CF54 Final Report

Development west coast Taiwan

Redesign coastal area between Da'an River and Dajia River

Appendices

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Appendix A Natural boundary conditions

A.1 Tidal information

From Taichug Harbor bureau data[1] follows, the reference level is LWS:

- Mean water level 2.63 m
- Mean tidal range 3.63 m
- Mean High Water level MHW 4.45 m
- Mean Low Water level MLW 0.82 m
- Highest high water level 5.86 m (23 September 1971)
- Lowest low water level -0.55 m (21 January 1988)

These tides have been recorded at Taichung Harbor, which is located approximately five kilometers south of the Dajia River mouth.

A.2 Rainfall data

From central weather bureau data[2] follows the monthly mean precipitation for Taichung and Wuchi, this is in the tables below.

Table A.1 Precipitation in Taichung and Wuchi in mm

Station	Jan	Feb	Mar	Apr	May	Jun
Taichung	36.3	87.8	94	134.5	225.3	342.7
Wuchi	28.5	84.5	106.1	131	222.5	217.7

Station	Jul	Aug	Sep	Oct	Nov	Dec	Total	Duration
Taichung	245.8	317.1	98.1	16.2	18.6	25.7	1642	1971-2000
Wuchi	165.9	213.2	68.7	9.9	14.9	20.1	1283	1976-2000

A.3 Extreme rainfall

The following figure gives rates of recurrence for the maximum single day and two-day rainfall. The continuous line depicts the return-rate for the maximum one-day precipitation. The value of that precipitation is depicted on the vertical axis. The dashed line gives the return-rate for two-day maxima.

[1] http://www.tchb.gov.tw/ENG/e430.php?targE=tr3
[2] http://www.cwb.gov.tw/V4e/index.htm

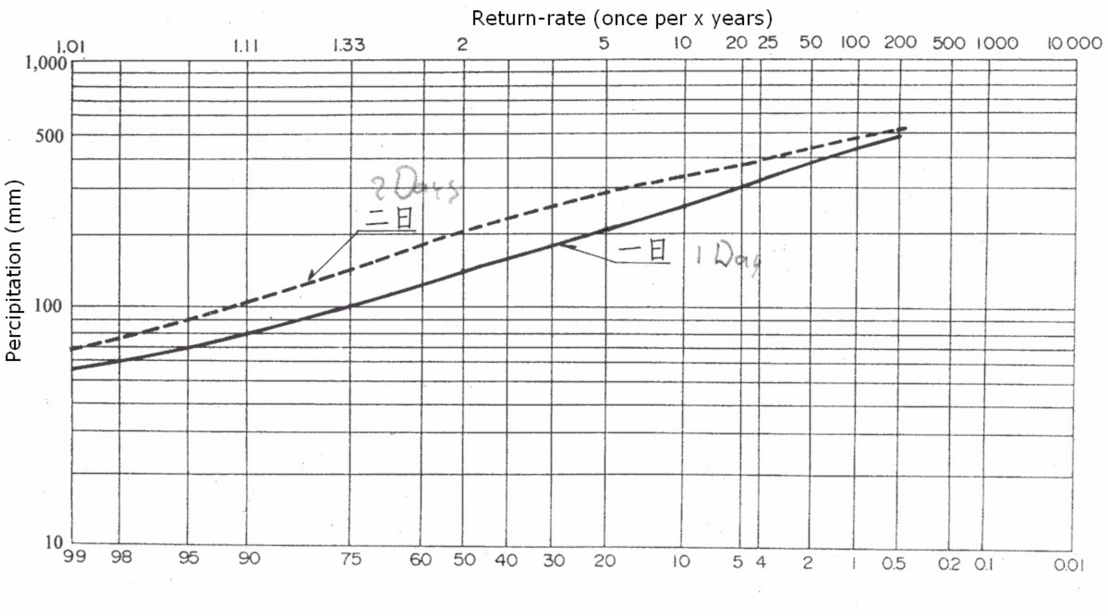

Figure A.1 Return-rate maximum one- and two-day precipitation

The following figure shows the development of an average rainfall-event. This figure shows the averaged distribution of six representative rainfall-events.

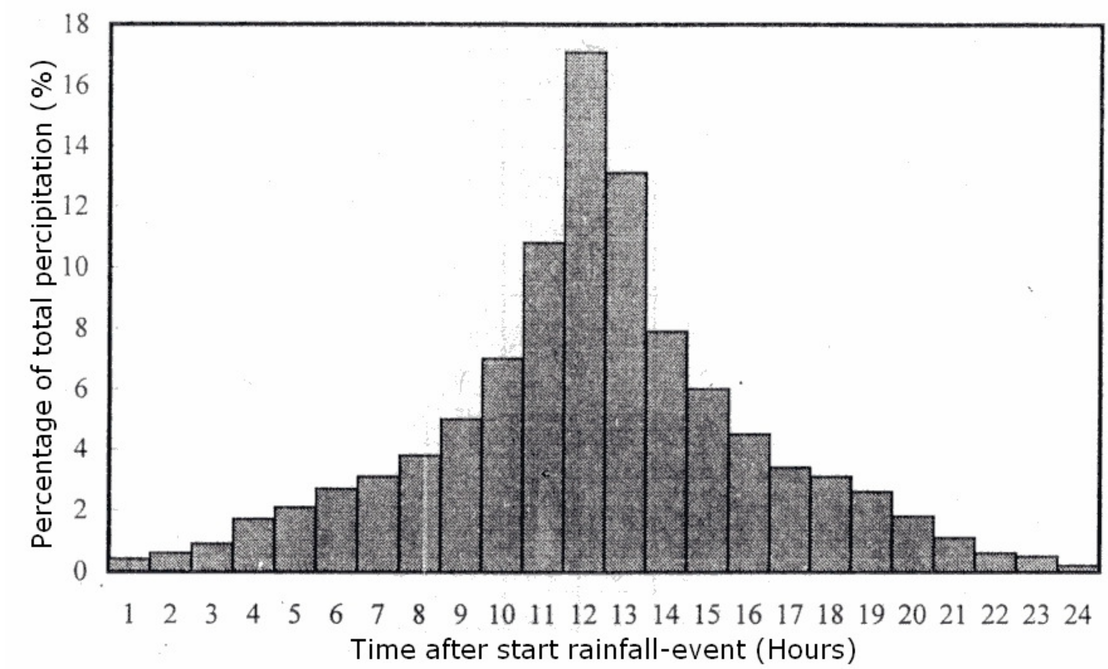

Figure A.2 Distribution within rainfall-event

A.4 Wave climate

From the study *"Taichung Harbor dike closure for South landfill area"* follows data about the return-rate of waves; unfortunately the wave-height with a return rate of 100 years is given. With logarithmic calculations the wave-height with a return period of 250 years is determined. In table A.2 the wave-heights according to their direction and return-period are given with the associated wave-period. In figure A.3 the wave-heights are plotted against the return-rate on a logarithmic scale.

Table A.2 Return period of waves

Return period	5 year		10 year		25 year		50 year	
Direction	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)
N	2.87	7.60	3.11	7.91	3.35	8.22	3.57	8.49
NNW	4.77	9.80	5.18	10.22	5.60	10.63	5.98	10.98
NW	5.19	10.23	5.58	10.61	6.07	11.06	6.43	11.39
WNW	4.66	9.69	5.08	10.12	5.51	10.54	5.89	10.90
WNW	4.24	9.24	4.68	9.72	5.04	10.08	5.27	10.31
WSW	4.45	9.47	4.88	9.92	5.32	10.36	5.63	10.65
SW	4.87	9.91	5.28	10.32	5.70	10.72	6.07	11.06

Return period	25 year		50 year		100 year		250 year	
Direction	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)	Hs (m)	Ts (sec)
N	3.35	8.22	3.57	8.49	3.74	8.68	4.02	9.05
NNW	5.60	10.63	5.98	10.98	6.27	11.25	6.76	11.73
NW	6.07	11.06	6.43	11.39	6.79	11.70	7.29	12.17
WNW	5.51	10.54	5.89	10.90	6.19	11.17	6.69	11.67
WNW	5.04	10.08	5.27	10.31	5.59	10.61	6.01	11.05
WSW	5.32	10.36	5.63	10.65	5.84	10.65	6.35	11.37
SW	5.70	10.72	6.07	11.06	6.36	11.32	6.85	11.79

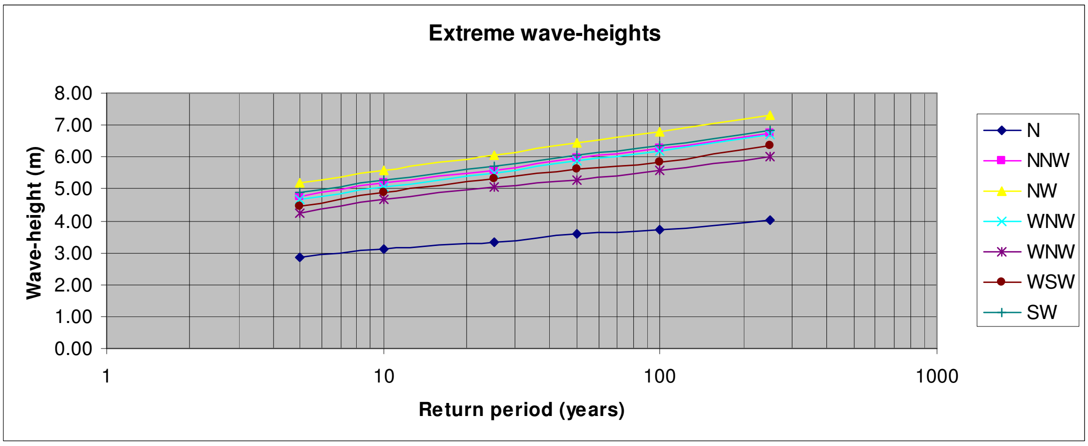

Figure A.3 Extreme wave heights plotted against the return period on a logarithmic scale

Appendix B Literature study

When generating alternatives, various aspects need to be looked at. Also, different sub-problems can have many different kinds of solutions. To gain a better understanding, this chapter will list the governing principles of every subject, aspect, or solution-mode relevant to this project. This information coming from the available literature can also be used later, when the alternatives have been created, and when alternatives are being further elaborated. When this stage in the project has been reached, this chapter can be used as a reference-guide.

B.1 Coastal protection

B.1.1 Optimization of the dike profile

The profile of the dike can be optimized in various ways, according to (ref *TAW Technisch Rapport waterkerende grondconstructies, Hoofdstuk 6 Optimalisatie van het dwarsprofiel*). These optimizations can be divided in to three sections, namely:

- Ground-elements and materials
- Ground-research
- Ground-improvements

<u>Ground elements and materials</u>

When optimizing the cross-profile of the dike, looking at the ground elements and materials, the next variables will be looked at:

- Angle and shape of the outward slope
  The reduction of wave run-up can be obtained by a mild outward slope, using a high foreshore, wave retarding structures, a rough outward slope or a "berm" on the right height. An example of these variables is given in figure. By reducing the wave run-up, the crest height can be reduced.

- Strength of crest and inward slope
  By allowing more wave-overtopping, the crest height can be lowered, although the requirement for this is that the strength of the crest and the inward slope has to be sufficient. Also measures have to be taken for drainage of the inward slope.

- Revetment outward slope
  With a high foreshore a good grass revetment can be sufficient, in all other cases the tidal zone has to be protected with stone materials. Under MLW + 0.5 meter the defense can only be applied as a fascine mattress. From MLW + 0.5 to MHW the revetment has to consist out of pitched stones, block mats, etc. Above MHW a heavier version of the revetment is needed because of the wave attack.

- Angle and shape inward slope
  The angle of the inward slope and the dimensions of the inward "berm" are determined by stability requirements. The nature of the stability problem determines the most effective solution and the

dimension of the inward "berm" is also dependent on the crest height.

- Structure ground-body
  Stability aspects are highly dependent on water pressures in the dike body, and there by also on the structure of the ground body. Aspects to be taken in account are piping and the level of the water line inside the dike body.

<u>Ground-research</u>

The extent of the ground-research depends on the phase of the project. In the early phases a general picture of the soil will suffice. This picture has to be accurate up to a level, so that in a later phase it won't turn out that a wrong decision was taken. It is important to determine the dependence of the project on the soil conditions and the probability that the soil has other characteristics then expected.

<u>Ground-improvement</u>

A division can be made between improvements in a weak subsoil and a permeable subsoil. On a weak subsoil a large excess height is needed to compensate the expected settlement. There is also extra lateral space needed to give the dike its needed stability in the form of gentler slopes. In the optimizing process it can be worth investigating removing the weak soil and replacing it by sand.

Permeable soil layers can induce piping problems; to avoid these provisions can be made by means of "pipingberms". Wide "pipingberms" can be undesirable, so other solutions should be accounted for.

B.1.2 Revetment

The function of the revetment is to protect the underlying body from erosion and/or reducing the wave run-up to maintain the strength required. Also there is a need of limiting the required maintenance; the damage should always be less than the economical tolerable damage. The revetment has an esthetical and environmental contribution. The environmental contribution will be dealt with later on.

The revetment is not allowed to fail during loading with the norm frequency, the dike self is not allowed to fail as a consequence of failing of the revetment.

There are three main categories of revetment, these are listed below and are shown in figure B.1:

- Loose rock
  This is a top layer of dumped materials, placed on a geo-fabric or a granular filter. Loose rocks on dikes are usually only used below Mean Water Level, mainly because of accessibility and esthetic reasons.
- Closed layer
  This is a watertight top-layer without a filter. Examples of this type are asphalt and concrete revetments and clay with grass revetments. Asphalt and concrete revetments are used when wave-attack is severe. Regular inspections for cracks and fissures are mandatory.

- Semi-open layer
  This layer consists of a permeable top-layer of pitched stone on a filter construction on a non permeable clay layer. The forces on this top layer are determined by the ratios between the thickness and the permeability of the different layers.

