:	flow reversal from flood to ebb.
Ipswichian glacial period	:	name used on the British Isles for the 3rd interglacial period that in Northern Europe is called Eemian.
Low-carbon energy sources	:	energy sources that do not emit much carbon dioxide into the atmosphere, such as nuclear energy, fossil fuels with carbon capture and storage *and* renewable energy sources.
Low water	:	the lowest level reached by the water during the tidal cycle.
Low-water slack	:	flow reversal from ebb to flood.
mODN	:	chart datum, water level with respect to Mean Sea Level in Newlyn.
Mud stone	:	a fine grained sedimentary rock whose original constituents were clay and mud.
Net Present Value	:	difference between discounted value of the revenues minus the discounted value of the expenses.
Ofgem	:	Office of Gas and Electricity Markets.
Perigee	:	point in the moon's orbit around the earth were it is nearest to the earth.
Perihelion	:	point in the earth's orbit around the sun were it is nearest to the sun.
Present Value	:	today's amount of money that over a period of n years with a real interest rate r exactly results in the amount of money desired at that future date.
Rapeseed	:	in Dutch: *koolzaad*.
Real interest rate	:	interest rate minus inflation rate.
Refurbishment	:	an investment made to repair or improve existing equipment or civil works, with the purpose to restore the unit to or above its original state.
Relative sea level rise	:	the sea level rise related to the level of the continental crust. Changes can be caused by absolute changes of the sea level and/or by absolute movements of the continental crust
Return period	:	
ROC	:	Renewables Obligation Certificates, certificates that are issued for each MWh of eligible renewable electricity produced.
Salt marsh	:	low coastal grassland frequently overflowed by the tide.
Saw-tooth asymmetry	:	vertical asymmetry of the horizontal tide, resulting in a pitched forward wave profile. The high-water slack duration is longer than the low-water slack duration or vice versa.

Seahenge	:	a timber circle with an upturned three root in the centre. It was apparently build in the 21st century BC, during the early Bronze age. Most likely for ritual purposed, the site is located on the inter tidal flats near the village of Holme-next-the-Sea.
Semidiurnal tide	:	tidal wave with a periodicity that is close to half of the duration of a solar day.
Shingle	:	very coarse gravel.
Skewness	:	deformation of the horizontal tide, as a result the velocity signal is asymmetric around the vertical (shorter duration of positive water levels than negative water levels or vice versa).
Spit	:	or sandspit is a deposition landform found off coasts. At one end the spit extends into the sea, while the other end is connected to land. It is a type of bar that grows into the direction of the littoral drift (longshore drift).
Swale	:	a low tract of land that is moist or marshy. In Dutch: *duinvallei*.
Swell	:	regular and long-crested waves, generated in a distant storm. The waves are the result of frequency and direction dispersion.
Tidal prism	:	1) the volume of water exchanged in a basin between mean high tide and mean low tide; 2) the volume of water entering or leaving the basin per half tidal cycle.
Tidal stream	:	periodic horizontal flow during ebb tide and flood tide.
Tidal window	:	time frame in which it is possible for vessels to enter a harbour.
Tide	:	periodic vertical rise and fall of the water level.
Tide locking	:	time during which it is not possible for the river discharge to freely flow into the estuary because the outside water level is to high.
Townlands	:	an arch like broad bank of silt around the Wash estuary and form the remains of the river embankments that formed naturally during the Bronze and Iron ages.
V.A.T.	:	Value Added Tax, currently 20% in the United Kingdom according to the Her Majesty Revenue and Customs.
Wind sea	:	the initially random wave field as generated in a storm. The waves are short-crested and have an irregular schape.
Woltonian glacial period	:	name used on the British Isles for the 3rd major glacial period that in Northern Europe is called Saalien.

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