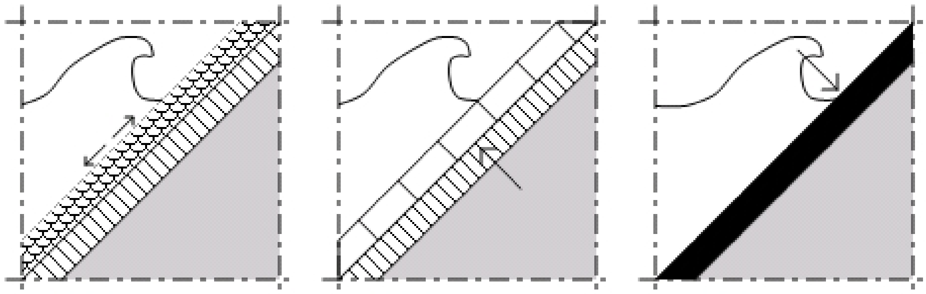

Figure B.1 Different types of revetment: Loose rock, semi-open and closed layer (adapted from: *Technisch rapport waterkerende constructies, TAW*)

The filter placed below the outer placed-block-revetment has two functions. It safeguards the geotechnical stability, meaning that it prevents material from washing out. Secondly, it is used to level out dimensional errors. For more geo-mechanical calculations on stability, see *TAW leidraad Toetsen op veiligheid*.

B.1.3 Soft solutions

Soft solutions can be seen as "working with nature". A dynamical equilibrium is possible with a ground body build out of sand. The profile of the coastal protection adapts itself to the hydraulic boundary conditions. During storms the profile flattens and during calm periods the profile becomes steeper again. Import guidelines for dynamical equilibrium can be found in ref *Leidraad Zandige kust, TAW*.

Soft solutions can be a good solution when a coast is eroding, this because of the fact that a hard solution does not stop the coast from eroding. For example, there is the danger that in the long run the complete beach will disappear, due to a beach-wall. There are however some hard solutions to solve the structural erosion, but on the long run these will be more expensive than a soft solution in combination with regular maintenance, for example regular beach nourishments.

B.1.4 Hard solutions

A hard solution is a solution where no dynamical equilibrium is allowed, and thus has to be protected against waves and currents. With eroding coasts an application of solely a hard solution is never a good solution. A soft solution is always required this because of the continuing erosion and the possible instability of the hard solution. Mixed solutions can always be applied.

A hard solution can be ecological sound, when the following aspects are taken in mind:

- Use of area characteristic material;
- Use of material on which and between animals on plants can settle;
- Use of material which does not harm the environment;
- Limit the use of material.

B.1.5 Ecologically sound shores

When assessing the possibility for the development of nature near or on a dike, a couple of restrictions have to be taken into account: the safety against flooding, the depth of nearby (shipping-) canals or gullies, and the quality of the water and the soil.

There are also determining factors for the development of nature:

- Tidal difference
  The duration of flooding or exposure to a dry environment determines the supply of food for organisms living in this zone. Different animals or organisms prefer different depths or exposure-times.
- Erosion/sedimentation and soil-composition
  On places with large dynamics in sedimentation and erosion not a lot of fauna is found. These are found more on places with calmer waters.
- Salinity
  Only a few kinds of plants adapted for saline environments. As the salinity of the environment decreases, more normal kinds of vegetation will occur.
- Climate
  Extreme events such as floods or droughts can have an impact on the composition of both vegetation and animal-population alike.
- Different functions for people and animals
  While the prime function of the coastal protection is to prevent flooding, it also has other functions, such as foraging grounds for animals, and recreational areas for people.

The *TAW* has issued a number of guidelines to be used in designing and constructing an environmentally sound dike. Important guidelines are: *Leidraad zee- en meerdijken, Grondslagen voor waterkering,* and *Zandige kust*.

B.1.6 Construction

Looking at the construction of the dike there are several aspects, to which attention has to be paid. These aspects are the deposit of the soil and the compaction of it, stability during construction and the excess height by completion.

The soil can be deposited in two ways, namely dry deposit and wet deposit. With dry deposit of the soil, the soil is being carried with trucks and dumped on site as good as possible into the profile of the dike. Sand can be handled rather well, depending on the moisture rate. A good compaction of the soil can be obtained simply by driving the construction traffic in specific manner. Clay has to be compacted with vehicles with tracks, because other vehicles cannot manage to realize the needed compaction.

The soil can also be deposited with water, i.e. large amounts of sand are transported in suspension. The sand can be deposited as well under water as above. While depositing the sand underwater, a loosely packed soil will be obtained, which is sensitive to flow slide. It is better to first construct a floor of coarse material that is higher than MLW, before applying hydraulic deposit of sand. The construction rate of hydraulic deposit is almost always higher than when dry deposited, this in combination with the presence of water can be rather complicated.

During construction it is important that the stability of the construction and supporting constructions is guaranteed at all times. For example, if there are weak soil layers present, it is important to calculate whether the construction is possible. If the sand can not be applied at once without the loss of stability, additional measures are necessary. These measures can be, phasing the project, applying vertical drains, soil improvement, improvement of the stability with geo-textiles, appliance of light material and in exceptional cases adaptation of the design.

Due to settlement of the whole construction, an excess height is necessary. A guess of the expected settlement can be made with the help of a settlement-beacon. It is important to wait with the finishing of the dike, i.e. the appliance of the hard revetment, until the rate of settlement has decreased. If the revetment is applied too soon, damage will occur.

B.1.7 Failure mechanisms for dikes

There are several ways in which a dike can fail. The aspects on which a dike can fail are:

- Crest height
- Inward and outward macro stability
- Micro stability
- Stability in case of wave overtopping
- Sand carrying springs (piping)
- Stability foreshore

Attention has to be paid to remediation measures with each design. This design is then tested according to the program of demands, after which the design can be optimized according to the first paragraph.

When the strength and stability of the design are reviewed, the following aspects are important:

- Loads on the construction
- Strength-characteristics of the soil
- Mathematical model used for checking whether or not the ULS has been passed
- Geometry of the construction

<u>Crest height</u>

The crest height is determined according to the decisive high-water levels, high-water surges, local water-elevations, wave overtopping height, and the strength characteristics of the soil. Other important factors are the geometry of the outward slope and the time. More info on quantifying crest height is found in *leidraad Zee- en Meerdijken*.

Settlement can also lead to failure of the dike, since the crest height could fall beneath its designed height, causing excess overtopping. The total settlement can be divided into three kinds; settlement of the subsoil, set, and soil-subsidence.

<u>Macro stability</u>

Macro stability concerns the resistance against the creation of straight or curved slide-plains and the loss of equilibrium due to the creation of large plastic zones. For the analysis of macro stability the following data is needed:

- Geometry; the cross profile
- Layer configuration of the subsoil and the dike
- Volumetric weight and strength-characteristics of each layer

- Loads; decisive location of the freatic line and water pressure gradient in the subsoil.

For reviewing the macro stability two methods are used; slide plane calculations and Finite Element Methods. The first method is the one most commonly used and is the less laborious of the two. More info on both methods is found in chapters 5.3.3 and 5.3.4 of *Technisch rapport waterkerende grondconstructies*.

Micro stability

Micro stability regards the stability of soil layers with limited thickness at the surface of the inward slope, affected by groundwater flowing through the dike. Problems are caused from within the dike: micro instability is usually the effect of high water pressures within the dike, pushing away the top layer if it's less permeable than the core, or the washing out of ground particles because of groundwater spouting out of the dike, if the top layer has more or less the same permeability of the core.

Stability in case of wave overtopping

Overtopping of the dike will lead to water infiltrating the inward slope and eroding its cover. This can cause a decrease of the stability of this slope, possibly leading to failure in the way of a slide plane.

Most dikes are designed with such a crest height, that overtopping should never be a problem, and doesn't have to be used as a design criterion. However, in complex situations it may be necessary to allow a certain overtopping-discharge. In this case the crest and the inward slope have to be designed in such a way, that they can withstand this discharge.

Sand carrying springs (piping)

The stability of ground structures can be threatened due to heave and piping. The lift of ground particles and the regressing erosion are being induced by the high piezometric head which causes ground particles to be carried along with the water flow in the sand layers. Due to this transport mechanism progressive canals can develop underneath the structure. Besides that, the grain pressure can drop.

Both phenomena can be prevented by using "kwelbermen" or "kwelschermen" (e.g. Sheet piling). For an example of a "kwelscherm", see figure B.2. This measures lengthen the path water has to travel, which reduces to flow velocity and the piezometric head.

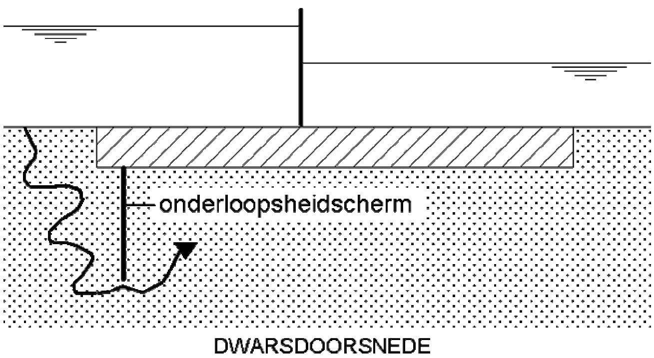

Figure B.2 Example of a "kwelscherm" [3]

[3] http://www.ipl.citg.tudelft.nl/lexicon/figuren/figA/Image2.gif

Stability fore shore

If there is a slope beneath the water level in front of a ground structure, with or without a foreshore, instability of this slope can be expected. This possible instability can be divided in instability due to shear and instability due to flow slide. This can influence the stability of the coastal protection. For guidelines and calculation methods see (ref) *Technisch rapport waterkerende grondconstructies, TAW, paragraph 5.7.4*.

Instability due to shear can occur at foreshores which consist out of cohesive materials like clay or peat and sand. If a layer beneath the fore shore has bad soil properties (cohesion and internal angle of friction) there has be paid attention to shear along a straight shear plane.

Instability due to flow slide can occur at foreshores which consist out of loose sand. This is a mechanism where water saturated sand undergoes large displacements. The sand "flows" as a result from liquefaction. Liquefaction is the result of loose sand and a bad geometry, although there is always a cause needed, like

- A rise in shear stresses;
- Becoming steeper of the slope or deepening of the gully;
- Applying a surface load;
- Vibrations;
- Wave-loads during severe storms;
- A quick drop of the outside water level.

B.2 Land reclamation

Many countries have reclaimed land from the sea, varying from cultivating tidal flats to making artificial land by depositing sand on a special place. Land reclamation is a solution for long term spatial problems. First a short introduction on land reclamation in various countries will be given, then some of the problems will be discussed which are related to the reclamation of new land. Last the ways how land reclamation can be achieved are discussed.

B.2.1 Ways of land reclamation

There are two ways of land reclamation. The first is the so called land-fill method, which is used for example in Singapore. Large amounts of sediment are harvested, either land-based from excavating hills, or dredged from the sea-bottom, and are then deposited on the site. To protect the new land face from erosion, a dike or revetment is built. The second way of land reclamation is by building a dike around the targeted area. The area is then drained by gravity if the height of the land allows it, or pumped dry if artificial drainage is necessary. Due to settlements and the original geography of the area, drainage is always necessary and measures have to be taken to let the water out and keep the sea at bay.

B.2.2 Land reclamation projects

In Korea there have been many small and large scale land reclamation projects, varying in size from several hectares up to 18.000 hectare. The main use in the early years was purely agricultural, for as the tidal flats offered a very good basis. This did not change until the 1970's, when they where also reclaimed for other purposes like irrigation and infrastructure. After 1980's the urbanization of the country was the driving force and the tidal flats were converted into industrial and urban areas. The inter-tidal areas were primarily seen as an area to seek new land for economic growth; the natural value was not recognized.

No other country in the world has grown as fast as Singapore. In the last forty years, Singapore has grown 20%, as can been seen in figure B.3 on the next page. The method used in Singapore is the landfill-method. First they used sediments from the only hill to reclaim land. After that they bought sand from neighboring countries such as Indonesia. There are many dredging companies working to make the island grow, however activities have been slowed down since Indonesia has stopped selling sand to Singapore.

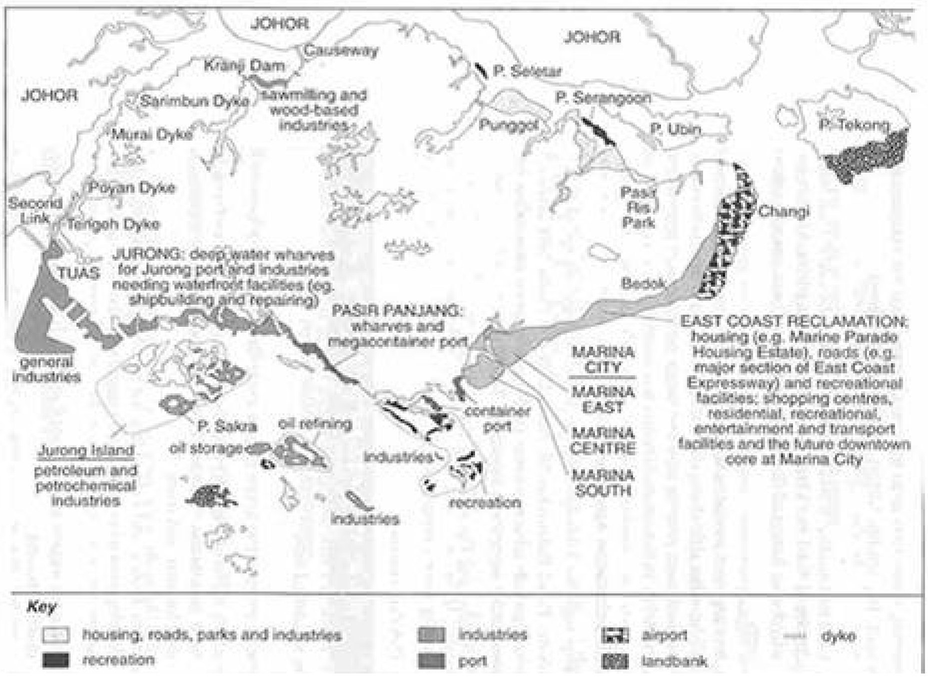

Figure B.3 Land reclamation in Singapore[4]

B.2.3 Known problems

The current coast is propagating, thus there is more sedimentation than erosion. It is not obvious that this is still the case if land reclamation has taken place. Currents can be changed into directing sediment off-shore instead of depositing it on the coast. Also problems can arise in regard to navigational channels due to the changed pattern of sedimentation and erosion. When developing a new land reclamation area west of Rotterdam in the Netherlands, simulations have been applied to optimize the shape of the reclamation. This was done to protect weak coasts from extra sedimentation[5].

Tidal areas have a unique eco-system. They are formed by deposited fine sediments and organic particles, so it is a very fertile area. Thereby it forms a spawning ground for fishes and it provides a feeding ground for a variety of birds, including migrating birds. Furthermore it can act like a filter; some of the organic particles can bind hazardous pollutants to them, preventing them from being dispersed into the environment. Another important role of the tidal flat is that water has to advance over it before it reaches the coast, dissipating wave

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4 http://library.thinkquest.org/C006891/reclamation.html

5 Project Leeuwestaart, Technical feasibility of a largescale land reclamation, WL | Delft Hydraulics, 2005

energy as it does. Tidal areas are important for both humans and nature, so when they have to be modified, it has to be done with great care.[6]

When land is reclaimed from the sea there will arise two problems. The first problem is the increased turbidity of the water, which can be disastrous to sea-life. In Singapore land is land reclamation has dramatically decreased the underwater visibility from 10 meters in the early 1960's to less then 2 meters in the late 1990's[7]. This destroyed almost 60% of Singapore's coral reefs. The other problem is, when dredging or mining sand, the coast will from a new equilibrium, hereby threatening current beaches or even small islands. The place and the method by which the sand is mined have to be carefully chosen.[8]

B.3 Drainage

During the typhoon season there is a high risk that the present drainage system will fail. Due to the fact that the outlets are clogged with waste material, sediment and tree trunks from the mountains, transportation of the water into the sea is blocked. It is not clear if the drainage system would be sufficient if the outlets are cleared of debris. It is also possible that some alternatives in the next phase demands changes in the current drainage system. Therefore two methods to determine the discharge of the several small drainage rivers will be lined out. The first method is the Rational Method. The second method is the Unit Hydrograph / Curve number method.

B.3.1 Rational method

The rational method is a method which gives fast practical results for smaller areas up to 80 hectares. The Rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed).

The rational formula:

$$Q = C*I*A/360$$

Where:

- C = Run-off coefficient which depends on the type of drainage area
- I = Average rainfall intensity [mm/hr]
- A = Drainage area [m^2]

The average rainfall intensity 'I' depends among other things on the time of concentration 't_c' which is difficult to determine without calibration. The rate of runoff resulting from any constant rainfall intensity is at its most when the duration of rainfall equals the time of concentration. That is, if the rainfall intensity is constant, the entire drainage area contributes to the peak discharge when the time of concentration has elapsed. This assumption becomes less valid as the drainage area increases. For large drainage areas, the time of concentration can be so large that the assumption of constant rainfall intensities for such long periods is not valid, and shorter more intense rainfalls can produce larger peak flows. Additionally, rainfall intensities usually vary during a storm.

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[6] http://www-cger.nies.go.jp/lugec/Proceedings/12)Manik%20Hwang.pdf

[7] http://www.oceansatlas.com/servlet/CDSServlet?status=ND0xODAwOCZjdG5faW5mb192aWV3X3NpemU9Y3RuX2luZm9fdmlld19mdWxsJjY9ZW4mMzM9KiYzNz1rb3M~

[8] http://www.ecologyasia.com/html-newsurl/singapore-land-reclamation.htm

B.3.2 Unit hydrograph/Curve number method

Several runoff determination techniques are developed by the U.S. Department of Agriculture and Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS). The following CN-method is one of them. In origin the method is meant for American circumstances, but nowadays it is used worldwide. Because the method is based on empirical relations, unique for a river it is difficult to use in other areas.

The method is a combination of the Unit hydrograph and the principle of precipitation-retention determined with the Curve Number method. The curve number method determines the retention of a certain area. The Curve number is the relation between the precipitation and the drain height. This number is empirically determined for various characteristic areas. In the Unit hydrograph there is a relation between precipitation, drain height and the discharge of the river. The discharge will be determined for a standard rainfall, for example a rainfall of one mm in one hour. With this standard rainfall more complex rainfall can be calculated as well.

The Unit Hydrograph method provides good results. However in the case of this project it is very difficult to deal with the empirical relations, since there is no known measurement data from the selected area.

B.3.3 Conclusion

The concentration time for the Rational Method is difficult to determine but easier to estimate than the empirical relations of the Unit Hydrograph. The Unit Hydrograph method is more sophisticated and gives better results if the input is correct. Correct input is difficult to realize and the less accurate results for the Rational Method will satisfy the purpose of this project.

B.4 Retention and detention reservoirs

One of the many possible solutions to the drainage-problem behind the coastal protection may entail the use of a reservoir for the retention or detention of water. This paragraph will briefly describe what retention and detention, and what processes are involved in designing a reservoir.[9,10]

B.4.1 Retention

A retention basin is a natural or artificial wetland, which is used to temporarily store excess water. This surplus of water comes from heavy rainfall. Water, which would otherwise flow into other areas, is now directed into this basin. This water will remain here, in other words it will be retained in the local area in which it was deposited. There the water will be stored for later use.

B.4.2 Detention

One speaks of detention of water, when the water isn't stored in the basin for a longer period of time to be used later, but rather when the water is briefly stored, and then released into another water body in a controlled manner. In the case of a detention basin the excess water can come from both heavy rainfall and/or high river levels. By definition the only primary function for a detention reservoir is to deal with excess water, while the retention reservoir also serves

[9] http://en.wikipedia.org/wiki/Detention_basin

[10] http://clean-water.uwex.edu/plan/drbasins.htm

as a water supply source during droughts. For the rest retention and detention basins are more or less similar to each other.

B.4.3 Design parameters for reservoirs

The size of the reservoir depends on the size of the area it drains from. It also depends on the amount of water it is supposed to store during an extreme event, such as a storm, a river flood, or a monsoon. This amount is usually defined by the return period of the extreme event, e.g. a 100 year storm or a 50 year river flood. Another important factor is the amount of impervious surface area in the area drained from. More impervious area means less storage in the soil and faster run-off. The size of the reservoirs varies between 4000 and 50000 square meters, and the depth can range from three to twenty meters.

The basin is always equipped with a small spillway, in case the inflow of water ever exceeds the capacity of the basin. A detention basin also has a main outlet, apart from the spillway. The size of this outlet is determined by the speed in which the water has to be discharged, but is also restricted by the downstream canal's capacity to handle such a discharge.

Another factor which can play an important role in designing a reservoir is the amount of mixing in the water. If the water in the reservoir remains stagnant for too long, or if the basin suffers from excessive eutrophication, algae-blooms or other kind of pests can occur.

Reservoirs can have secondary functions next to the intended storage of water. For instance, it can be a place for ecological development, recreation, and maybe even fishing farms.

B.5 Waterborne sediment transport

In coastal engineering there are two main kinds of sediment transport, which are to be taken in account. There is longshore transport along the coast and cross-shore transport perpendicular to the coast. With each kind two processes are relevant, structural erosion/accretion and erosion/accretion during storms. The driving powers behind waterborne transport of sand are tides, waves and wind.

B.5.1 Longshore transport11

The longshore transport of sediment can be divided into three parts: the transport due to tidal current, the transport due to wind-driven currents, and the transport due to wave-driven current.

The tidal current is only present along coasts with horizontal gradients in tidal phase. The exact tidal forces on the water column can be computed if the different tide-characteristics like phase and amplitude are known. However, using the Chezy-formula, it can be derived, that the tidal current moving along the shore will have lower velocity in shallower depths. In shallower regions towards the coast, the tidal velocities will be less, and increased bed shear-stress due to waves will cause the velocities to be even smaller.

When wind blows over a water surface, the upper parts of the water layers will start to move in more or less the same direction as the wind direction. In land- and seaward-direction, this will result in an opposite directed current in the lower water layers and a water level set-up or set-down. In directions parallel to the coast a longshore current is created. However, the highest velocities occur at the

[11] CT5309 Lecture notes, chapter 6

water level, and quickly decrease towards the bottom, where the highest sediment-concentration occurs. Therefore, the effect of wind on longshore current can often be neglected, except for during storms.

The wave-driven current is located in the surf-zone, where the waves are breaking. Because of this breaking, the longshore radiation stress-component will change, resulting in a force on the water column. Since there is no closed boundary in the water along the shore, no hydraulic pressure gradient can develop, and the water column will accelerate. This current will induce a bottom shear-stress, which will restore the equilibrium of forces in the water column.

Longshore transport will determine to a large extent the shoreline dynamics. The biggest changes in recession or progression of the coast will occur at the places with the highest gradients in longshore-transport. If the transport increases along the coast, the shoreline will erode and move backward. If the transport decreases, the shoreline will move seaward.

To model the actual sediment transport in these currents, several methods have been devised. Most used longshore transport formulae are the CERC- and the Bijker-formula. Both of them are described extensively in the CT5309 lecture notes.

B.5.2 Cross-shore transport12

Cross-shore sediment transport only describes the transport in onshore or offshore direction, perpendicular to the coast. This will only change the shape of the cross-shore profile, without actually changing the total amount of sediment per running unit of length of coast.

The most relevant processes and phenomena affected by or affecting cross-shore transport are: the dynamics of bars in front of the coast, changes on profile during different seasons, dune erosion, scour in front of hard elements, sea level rise, land reclamation, and shoreface nourishments.

To model cross-shore transport the same basic formula that exists for longshore transport has to be solved, although this is more difficult due to different time-scales and other more complex driving forces.

B.5.3 Classifications of coasts13

Coasts can be classified according to either their geological situation, or their hydraulic boundary forces. When classifying according to geology, three different kinds of coasts can be discerned, namely collision coasts, trailing edge coasts and marginal sea coasts. Collision coasts appear on the rims of active colliding tectonic shelves. In fact, the east coast of Taiwan can be classified as a collision coast with its steep cliffs and rapidly declining depth-contours. Trailing edge coasts appear on places where tectonic shelves diverge. Typical trailing edge coasts are the Brazilian and American Atlantic coasts, and Egyptian Red-sea coast. Trailing edge coasts can be very variable, with either lowlands dropping into the sea, or also steep shores, much like the collision coasts. Marginal coasts are coasts protected from the open ocean by other land-masses, and show the greatest diversity of all coasts. The west coast of Taiwan is an example of such a coast.

[12] CT5309 Lecture notes, chapter 7
[13] CT 5303 Lecture notes, chapter 2

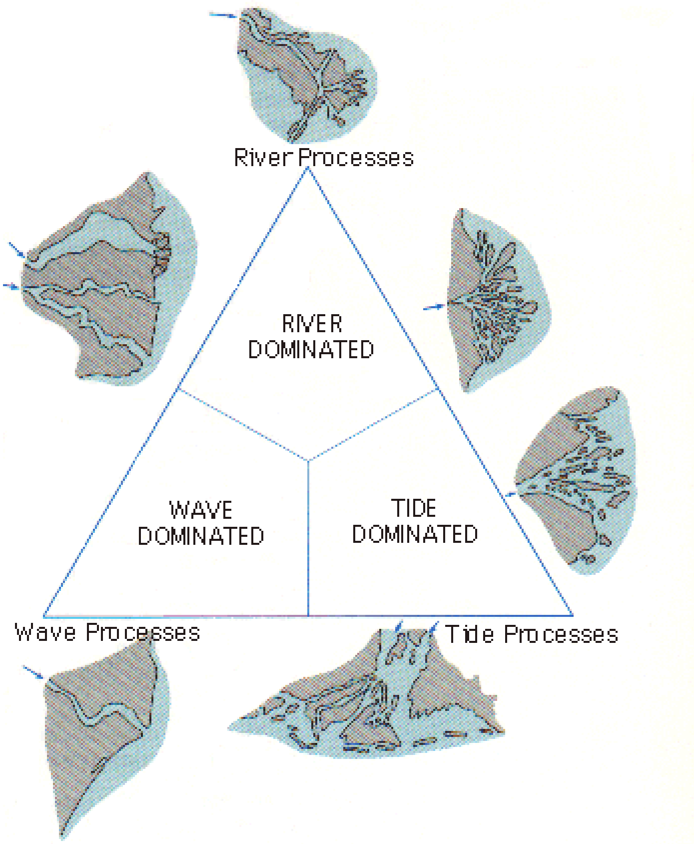

Figure B.4 Classification according to the hydraulic boundary forces[14]

Another classification is according to the hydraulic boundary forces, namely the storm dominated, ocean dominated, wave dominated, tide dominated and river dominated coasts, see also figure B.4. Most often, a coast will be affected by various hydraulic boundary forces, and not just by one. Most important the various combinations amongst river-, tide- and wave-dominated coasts are seen al over the world.

B.5.4 Interactions between river and coast15

The sediment coming out of the mouths of rivers can be deposited in several ways. Either it is taken by the longshore current on a mixed energy coast, and deposited along the shore, or the sediment simply flows into open water and is deposited in front of the delta or estuary, when there isn't a current to speak of which can take the sediment elsewhere. Transitional forms of the movement of sediment out of rivers and along coasts are also possible.

B.5.5 Coastal protection16

When coasts are suffering from either structural erosion or erosion during severe events such as storms, several countermeasures can be taken, like shoreface nourishments, seawalls, revetments, breakwaters, groynes, and detached breakwaters. All these measures, except for the nourishments, involve the construction of structures in or near the sea. There are benefits as well as

[14] http://maritime.haifa.ac.il/departm/lessons/ocean/lect20.htm
[15] CT 5303 Lecture notes, chapter 3
[16] CT 5309 Lecture notes, chapter 12

disadvantages for every measure, and these should be looked at when deciding which measure to take.

- Nourishments
  The depleted supply of sand on a beach is simply replenished by dredging sand from one place, and put it on the beach. While this may seem not to be a durable or sustainable solution, it can be the cheapest solution over the long run, without much negative side-effects.

- Seawalls and revetments
  A seawall is actually just an artificial dune-face. The only difference is the seawall isn't supposed to erode during storms, and in this fact also lays its biggest disadvantage. There is no more supply of sand from the dune to the beach. This is why a seawall is no solution for a structural erosion problem, and can in fact effectively cause the beach to disappear into the sea. A revetment is similar to the seawall, but has a distinct slope, while the seawall has an almost vertical face.

- Breakwaters and groynes
  The differences between breakwaters and groins are in size. Both are structures extending perpendicular to the coast, but breakwaters are much longer, wider, and higher than groins, and are not applied in series. In coastal sediment transport processes they behave more or less the same way. Both of them interfere in the longshore current and slow it down. When this current slows down, sedimentation will occur updrift of the structure, while downdrift erosion will occur.

- Detached breakwaters
  These breakwaters are comparable in size to the groins, but run parallel to the beach. They are most often applied in serried with gaps between them. The breakwaters reduce the wave height, and thereby the velocity of the wave-driven current. However, to be effective even during storms they have to be high, and the gaps between them spawn rip currents.

B.6 Windborne sediment transport

One of the problems faced at the project area is the nuisance caused by sand. On the one hand there is the nuisance of sand blown over the dike in the urban area and on the other hand sand causes trouble when blocking the outlet canals and outlet valves.

Because large plains fall dry with low tide and there normally is a moderate to powerful monsoon wind and occasionally an extreme typhoon wind, it is expected that sand transport by wind plays an important role in the total sediment transport at the coast of the project area. Therefore it can be important to estimate how much sand is transported in what direction and where it will be deposited. Furthermore if a solution is presented which involves the artificial creation of dunes for example by sand fencing an estimation of the dune growth and an assessment of the effectiveness of sand fencing is indispensable.

In this paragraph a literature study is presented about wind-blown sediment transport. It has to be noted that wind-blown sediment transport is dependent from many factors, thus the result of the method will be only a rough estimation of the possible sand transport. Most of the information gathered comes from the USA Corps of Engineers Coastal Engineering Manual 2002.

B.6.1 Processes

Three processes are important in wind blown transport, namely saltation, suspension and surface creep also referred to as traction. In figure B.5 the processes are depicted.

Saltation is the process of sand grains moving by bouncing along the surface. The sand grains are carried up into the moving air by turbulence and are pushed in the wind direction. Then they settle again due to their weight. When fine grains are incorporated into the atmosphere one speaks of suspension. The process of sand being moved on the surface because of the impact of settling sand is called surface creep or traction. Saltating grains are capable of moving much larger surface grains by surface creep due to their impact. On beaches, saltation is the more important of the two processes.

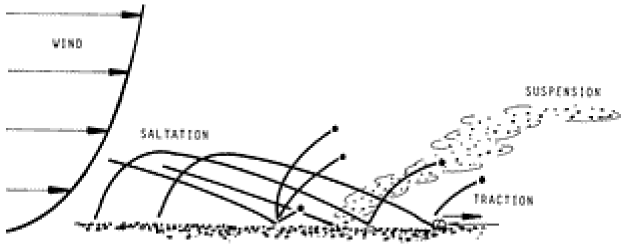

Figure B.5 Sand transport by wind[17]

The extents in which these processes take place depend mainly on the wind speed near the surface and the size and density of the sand grains. The wind speed near the surface on its turn depends greatly on the type and form of the surface and the presence of obstructions.

B.6.2 Procedure for estimating wind-blown sand transport

The following data is needed:

- (Preferably hourly) average wind speed and direction data
- Precipitation and evaporation records.
- Density and median grain size of beach sand at study site.

With these data the following components need to be computed:

- Critical shear velocity for the mean grain diameter.
- Critical (or threshold) wind speed at 2 meter height.
- Potential sand transport rate.

The critical shear velocity for sand depends on the soil moisture, so a distinction has to be made between 'dry' and 'wet' days, wet days being the days in which precipitation exceeds evaporation. Also snow days must be taken in account, because there is no sand transport until the snow layer has melted.

The critical wind speed is the wind speed above which sand transport takes place. There are empirical formulas to calculate the critical wind speed at 2 meters above ground level depending on the type of area. However since mostly the data available are wind speeds at higher altitudes the critical wind speed at 2 meters must be converted to critical wind speed at these higher altitudes using a power-law relation.

[17] http://www.deh.gov.au/coasts/publications/nswmanual/appendixb9.html

If the wind speed data available is derived from land based weather stations the wind velocity must be converted to sea wind velocity's this can be done by a number of empirical formulas depending on the situation.

B.6.3 Wind-blown sand transport and dunes

Sand dunes are created, enlarged and altered by wind-blown sand. They are created if sand is trapped due to the presence of vegetation or other obstructions to the wind. Dunes can fulfill a number of functions, amongst others protection from flooding, erosion and wave attack. They can also shelter the landward area from wind-blown sand.

Artificially creation of sand dunes can be done in three ways: by beach nourishment, grading existing sand on the dry beach or 'beach scraping' which is removing sand from below the high water line at low tide and using it to construct a protective dune. The natural development of dunes by wind can be stimulated by planting vegetation or installing sand fences. This can be done in cases where there is enough sand supply and there are wide beaches. Because of the influence of dunes on the wind pattern, see figure B.6 there is very little sand transported from the dune to the beach by wind. Dunes grow in the direction of the sand supply, so they grow seaward.

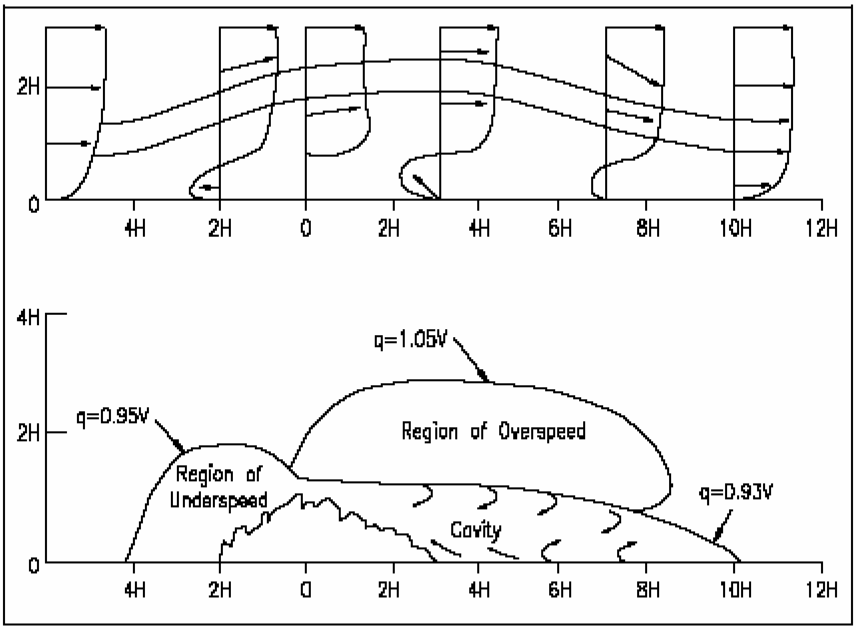

Figure B.6 Influence of dune on wind pattern[18]

When sand fences are used to stabilize dunes regular inspection, replacement of damaged fences and placing of new fences is needed in order to continue the stabilizing process as the dune grows. The shape and size of the dunes can be determined by strategic locating of the fences. Mostly the growth of the dune is limited rather by the limited sand supply than by the capacity of the fences. In most cases the fences are filled with sand within one year after construction. After a dune is created with sand fences vegetation must be planted to keep the dunes stable when the fences eventually deteriorate. Stabilizing by vegetation has the advantage that it can grow along with the dune and to certain extend recover itself when damaged. Vegetation is however very vulnerable and in the first two years after planting fertilization is needed.

[18] USACE Coastal Engineering Manual 2002

Dune growth depends on the wind direction relative to the dune axis, the ability of the dune to trap and hold sand and the limits of the sources of sand. So the potential sand transport rates which can be calculated using the procedure above need to be diminished with the losses of sand due to erosion. A simplified method to do this is by determining a sand trapping factor depending on the grain size grades of the beach and the dunes. In this way it is determined how much units of beach sand is needed to provide one unit of dune sand.

B.7 Debris

Since part of the cause of flooding problems at Taiwan west coast seems to be the blocking of outlets by waterborne debris there has been a survey on similar problems at other coasts in the world. It was found that in many countries floating debris forms a problem, but the problem is mostly pollution of the beaches and the inherent disadvantages to recreation and not blocking of outlets. Also it was found that there is currently much research going on about woody debris which focuses on the positive ecological effect of woody debris in rivers.

Figure B.7 Debris next to outlet near Wuchia port

B.7.2 Causes

When speaking of floatable debris a distinction can be made between larger mostly woody debris or debris from decaying vessels or rafts and smaller debris, mostly plastic waste. Common examples of the latter are: beach litter, street litter, fishing gear, aquatic vegetation, sewage related waste and medical waste. The sources can vary to a great extend, and can be divided in marine based and land based sources[19]. Also the region of sources can vary greatly. In a study on Canadian coast it was found that debris from 36 different countries was washed ashore on the beaches.[20]

Furthermore a distinction can be made between natural debris and man made debris. The former is more accepted on beaches with less intensive recreational use or in periods with less waterfront activity. Land based sources of debris consists of debris from beach users on the one hand and sewage overflows, storm water discharges and more general (often illegal) littering practices on the other hand. Large woody debris is another form of debris from land based sources and has natural causes like storms. The wood is deposited in the rivers up streams and in the wet season transported to the sea. Marine based sources

[19] http://www.science.ulst.ac.uk/ics2002/tudor_dt%20et%20al.pdf
[20] http://www.pitch-in.ca/Media/E-Media1310.html

contain material swept away from vessels unintended, waste from vessels dropped into the sea and also pieces of fishing nets. Sea borne material can be washed ashore and away again on different places for several times so it is hard to determine when and where it was deposited into the sea.

Discharges from both sewer overflows and storm sewers are triggered by rainfall events. The correspondence, however, between rainfall events and floatable debris slick formation is based on a variety of factors including rainfall intensity, duration of rainfall, time frame between a particular rainfall event and the previous rainfall event, and the location of a rainfall event.[21]

The influence on wind on floatable debris is less clear, because the wash up of the debris depends on a combination of wind but also tidal movement and other than wind induced currents.

B.7.3 Impact

Dispersion and wash ups of debris have great negative economical and environmental effects. The economical effects relate to the limitations to beach use due to pollution. In the case of Taiwan west coast it is also suspected to be the cause of flooding, because it hampers dewatering of the area. These floodings induce damage to infrastructure and possessions of people living in the flooded area.

The environmental damage to be taken in account consists mostly of entanglement of sea life which results in starvation and suffocation due to swallowing of plastic material which is mistaken by prey.

A special form of debris is large woody debris in rivers. From recent research it follows that especially in sandy rivers large wood snags provide a habitat for many kinds of fish. Also large woody debris helps provide high water quality, prevents riverbank erosion and maintains aquatic habitat and physical setting.[22] It must be noted though that the rivers concerned in this research are generally slow flowing rivers, whilst at Taiwan west coast the rivers are very steep and can have extreme discharges and high current speeds during typhoon season.

Figure B.8 Woody debris in a river[23]

[21] http://www.epa.gov/region02/water/action_plan/assess98.pdf
[22] http://www.rivers.gov.au/model/woodydebris.htm
[23] http://130.18.140.19/noxubee/Resources/stream/woodydebris.htm

B.7.4 Solutions

The main solution provided for the debris problem is as obvious as it is common: cleaning up. In case of extensive beach use many beaches are subject to daily (or nightly) cleanings. In many cases this is done with mechanized sand rankers. These are mainly effective for larger materials. Therefore there are also a lot of efforts to be taken up by hand work. This can range from voluntary activities to educational activities to employing prisoners. To intercept waterborne debris also the use of skimmer and booming vessels is highly effective. With these countermeasures scheduling, communication and clear division of responsibilities are very important.

More structural solutions to prevent floatable debris from washing ashore are the use of catch basins and hooding and screening of debris carrying channels and creeks. This can be done with simple bar screens to more complicated raking machines. There are many different types of litter traps. Also the overall increasing of the capacity of sewer systems so overflowings will decrease is a structural solution.

There are also solutions on the behavior side of the problem, like public education programs.

Figure B.9 Impression of raking machine[24] and a floating litter trap[25]

B.8 Earthquakes

When designing structures in an area that suffers from strong seismic activity, these designs have to be tested on their ability to withstand the loads they are subjected to when struck by an earthquake. This chapter gives a general approach to testing a design under earthquake loads. This approach is described extensively in the book Seismic design guidelines for port structures written by the International Navigation Association (PIANC).

B.8.1 Earthquake motion[26]

One of the key parameters when looking at earthquakes is the Peak Ground Acceleration (PGA). This is the intensity of the bedrock motion. It can be split up into the vertical and the horizontal acceleration. The parameter is either used by itself, or to scale ground motion characteristics. When using a probabilistic approach this level of bedrock motion is defined as a function of a return period, or a probability of exceedance. Other relevant parameters are: peak ground

[24] www.bgusa.com/ SewageBoskerBrochure.html
[25] http://wsud.melbournewater.com.au/content/treatment_measures/litter_traps/litter_trap_types.asp
[26] Seismic design guidelines for port structures, page 7 & 130

velocity, accelerogram (time history of acceleration), peak ground displacement, duration, and the equivalent number of uniform cycles.

B.8.2 Local site effects[27]

The soil deposits between the bedrock and the surface can modify and even amplify the motion coming from the moving bedrock. It can change the amplitude, frequency and duration. Some soils tend to amplify the ground motion, while other kinds of soil can dampen the tremors. The main mechanism responsible for amplification is related to resonance of the vibration in the soil layers. When designing on earthquake loads, local site effects are incorporated by either using site amplification factors, based on older data, or a site-specific analysis is made.

B.8.3 Liquefaction[28]

One of the most important failure mechanisms that can occur during earthquakes is called liquefaction. When saturated soil is shaken rapidly by the earthquake, it will try to densify. The shearing motions that accompany this will cause the pore volume to decrease, and the pore water pressures to rise. If the soil is loose and cohesion-less enough, the pore water pressure can rise above the effective stress between the soil particles, and the whole soil will behave like a thick fluid. Bearing capacity is temporarily lost, and large deformations will occur. The pressure of the ground water may cause water spouting out of the ground. When the high pore water pressures have bled away in one way or another, the contact between the soil particles will be restored. Some soils will densify in the process, while others won't, and will remain susceptible for future liquefaction.

The potential for liquefaction of a soil is related to its ability to prevent excess pore pressures from developing, and to the magnitude, duration, and the number of cycles of the actual shear stress.

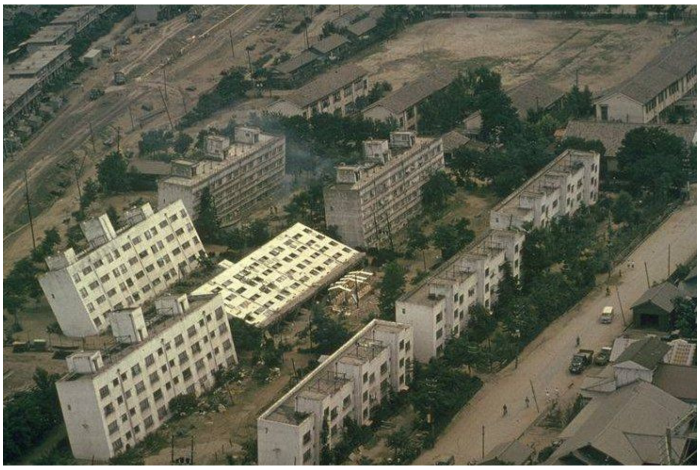

Figure B.10 Overturned buildings due to liquefaction, 1964 Niigata earthquake, Japan[29]

[27] Seismic design guidelines for port structures, page 8 & 138
[28] Seismic design guidelines for port structures, page 9
[29] http://www.ce.washington.edu/~liquefaction/html/what/what2.html

B.8.4 Liquefaction potential assessment[30]

The potential of soils for liquefaction to occur during earthquakes is assessed using field data e.g. coming from the standard penetration test (SPT) or the cone penetration test (CPT). This data is used to determine the resistance of the soil, quantified in cyclic resistance ratio (CRR). The seismic load is calculated using the design earthquake parameters. This load is quantified in the cyclic stress ratio (CSR). If the load is bigger than the resistance, the soil is deemed liquefiable.

B.8.5 Seismic hazard and design earthquake motion[31]

To calculate the maximum load on a structure during an earthquake, one must determine the earthquake motion parameters to be used in the calculations. Most codes and guidelines simply use the biggest earthquake on record in the region to determine these values, but if there are more sources of quakes involved, two other approaches for determining these values can be used; a deterministic and a probabilistic one. The deterministic approach generally takes the largest possible earthquake which could affect a given site under the known tectonic conditions, looking at the seismic history. The probabilistic approach uses several values for the motion parameters, all with their own annual exceedence probability. The procedures for both approaches are showed below.

Deterministic approach:

1. Step 1: Identify all the seismic active faults in the area of interest.
2. Determine the maximum credible earthquake possible for each fault, and determine the distance from the fault of this earthquake to the structure.
3. Determine the attenuation-relationships, meaning a relationship that describes how much the intensity of the tremor will decrease over distance. In a deterministic approach this is usually a relationship that corresponds to a conservative estimate of design parameters.
4. Calculate the values of motion parameters (acceleration, duration, etc.) at the site from each quake-source, and select the maximum.
5. If local site effects are important, convert the motion parameters to the local site conditions using site amplification factors.

Probabilistic approach:

1. Identify all the seismic active faults in the area of interest.
2. Define the parameters of occurrence statistics (probability distribution, recurrence relationships).
3. Determine attenuation-relationships. Parameters are estimated based on probabilistic evaluation of mean values obtained from the attenuation relationships. This relationship is calculated from statistical analysis of data from earthquakes in the same region or regions similar.
4. Calculate the annual number of occurrences of earthquakes from each source which produce a certain PGA at the site. Add these up to calculate the total annual number of exceedance. From this calculate

[30] Seismic design guidelines for port structures, page 60, 191, 199
[31] Seismic design guidelines for port structures, page 141 through 149

the return period. This will then be used to calculate the probability of that certain PGA being exceeded during the lifetime of the structure.

5. If local site effects are important, convert the motion parameters to the local site conditions using site amplification factors.

B.8.6 Damage to breakwaters[32]

The book "Seismic design guidelines for port structures" doesn't give specific guidelines on designing dikes, but it does deal with the design of breakwaters. Damage to breakwaters mostly occurs in the form of settlements beneath the breakwater, causing a decrease of crest height. This doesn't have to mean immediate catastrophe, because an earthquake isn't likely to occur simultaneously with the design water level. Also damage in the form of intrusion of the breakwater material into the soil underneath, or in the form of flattening of the cross-section because of slide plane failure can occur.

To analyze the perceptibility of breakwaters to these kinds of damage, the design has to be checked on macro-stability using either slide plane calculations or even finite element methods. The breakwater and the soil beneath it should also be checked for liquefaction potential.

Figure B.11 Earthquake-related failure of the coastal protection at Navlakhi port, India 2001[33]

B.8.7 Remediation of liquefiable soils[34]

There are several ways to prevent liquefaction from occurring altogether. These methods involve improving the quality of the soil to such an extent, that either the soil skeleton can withstand the forces caused by the shaking, or that excess pore pressures can not develop. A brief outline of remediation techniques is given below.

- Compaction
  The soil is densified using vibration or impact. This will make the soil less susceptible to liquefaction. Drawbacks are that the effectiveness depends on the soil gradation, the compacting itself causes noise and vibration hindrance, and influences the surrounding ground and adjacent structures. Yet, this is the most commonly used method to prevent liquefaction.

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32 Seismic design guidelines for port structures, page 21, 52, 96, 329, 354, 378
33 http://gees.usc.edu/GEES/RecentEQ/India_Gujarat/Report/AerialSurvey/Bardet_Feb12.html
34 Seismic design guidelines for port structures, page 273

- Pore water pressure dissipation
  Denying pore water pressures from reaching excess heights will also effectively prevent liquefaction. Therefore, the installation of permeable stone or gravel columns or other kinds of drains is an effective method. The installation usually cause less hindrance than the compaction-method, but requires more complex calculations, more data on many soil parameters, and is less reliable. That's why drainage solutions are usually only applied when other methods are deemed impractical or too difficult.

- Cementation and solidification
  Mixing the soil susceptible for liquefaction with a stabilizing material to cement the soil skeleton is another way to improve the strength of the soil. Attention has to be paid to the mixing of the soil with the stabilizing material, the quality control of the treated subsoil layer, and the quality of the groundwater.

- Replacement
  If a certain soil is extremely likely to liquefy, and can not be treated in anyway to withstand this, the best solution is to remove the soil altogether, and replace it with a filling better suited, such as gravel or soil mixed with the previously mentioned stabilizing material.

- Lowering of the ground level
  Lowering the ground water level will decrease the likelihood of excess pore pressures an will increase the effective confining stress between the soil particles, making it impossible for these pore pressures to approach the initial vertical effective stress. Disadvantages are the necessity to drain the area more or less continuously, which imposes costs, and the possibility of settlements in the surrounding area.

- Shear strain restraint
  A continuous underground wall around the soil will reduce the soil shear strain levels during earthquake loading. This method has advantages when access to the soil is difficult. As a 'wall' also a screen of the previously mentioned cemented soil is possible. Drawbacks of this method are the complexity of the design, and the justification of the solution.

- Preload
  Preloading methods overconsolidate the soil, thus increasing the resistance to liquefaction. This can be achieved either by applying a surcharge, or by temporarily lowering the ground water level. It is however necessary to have detailed information on the soil layers and on some places it may be difficult to achieve large overconsolidation ratios.

Appendix C Wave run-up and wave overtopping

The ultimate height of the crest of the dike above the design water level can either be defined as the wave run-up height, or the freeboard needed to counter a certain amount of wave overtopping. In this appendix the methods to calculate both the run-up and the freeboard will be explained.

C.1 Wave run-up

The wave run-up is calculated according to Dutch standards[35]. The general formula is as follows:

$$\frac{z_{2\%}}{H_{m0}} = 1.75 * \gamma_b * \gamma_f * \gamma_\beta * \xi_0$$

with:

H_{mo}	m	significant wave height at toe of the dike
ξ_0	-	Iribarren number
γ_β	-	incidence reduction factor
γ_f	-	roughness reduction factor
γ_b	-	"berm" reduction factor
$z_{2\%}$	m	2%-wave run-up height above still-water level

Comments on symbols and units:

γ_β The incidence reduction factor is dependent on the angle of incidence β: $\gamma_\beta = 1 - 0.0022*\beta$ with a minimum value of 0.8. However, when a typhoon passes over the coast, it is likely some part of the coastal protection will face winds head-on. Therefore, the value of this factor will always be 1 when this formula is applied in this project.

γ_f The roughness reduction factor is determined by the type of revetment used. Ordinary values for this factor are given in table C.1.

Table C.1 Roughness factors

reduction	Revetment type
1	asphalt/concrete/grass
1	placed blocks
0.6	loose stone

γ_b the "berm" reduction factor is computed as follows, but only if the ehight of the "berm" is located on the still water level:

$\gamma\_b = 1- B/L_{"berm"}$

B The actual width of the "berm" in meters

[35] Technisch rapport golfoploop en golfoverslag bij dijken, TAW, 2002

L The "berm" length, as defined in the figure below. With equal slopes below and above the "berm" this length in meters becomes:

$$L_{berm} = B + 2 * \frac{H_s}{\tan \alpha}$$

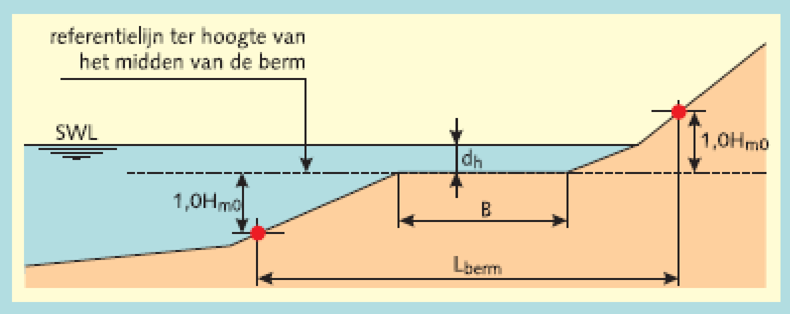

Figure C.1 Definition "berm" length

The wave run-up height has a maximum value of:

$$\frac{z_{2\%}}{H_{m0}} = \gamma_f * \gamma_\beta * \left(4.3 - \frac{1.6}{\sqrt{\xi_0}} \right)$$

This formula has the same input as the first formula, with one small difference: γ_β is defined as: $\gamma_\beta = 1 - 0.0033 * \beta$. This doesn't make much of a difference for this case, however, since the waves are assumed to approach the coast perpendicular.

C.2 Wave overtopping

For the calculation there are two formulas; one for breaking waves ($\gamma_b * \xi < 2$) and one for non-breaking waves ($\gamma_b * \xi > 2$). The formula in the case of breaking waves:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} * \gamma_b * \xi_0 * \exp\left(-4.3 * \frac{h_k}{H_{m0} * \xi_0 * \gamma_b * \gamma_f * \gamma_\beta} \right)$$

with:

h_k	m	free crest height, or Freeboard
H_{mo}	m	significant wave height at toe of the dike
ξ_0	-	Iribarren number
γ_β	-	incidence reduction factor
γ_f	-	roughness reduction factor
γ_b	-	"berm" reduction factor

When the waves stop breaking, the overtopping has reached a maximum. This maximum is calculated as follows:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.2 * \exp\left(-2.3 * \frac{h_k}{H_{m0} * \gamma_f * \gamma_\beta} \right)$$

In case of a shallow foreshore, the first formula will be altered slightly:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = 0.21 * \exp\left(\frac{h_k}{\gamma_f * \gamma_\beta * H_{m0} * (0.33 + 0.022 * \xi_0)}\right)$$

The transition from relatively deep to shallow foreshores is located around breaker index values (ξ) of about 6. The formulas mentioned above are therefore applied to a breaker index of 5, and the formula for shallow foreshore is applied from an index of 7 or higher. Between ξ-values of 5 and 7 linear interpolation is applied.

The way of designing using overtopping-formulae is to calculate a certain freeboard using a maximum allowable overtopping discharge over the dike per running meter of dike. The value of this maximum depends on the quality of the inner slope of the dike. Table C.2 will give maximum discharges for different kinds of inner slopes.

Table C.2 Quality of inner slope with discharge according

Quality of inner slope	Maximum allowable discharge (l/m/s)
Sandy soil with poor quality grass cover	0.1
Sandy clay soil with moderate quality grass cover	1
Clay soil with outer-slope-quality grass-cover or a	10

Other considerations for choosing a low overtopping discharge are the accessibility during the storm or the fact, that during a storm the drainage system on the landward side of the dike already has to cope with a peak discharge. Large amounts of overtopping might lead to a transgression of the drainage capacity and result in flooding. Because accessibility of the dike during storm conditions for pedestrians is not an issue (pedestrians are expected to stay home), but the drainage is an issue, an overtopping discharge of 0.1 l/m/s is used for design.

C.3 Example calculation: Land reclamation dike

The wave conditions at the toe of the dike are as defined in table C.3.

Table C.3 Conditions at land reclamation dike

Symbol	Value	Unit	Definition
Hm0	1.92	m	Significant wave height
Tm-1,0	11.1	m	Spectral wave period
L0	191	m	Deep sea wave length
tan	1/6	-	Tangent slope of dike

From these values the surf similarity parameter or Iribarren-number can be computed:

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{Hs}{L0}}} = 1.66$$

According to the angle of incidence and the applied revetment certain reduction factors are obtained, see table C.4.

Table C.4 Values for reduction factors

Symbol	Value	Unit	Definition
β	0	degrees	Angle of incidence
γ_β	1	-	Incidence factor
γ_β	0.7	-	Roughness factor

The "berm" is located on the still water level, and the slope on either side of it has the same inclination. The "berm" length is therefore simply the "berm" with plus twice the wave height divided by the tangent of the slope. The "berm" reduction factor becomes as defied in table C.5.

Table C.5 Geometry of "berm"

Symbol	Value	Unit	Definition
B	15	m	Berm width
L	38	m	Berm length
b	0.61	-	Berm reduction factor

With these values finally the wave run-up can be calculated:

$$z_{2\%} = 1.75 * 1 * 0.7 * 0.61 * 1.66 * 1.92 = 2.36m$$

With the rules and formulae for wave overtopping and using a maximum overtopping discharge of 0.1 l/s/m, the required freeboard becomes:

$$\frac{0.1}{\sqrt{g * 1.92^3}} = \frac{0.067}{\sqrt{1/8}} * 0.61 * 1.66 * \exp\left(-4.3 * \frac{h_k}{1.92 * 1.66 * 1 * 0.61 * 0.7}\right) \rightarrow h_k = 2.99m$$

As one can see, the freeboard calculated according to overtopping rules is stricter than the height calculated according to wave run-up. This is generally the case when applying these formulae.

Appendix D Design of revetments

D.1 Stability of loose rock under waves

D.1.1 The van der Meer formula

The design of a revetment made out of loose rock under wave attack is done using the Van der Meer formula:

$$\frac{H_{m0}}{\Delta d_{n50}} = 6.2 * P^{0.18} * \left(\frac{S}{\sqrt{N}}\right)^{0.2} * \xi^{-0.5} \qquad \text{(for plunging breakers)}$$

$$\frac{H_{m0}}{\Delta d_{n50}} = 1.0 * P^{-0.13} * \left(\frac{S}{\sqrt{N}}\right)^{0.2} * \xi^{P} * \sqrt{\cot\alpha} \qquad \text{(for surging breakers)}$$

with:

H_{mo}	m	significant wave height
D_{n50}	m	median nominal diameter rock
P	-	permeability factor
S	-	damage level
N	-	number of waves
ξ	-	Iribarren number
α	degrees	slope of structure

The transition between surging and plunging breakers is defined as follows:

$$\xi_{transition} = \left[6.2 * P^{0.31} * \sqrt{\tan\alpha}\right]^{\left(\frac{1}{P+0.5}\right)}$$

When $\xi > \xi_{transmission}$ the formula for surging breakers has to be applied. However, for cotα > 4 surging waves do not exist in practice, and the formula for plunging breakers should be applied, which also gives a conservative value for the stone size.

Comments on symbols and units:

P The value permeability factor will be taken as 0.1, since the dike will have a relatively impermeable sand/clay core.

S The damage level is defined as an erosion area divided by the square of the stone diameter, and as such represents the number of removed stones. A value of 2 or 3 can be taken as the threshold of motion, while a value of 10 represents failure of the structure. Since a typhoon season can spawn multiple typhoons in relatively short time, it has to be possible to repair damage in between storms. Therefore, a value for S of 3 will be used for determining stone-size.

N The number of waves gives an indication of the duration of the storm. Since the storms hitting this coast are relatively long-lasting, a value of

7500 can be taken for N. With this value the damage will have reached a certain equilibrium.

D.1.2 Example calculation: revetment land reclamation dike

The wave conditions at the toe of the dike are as stated in table D.1.

Table D.1 Conditions at land reclamation dike

Symbol	Value	Unit	Definition
Hm0	1.92	m	Significant wave height
Tm-1,0	11.1	m	Spectral wave period
L0	191	m	Deep sea wave length
tan α	1/6	-	Tangent slope of dike

From these values the surf similarity parameter or Iribarren-number can be computed.

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{Hs}{L0}}} = 1.66$$

To compute the stone size the design parameters from table D.2are applied.

Table D.2 Parameters used for calculating stone size

Symbol	Value	Unit	Definition
P	0.1	-	Permeability factor
S	3	-	Damage level
N	7500	-	Number of waves

Using these values the transition breaker parameter can now be determined:

$$\xi_{transition} = \left[6.2 * 0.1^{0.31} * \sqrt{1/6}\right]^{\left(\frac{1}{0.1+0.5}\right)} = 1.43$$

The actual Iribarren-number is higher, so the formula for surging breakers has to be applied. Using this formula goes as follows:

$$d_{n50} = 1.0 * 0.1^{-0.13} * \left(\frac{3}{\sqrt{7500}}\right)^{0.2} * \xi^{0.1} * \sqrt{6} = 0.68m$$

This d_{n50} can be converted into a d_{50} of 0.82 meters.

D.2 Stability of placed blocks under waves

D.2.1 The Pilarczyk formula

The design of a placed block revetment under wave attack is done using the Pilarczyk formulas:

$$\Delta * D = \Psi_u^{-1} * \Phi^{-1} * \cos^{-1}\alpha * H_{mo} * \xi^b$$

with:

Δ	-	relative density rock (usually about 1.6)
Ψ_u	-	stability upgrading factor (Ψ=1 for riprap, Ψ>1 for blocks)
Φ	-	stability factor
α	degrees	slope
H_{mo}	m	significant wave height
ξ	-	Iribarren number
b	-	Iribarren-exponent
D	m	Specific size or thickness protection unit

Comments on symbols and units:

Ψ_u This stability upgrading factor is a system-defined empirical factor, and can actually only be determined using mathematical or scale-models. To be on the safe side a value of 1.5 is used here for block revetments.

Φ Stability function of incipient motion. A safe value of 2.25 is used here.

ξ Iribarren surf similarity parameter. The above formula is only valid for

ξ<=3, but for ξ>3 the sizes calculated at ξ=3 are still valid.

b Exponent related to the interaction between waves and the revetment type. For riprap a value of 0.5 is taken, for block revetments a value of 1.

D.2.2 Example calculation: revetment replacement dike

The wave conditions at the toe of the dike are as follows:

Table D.3 Conditions at replacement dike

Symbol	Value	Unit	Definition
Hm0	0.79	m	Significant wave height
Tm-1,0	11.1	m	Spectral wave period
L0	191	m	Deep sea wave length
tan ɑ	1/9	-	Tangent slope of dike

From these values the surf similarity parameter or Iribarren-number can be computed.

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{Hs}{L0}}} = 1.73$$

To compute the size of the blocks the following design parameters are applied:

Table D.4 Parameters used for calculating block size

Symbol	Value	Unit	Definition
Φ	2.25	-	Stability factor
ψ	1.5	-	Stability upgrading factor
Δ	1.6	-	Relative density rock
b	1	-	Iribarren exponent

Using these parameters the following block size can be computed:

$$D = 1.6^{-1} * 2.25^{-1} * 1.5^{-1} * (\cos 6{,}3)^{-1} * 0.79 * 1.73^{1} = 0.26m$$

D.3 Design of filters underneath a revetment

In this appendix the design for the filter underneath the armor revetment on the land reclamation dike will be described. In the following equations the f-index will always indicate the layer lying on top of the layer, of which properties have to be designed. This lower layer is always indicated with a b-index.

D.3.1 Granular filter layer

The top layer of the loose rock armor (stone class 300-1000) has a D_{f50} of 0.65 m and a D_{f15} of 0.54 m. The minimum size for the D_{b85} of the layer underneath the top layer is the D_{f15} divided by five

$$D_{b85} > D_{f15}/5 \rightarrow D_{b85} > 0.11m \rightarrow D_{b50} > 0.06m$$

which gives a stone class for the filter of 90-180mm.

The transition from outer armor to filter layer has to be checked for sufficient permeability to prevent a build up of water pressure. The rule is that the D_{15} of the upper filter layer should be at least five times bigger than the D_{15} of the bottom filter layer:

$$D_{f15} > 5 * D_{b15}$$

$D_{f15} = 0.54$ $D_{b15} = 0.05 \rightarrow 5*0.05 = 0.25 < 0.54$

As can be seen, the permeability is enough to prevent pressure build-up.

The 90-180 class stones are still too big to lie directly on top of the core material of the dike, so a second filter layer has to be applied. Using the same rules as used above one obtains:

$D_{b85} > D_{f15}/5 \rightarrow D_{b85} > 0.05/5 = 0.01$ m $\rightarrow D_{b50} > 0.005$ m

A filter layer with stones this size can still not be placed directly on the sandy core material of the dike. Therefore, to limit the thickness of this second filter layer, it will not be made out of stones. Instead of several granular filter layers, one layer of geo-textile will be applied.

D.3.2 Geo-textile

The only conditions posed to this geo-textile are that it won't let sand particles pass, and that it has sufficient permeability to prevent pressure build-up. To prevent sand particles from passing though the filter the following condition applies:

$O_{90} < 2 * D_{90}$

In this equation D_{90} is a property of the core material. Since this material is sand, O_{90} will have to be less than 0.6 mm.

To prevent a build-up of water pressure behind the filter, the geo-textile must have a permeability in the same order of magnitude as the core material, which in this case is sand. Therefore, the permeability k of the geo-textile must be about 10^{-3} m/s.

D.4 Design of a toe construction

To determine the stone size for the toe of a revetment two formulas are used: one for relatively deep toes and one for shallow ones.

For relatively deep toes ($h_t/h_m>0.4$) the following equation is valid:

$$\frac{H_s}{\Delta d_{n50}} = 8.7 * \left(\frac{h_t}{h_m}\right)^{1.4}$$

In this equation h_m and h_t are respectively the water depth in front of the dike and the depth above the toe itself, see figure D.1.

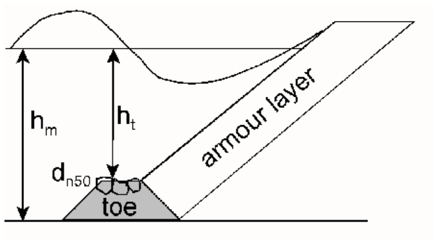

Figure D.1 water depths above toe

For shallow toes ($h_t/h_m<0.4$) the needed stone size can be calculated with:

$$\frac{H_s}{\Delta d_{n50}} = 1.1 * \left(0.24 * \frac{h_t}{d_{n50}} + 1.6\right)$$

The stone size in the armor layer can be seen as a maximum for the stone size calculated by these formulae.

For the toe of the revetment on the dike protecting the land reclamation the water depth in front of the dike, h_m, is 3.83 meters. If one simply buries the toe into the beach, the depth above the toe is simply the depth in front of the dike, and the use of the formula for deep toes is warranted.

Using this formula a stone size D_{n50} of 0.212 meters is calculated, which corresponds to a stone class of 10-60 kg. However, because this toe also has to support the revetment it is better to apply a bigger stone class. A rule of thumb for designing a toe is that the stone class should be no smaller than one class below the class of the revetment. So in this case the toe will consist of stone class 60-300 kg.

Appendix E Overtopping of vertical seawall

According to CEM EM 1110-2-1614, *'Design of coastal revetments, seawalls and bulkheads.'* The formula's below can be used to calculate an average overtopping rate of simple vertical seawalls, with no fronting revetments and a small parapet at the crest. This subject has been researched by Ward & Ahrends, 1992.

$$\frac{Q}{\sqrt{gH_{mo}^2}} = C_o \exp\left(C_1 F' + C_2 \frac{F}{d_s} \right)$$

with:

$$F' = F / \left(H_{mo}^2 L_o \right)^{1/3}$$

with:

Q	l/s/m	average overtopping rate
g	m/s2	gravitational force
H_{mo}	m	significant wave height in front of structure
C_0,C_1,C_2	-	regression coefficients
F'	-	Dimensionless freeboard
F	m	freeboard
d_s	m	water depth at structure toe
L_0	m	spectral wave length

Comments on symbols and units:

C_0,C_1,C_2 For a simple vertical seawall the regression coefficients are defined by:

C_o = 0.338

C_1 = -7.385

C_2 = -2.178

F The freeboard is the distance from design water level, also referred to as still water level (SWL) to the crest of the structure.

Q This is the average overtopping discharge per meter length of the structure. Because this is an average value overtopping by individual waves can be much higher.

At the project area the design wave height is 0.7 m with a spectral length of 191.1 m. The water depth at structure toe is 1.38 m. From the calculation below it follows that under the circumstances of the project area when the freeboard is set at 1.65 m the height of the structure will be sufficient to comply with the overtopping criterion of 0.004 l/s/m.

$C_0 := 0.338$

$g := 9.81$

$C_1 := -7.385$

$H_s := 0.7$

$C_2 := -2.178$

$L_0 := 191.107$

$d_s := 1.38$

$F := 1.65$

$$F' := \frac{F}{\sqrt[3]{\left(H_s^{\,2} \cdot L_0\right)}} \qquad F' = 0.363$$

$$Q' := C_0 \cdot \exp\left(C_1 \cdot F' + C_2 \cdot \frac{F}{d_s}\right) \qquad Q' = 1.708 \times 10^{-3}$$

$$Q := Q' \cdot \sqrt{g \cdot H_s^{\,2}} \qquad Q = 3.746 \times 10^{-3}$$

Appendix F Stability of gravity seawall

Since project area concerned is a seismic active region in the stability calculations earthquakes should be taken in to account. For the calculation below guidelines described in: '*Seismic design guidelines for port structures*', Pianc, 2001, are used.

F.1 Guidelines

F.1.1 Active earth pressures

In the pseudo-static approach, the earth pressures are estimated using the Monobe-Okabe equation (Monobe; Okabe, 1924). This equation is derived by modifying Coulomb's classical earth pressure theory (Coulomb, 1776; Kramer, 1996) to account for inertia forces induced by earthquakes. The body force vector, originally pointed downward due to gravity is rotated by the seismic inertia angle.

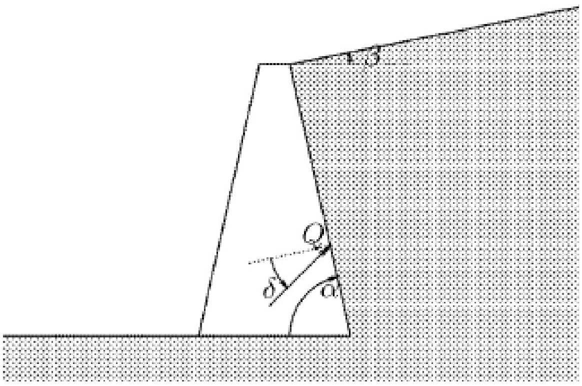

Figure F.1 Layout of gravity wall

For a retaining gravity wall with the layout of figure F.1 the *static* active earth pressure coefficient is given by[36]:

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\sin(\alpha-\delta)\left[1+\sqrt{\{\sin(\phi+\delta)\sin(\phi-\beta)\}/\{\sin(\alpha-\delta)\sin(\alpha+\beta)\}}\right]^2}$$

with:

K_a	-	static active earth pressure coefficient
α	rad	angle back of wall with base
β	rad	slope of backfill
ϕ	rad	internal friction angle of backfill soil
δ	rad	friction angle between backfill and wall

[36] Grondmechanica, Verruit, 2005

Comments on symbols and units:

ϕ Since currently the soil characteristics are not known, an internal friction angle of 30° is used, this is a good approximation for normal loosely packed sand. When the sand is compacted this value will be higher.

δ The friction angle between the backfill and the wall is assumed to be 2/3 of the internal friction angle of the backfill which is in this case 20°.

By rotating the geometry of Coulomb's solution through the seismic inertia angle, ψ, and scaling the magnitude of the body force to fit the resultant of the gravity and the inertia forces the *dynamic* active earth pressure coefficient is obtained. For a wall with a horizontal backfill (β =0) the dynamic active earth pressure coefficient, K_{ae} is given by:

$$K_{ae} := \frac{\sin(\alpha + \phi - \psi)^2}{\sin(\alpha)^2 \cdot \cos(\psi) \cdot \sin(\alpha - \delta - \psi) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \psi)}{\sin(\alpha - \delta - \psi) \cdot \sin(\alpha)}}\right)^2}$$

The seismic inertia angle is given by:

$$\psi := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

with:

ψ	rad	seismic inertia angle
k_h	-	horizontal effective seismic coefficient
k_v	-	vertical effective seismic coefficient

Comments on symbols and units:

k_h After Noda et al, 1975 an average relationship between the effective seismic coefficient k_e and the peak ground acceleration a_{max} is obtained as:

$k_e = 0.6*(a_{max}/g)$. In this calculation as peak ground acceleration the heaviest conceivable earthquake for the region is used (natural boundary condition BC-N8). This gives an a_{max}/g of 0.4, so k_e = 0.6*0.4=0.24.

k_v The vertical effective seismic coefficient is a fraction of the horizontal coefficient. For simplicity reasons in this calculation it is assumed that $k_v = 0$. As is common design practice (e.g. Ministry of Transport, Japan, 1989)

For completely dry soil, the dynamic active earth thrust, P_{ae}, which acts at an angle, δ, from the normal to the back of the gravity type wall of height, H, is given by:

$$P_{ae} := K_{ae} \cdot \frac{1}{2} \cdot \left(\gamma_d + \frac{\frac{1}{2} \cdot q_{sur}}{H}\right) \cdot H^2$$

with:

P_{ae}	kN/m	dynamic active earth thrust
K_{ae}	-	dynamic active earth pressure coefficient
γ_d	kN/m^3	unit weight of dry backfill
q_{sur}	kN/m^2	uniformly distributed surcharge
H	m	height of gravity type wall

Comments on symbols and units:

P_{ae} The point of application of the resultant force is typically chosen at a level of 0.4H to 0.45H (Seed and Whithman, 1970). Since 0.45H gives the highest turning moment in relation to the toe of the structure, 0.45H is used in this calculation.

γ_d As a first approximation a dry unit weight of 16 kN/m^3 is used. This is between the weight of dune sand and river sand.

q_{sur} As a surcharge 15 kN/m^2 is applied, this corresponds to traffic class VK60 in Dutch codes which is medium to heavy traffic. For a promenade this is a conservative value, but since there is be a possibility of sand trucks crossing the promenade and no safety factor for variable load is applied a conservative approach seems justified. Under earthquake conditions half of the surcharge for static design is used.

F.1.2 Hydrodynamic pressure

During seismic shaking, the free water in front of the structure exerts a cyclic dynamic loading on the wall; the critical mode occurs during the phase when suction pressure is applied on the wall. The resultant load can be approximated by (Westergaard, 1933):

$$P_{dw} := \frac{7}{12} \cdot k_h \cdot \gamma_w \cdot H_w^{\ 2}$$

with:

P_{dw}	kN/m	suction pressure load
k_h	-	horizontal effective seismic coefficient
γ_w	kN/m^3	unit weight of seawater
H_w	m	water depth

Comments on symbols and units:

P_{dw} The point of application of the resultant force lies at 0.4H above mudline.

γ_w The unit weight of seawater is 10.2 kN/m^3.

H_w For the water depth the design water level of 4.83m +EL is used. This gives a water depth of 1.33 m in front of the structure. This is again a conservative approach, since this high water table is not likely to occur in combination with an earthquake of this magnitude.

F.1.3 Safety factors

For a structure with a given geometry a force and moment balance check can be made. The forces to be taken into account are the dynamic active earth thrust, the hydrodynamic force, the inertia forces and the vertical forces. When the balance is made up the factor of safety against overturning and the factor of safety against sliding can be determined. The factor of safety against overturning, F_{so}, is calculated by comparing the stabilizing and overturning moments:

$$F_{so} = M_{stabilizing}/M_{overturning}$$

The factor of safety against sliding, F_{ss}, is calculated by comparing the horizontal forces and the friction forces between the structure and the foundation:

$$F_{ss} = F_{ver} * \mu_b / F_{hor}.$$

The factor μ_b is the friction coefficient. In this calculation it is taken 0.5, which is a conservative value for concrete-rubble friction according to the tables in USACE Coastal Engineering Manual, Part VI, Chapter 5.

F.2 Calculation

The geometry of a gravity seawall with a vertical front is determined by the height, the base over height ratio, b/H, and the angle α of the back of the wall with horizontal. Using a spreadsheet the stability of the seawall is calculated according to the above formulas. This is done for a number of combinations of b/H and α. In table F.1 below the results are condensed.

Table F.1 Cross section and safety factors for different geometries of seawall.

alpha b/H	50 0.87	55 0.96	60 1.05	65 1.13	70 1.22	75 1.31	80 1.40	85 1.48	90 1.57	deg rad
0.50	***	***	***	3.65 0.95 0.90	4.35 0.98 0.87	5.01 1.00 0.86	5.64 1.02 0.85	6.25 1.04 0.85	6.85 1.06 0.85	
0.55	***	***	***	4.34 1.10 0.93	5.04 1.12 0.91	5.70 1.14 0.89	6.32 1.15 0.88	6.93 1.17 0.88	7.53 1.20 0.89	
0.60	***	***	4.26 1.23 1.00	5.02 1.25 0.97	5.72 1.27 0.94	6.38 1.28 0.93	7.01 1.29 0.92	7.62 1.31 0.92	8.21 1.33 0.93	
0.65	***	***	4.95 1.38 1.03	5.71 1.40 1.00	6.41 1.41 0.97	7.06 1.42 0.96	7.69 1.43 0.95	8.30 1.45 0.95	8.90 1.47 0.96	
0.70	***	***	5.63 1.54 1.06	6.39 1.55 1.02	7.09 1.56 1.00	7.75 1.56 0.99	8.38 1.57 0.98	8.98 1.59 0.98	9.58 1.61 0.99	
0.75	***	5.47 1.69 1.13	6.32 1.69 1.08	7.08 1.70 1.05	7.78 1.70 1.03	8.43 1.70 1.02	9.06 1.71 1.01	9.67 1.73 1.01	10.27 1.75 1.02	
0.80	***	6.16 1.85 1.16	7.00 1.85 1.11	7.76 1.84 1.08	8.46 1.84 1.06	9.12 1.84 1.04	9.75 1.85 1.04	10.35 1.86 1.04	10.95 1.89 1.05	
0.85	5.89 2.01 1.24	6.84 2.01 1.18	7.68 2.00 1.13	8.44 1.99 1.10	9.15 1.98 1.08	9.80 1.98 1.07	10.43 1.99 1.06	11.04 2.00 1.07	11.64 2.00 1.07	
0.90	6.58 2.18 1.26	7.53 2.16 1.20	8.37 2.15 1.15	9.13 2.13 1.12	9.83 2.12 1.10	10.49 2.12 1.09	11.11 2.12 1.09	11.72 2.14 1.09	12.32 2.16 1.10	
0.95	7.26 2.34 1.28	8.21 2.32 1.21	9.05 2.29 1.17	9.81 2.28 1.14	10.51 2.26 1.12	11.17 2.26 1.11	11.80 2.26 1.11	12.41 2.27 1.11	13.01 2.29 1.12	
1.00	7.95 2.50 1.29	8.90 2.47 1.23	9.74 2.44 1.19	10.50 2.42 1.16	11.20 2.40 1.14	11.86 2.39 1.13	12.48 2.39 1.13	13.09 2.40 1.13	13.69 2.42 1.14	

Cell entry:

cross section A [m2]	
Safety factor overturning	**Safety factor sliding**

***	combination of alpha and b/H not aplicable under given H
	sliding and overturning factor below 1
	sliding factor below 1
	sliding and overturning factor above 1

A configuration of b/H=0.65 and α=60° is chosen. The geometry of this configuration is depicted in figure F.2 below. For the sake of completeness, the calculation of this configuration is written out on the next page.

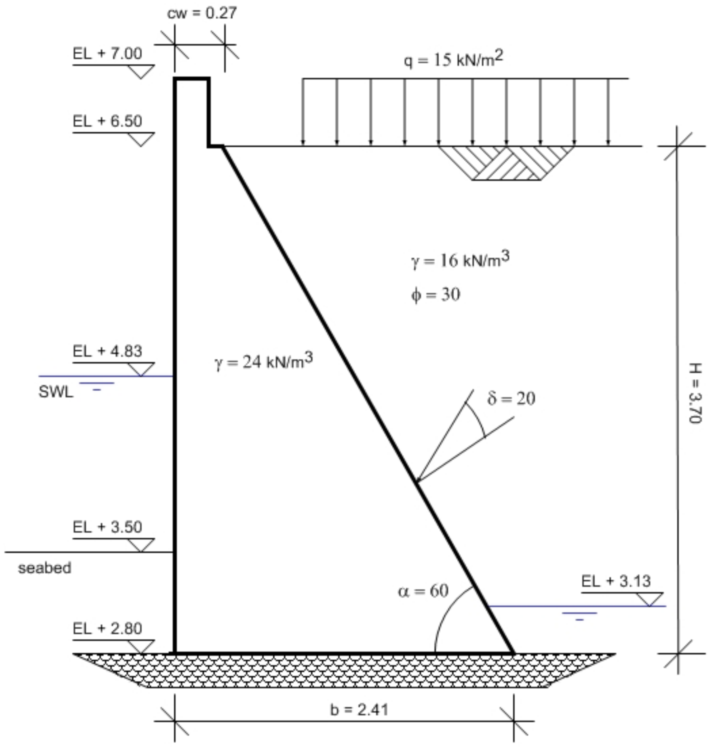

Figure F.2 Cross section of seawall

F.2.2 Active earth pressure

First the dynamic active earth pressure coefficient is calculated. Then the dynamic active earth thrust is determined.

$k_h := 0.24 \qquad k_v := 0$

$\psi := \operatorname{atan}\left(\frac{k_h}{1-k_v}\right) \rightarrow .23554498072086334143 \qquad \psi_{deg} := \frac{\psi}{2\cdot\pi}\cdot 360 \qquad \psi_{deg} = 13.496$

$\phi_{deg} := 30 \qquad \phi := \frac{\phi_{deg}}{360}\cdot 2\cdot\pi \qquad \phi = 0.524 \qquad \alpha_{deg} := 60 \qquad \alpha := \frac{\alpha_{deg}}{360}\cdot 2\cdot\pi \qquad \alpha = 1.047$

$\delta_{deg} := 20 \qquad \delta := \frac{\delta_{deg}}{360}\cdot 2\cdot\pi \qquad \delta = 0.349$

$$K_{ae} := \frac{\sin(\alpha+\phi-\psi)^2}{\sin(\alpha)^2\cdot\cos(\psi)\cdot\sin(\alpha-\delta-\psi)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\psi)}{\sin(\alpha-\delta-\psi)\cdot\sin(\alpha)}}\right)^2}$$

$K_{ae} = 0.948$

$\gamma_d := 16 \qquad q_{sur} := 15 \qquad H := 3.7$

$P_{ae} := K_{ae}\cdot\frac{1}{2}\cdot\left(\gamma_d + \frac{\frac{1}{2}\cdot q_{sur}}{H}\right)\cdot H^2 \qquad P_{ae} = 117.007$

This dynamic active earth thrust acts at an angle δ to the normal of the back of the wall, so it has can be divided in a horizontal contribution by:

$$P_{aehor} := P_{ae} \cdot \sin(\alpha - \delta) \qquad P_{aehor} = 75.21$$

Which acts at a distance of 0.45*H = 0.45*3.7=1.67 m above structure base.

The vertical contribution is given by:

$$P_{aever} := P_{ae} \cdot \cos(\alpha - \delta) \qquad P_{aever} = 89.632$$

and acts at the interaction between the wall and the backfill.

The passive active earth pressure induced on the structure by the soil of the seabed is not applied in this calculation because of the scour potential of the soil just in front of the structure. Again this is a conservative approach.

F.2.3 Hydrodynamic force

The hydrodynamic force in front of the wall gives a horizontal suction force of:

$$\gamma_w := 10.2 \qquad H_w := 1.33$$

$$P_{dw} := \frac{7}{12} \cdot k_h \cdot \gamma_w \cdot H_w^{\,2} \qquad P_{dw} = 2.526$$

This force acts at 0.4*1.33=0.53 m above sea bed.

F.2.4 Inertia forces

The inertia forces acting on the concrete structure are calculated by:

$$A := 4.946 \qquad \gamma_c := 24$$

$$P_{in.wall} := k_h \cdot A \cdot \gamma_c \qquad P_{in.wall} = 28.489$$

In which A is the cross section surface in m^2, and γ_c is the unit weight of the concrete in kN/m^3. This force acts at 0.5*3.7=1.85m above structure base.

The inertia forces acting on the backfill are calculated by:

$$H := 3.7 \qquad b := 0.65 \cdot H \qquad b = 2.405$$

$$P_{in.backfill} := (H \cdot b - A) \cdot \gamma_d \cdot k_h \qquad P_{in.backfill} = 15.178$$

This force acts at 2/3*3.7=2.47m above structure base.

F.2.5 Vertical forces

The gravitational force acting on the concrete wall is given by:

G_{wall} = A*γ_c = 4.946*24=118.9 kN/m

To determine the point of application the structure's cross section is divided in a rectangular part A1 and a triangular part A2. The horizontal distance from the toe to the point of application is calculated by:

$$\frac{A1 \cdot \frac{1}{2} \cdot c_w + A2 \cdot \frac{1}{3} \cdot (b - c_w)}{A1 + A2}$$

In which c_w is the crest width of the structure and b is the base width of the structure. The distance from the toe to the point of application derived in this way is: 0.81m.

The gravitational force acting on the backfill is calculated similar and is given by: $G_{backfill}$ = (H*b-A)*γ_d = (3.7*2.405-4.946)*16 = 63.23 kN/m and acts at a distance of cw+2/3*(b-c_w) = 0.27+2/3*(2.41-0.27) = 1.69 m

Since the groundwater table is only 33 cm above the structure base the horizontal force of the hydrostatic pressure of the groundwater is neglected.

The uplift force is calculated by:

$$\mathrm{SWL} := 4.83 \quad +\mathrm{EL} \qquad \mathrm{groundw} := 3.13 \quad +\mathrm{EL}$$

$$P_{uplift} := 0.5\cdot(\mathrm{SWL} - \mathrm{groundw})\cdot b\cdot\gamma_w \qquad P_{uplift} = 20.851$$

This force acts at a distance of 1/3*2.41=0.8m from structure toe.

F.2.6 Conclusion

The calculated forces acting on the structure are depicted in figure F.3 below.

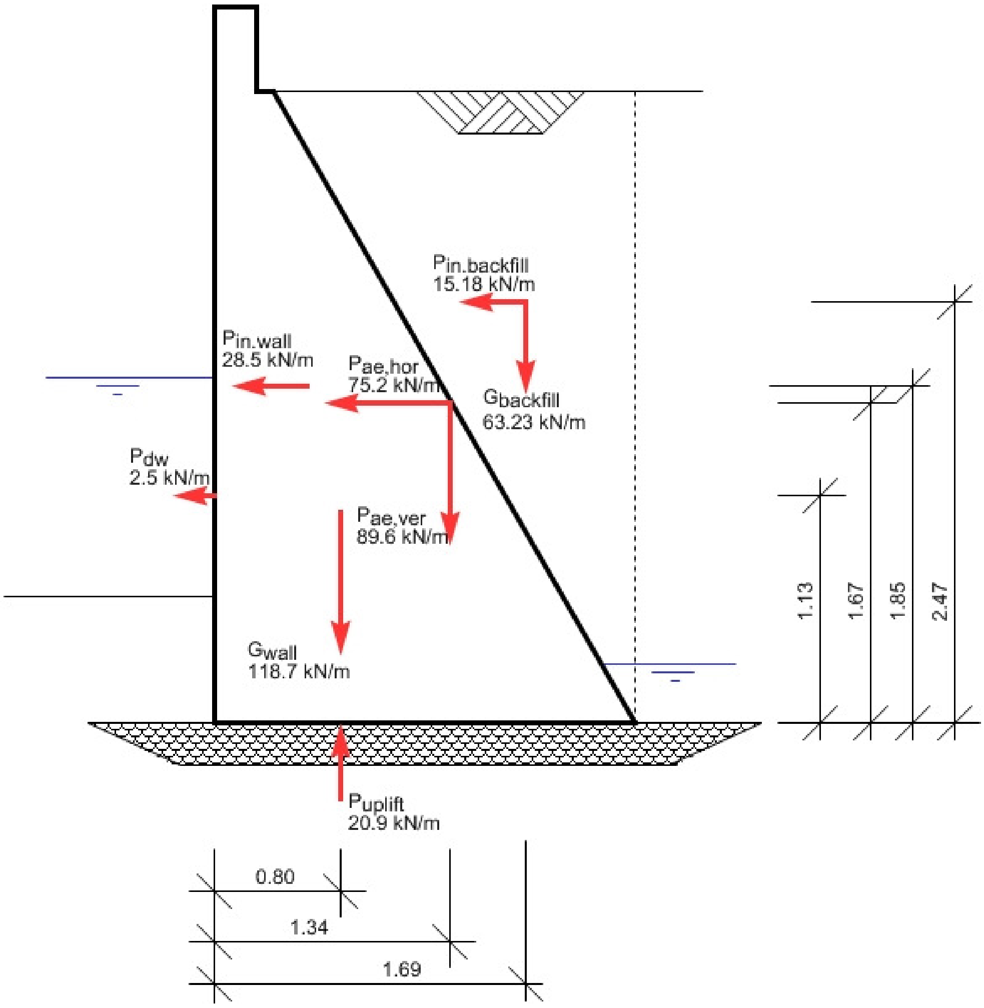

Figure F.3 Forces acting on structure

In the table below, the forces and moment balance is made up

Table F.2 Force and moment balance

	P [kN/m]	distance to toe [m]	moment [kNm/m]
Stabilizing forces			
$P_{ae,ver}$	89.63	1.34	119.83
G_{wall}	118.72	0.81	96.24
$G_{backfill}$	63.23	1.69	107.05
total:	271.58		323.12
resultant vertical force:	250.73	(total - P_{uplift})	
Overturning forces			
$P_{ae,hor}$	75.21	1.67	125.23
P_{dw}	2.53	1.13	2.85
$P_{in.wall}$	28.49	1.85	52.71
$P_{in.backfill}$	15.18	2.47	37.43
P_{uplift}	20.85	0.8	16.72
total:	142.26		234.94
resultant horizontal force:	121.4	(total – P_{uplift})	

The safety factor for overturning is thus given by:

F_{so} = 323.12/234.94 = 1.38

The safety factor for sliding is given by:

F_{ss} = 250.73*0.5/121.40 = 1.03

Appendix G **Model data**

The cross-profiles of the rivers are determined during a site survey; photos have been taken at several places along the river. The model is calibrated in such a way that in the current situation there is no flooding of the area. This is not very accurate, but its goal is to get an estimation of the run-off in the area and the needed drainage if land-reclamation is applied and for this purpose it is considered accurate enough.

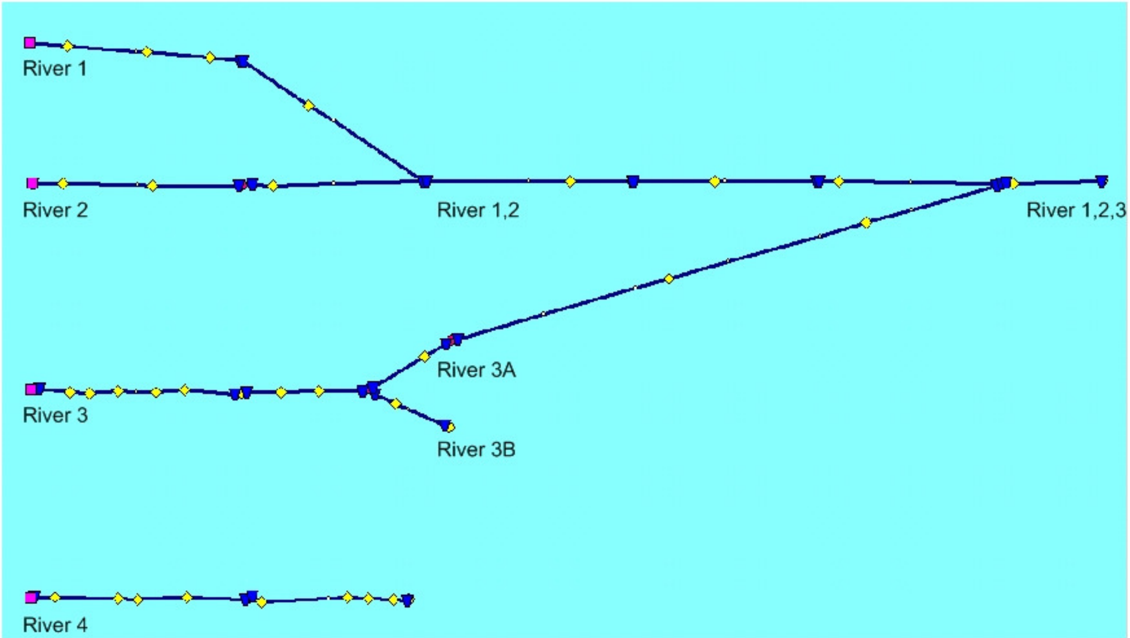

Figure G.1 Model in Sobek and their representative names

G.2 Cross-profiles

The cross-profiles are determined by a site survey and are listed in table G.2.

Table G.2 The cross-profiles of the different rivers

Name	Branch	Form	Bed witdth	Name	Branch	Form	Bed width
River 1	Normal	Rectangle	3	River 3	Normal	Rectangle	3
	Outlet	Rectangle	4		A	Rectangle	2.5
River 2	Normal	Rectangle	3		B	Rectangle	1.5
	Outlet	Rectangle	5	River 4	Normal	Rectangle	2.5
	1+2	Rectangle	2.5		Upstream	Rectangle	1.5
	1+2 upstream	Rectangle	2				

G.3 Determination run-off

The run-off is determined from the rainfall data presented in Appendix A. From this rain-fall data run-off curves are calculated for paved and unpaved areas. By using the map as in figure G.2 the ratio is determined for the areas along the rivers. In table G.2 to table G.6 the results from this process are given, the names of the rivers in the Sobek model are give in figure G.1.

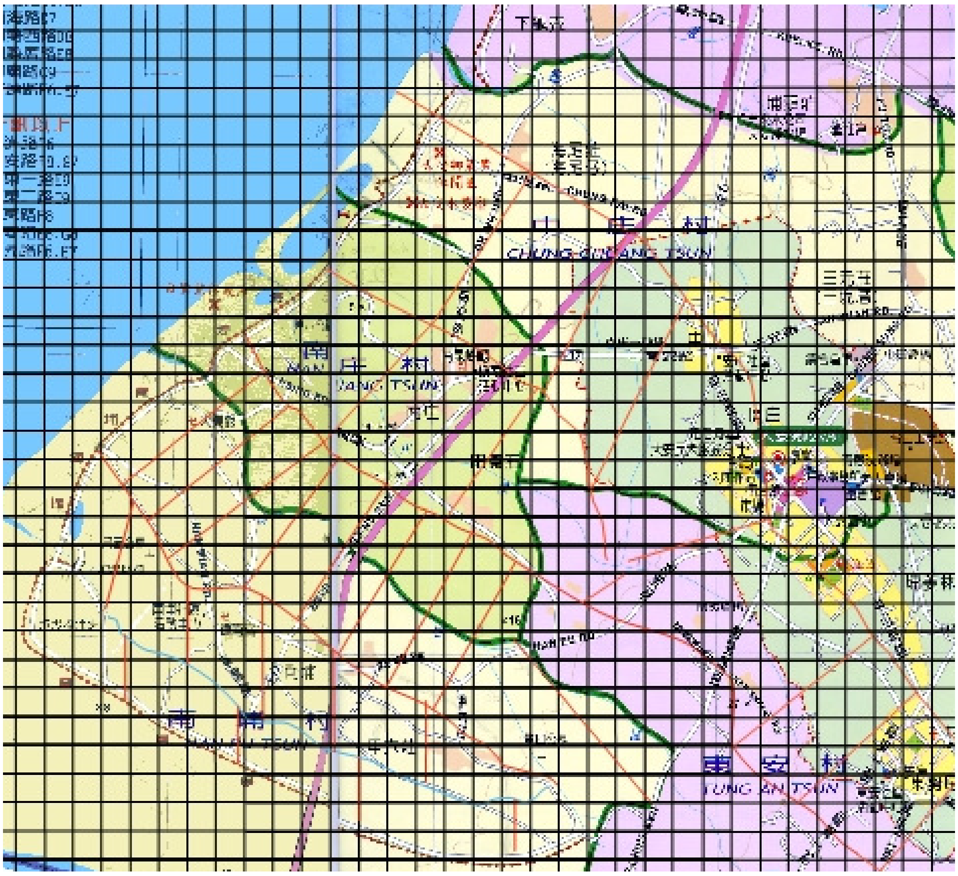

Figure G.2 Map with grid to determine the ratio between paved and unpaved areas

Table G.3 Paved and unpaved ratio for River 1

Branch				
Inflow at L = [km]	180	550	850	1400
Area [ha]	27	27	19	34
Side	L+R	L+R	L+R	L+R
% paved	60	60	60	30
% unpaved	40	40	40	70

Table G.4 Paved and unpaved ratio for River 2 and combined rivers 1+2 and 1+2+3

Branch				1+2	1+2	1+2	1+2	1+2	1+2+3
Inflow at L = [km]	150	600	1200	0	700	1000	1400	2000	50
Area [ha]	20	27	21	36	15	18	15	9	5
Side	L+R	L+R	L+R	L+R	L+R	L+R	L+R	L+R	L+R
% paved	70	50	55	60	70	20	60	70	70
% unpaved	30	50	45	40	30	80	40	30	30

Table G.5 Paved and unpaved ratio for River 3

Branch								
Inflow at L = [km]	200	300	450	650	800	1100	1300	1500
Area [ha]	10	10	10	10	10	10	10	10
Side	L+R	L+R	L+R	L+R	L+R	L+R	L+R	L+R
% paved	70	60	60	60	50	60	60	60
% unpaved	30	40	40	40	50	40	40	40

Table G.6 Paved and unpaved ratio for River 3, branches A and B

Branch	A	A	A	B
Inflow at L = [km]	1750	3300	3890	1750
Area [ha]	32	10	18	21
Side	L+R	L+R	L+R	L+R
% paved	70	60	70	60
% unpaved	30	40	30	40

Table G.7 Paved and unpaved ratio for River 4

Inflow at L = [km]	120	545	1120	1620	2000	120	445	795	1200	1750	1900
Area [ha]	10	10	10	10	10	10	10	10	10	10	10
Side	Left	Left	Left	Left	Left	Right	Right	Right	Right	Right	Right
% paved	60	60	60	60	60	80	65	15	50	60	50
% unpaved	40	40	40	40	40	20	35	85	50	40	50

Appendix H Calculation spillway

For the calculation a broad crested weir is assumed and because there is free flow after the weir the maximum discharge is achieved when there is critical flow so if the Froude number is equal to one.

$$Fr = \frac{u}{\sqrt{gh}} = 1$$

the critical velocity becomes:

$$u = \sqrt{gh}$$

If is assumed that the accelerating flow will give no extra energy dissipation one can use the Bernoulli formula:

$$H = z + \frac{pg}{\rho} + \frac{u^2}{2g}$$

A cross section of the weir is given in figure H.1.

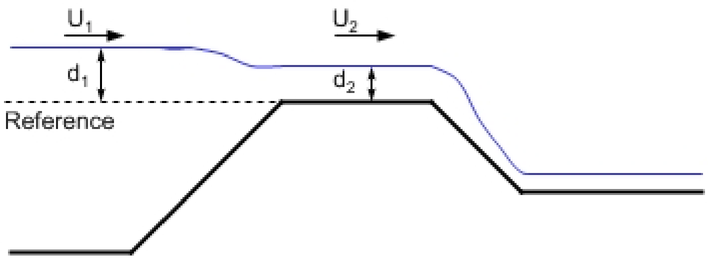

Figure H.1 Cross section weir

Using the Bernoulli formula in this figure the following is obtained (1):

$$h_1 + \frac{u_1^2}{2g} = h_2 + \frac{u_2^2}{2g}$$

The velocity towards the weir is zero and for u_2 the critical velocity has to be applied, so:

$$h_1 = h_2 + \frac{h_2}{2}$$

and thus

$$h_2 = \frac{2}{3}h_1$$

This filled in (1) gives:

$$h_1 + \frac{2}{3}h_1 + \frac{u_2^2}{2g}$$

and thus (2)

$$u_2 = \sqrt{\frac{1}{3} h_1 * 2g}$$

And for the discharge per meter width the following is obtained (3):

$$Q = u * h = \frac{2}{3} h_1 * \sqrt{\frac{1}{3} h_1 * 2g} = \frac{2}{3\sqrt{3}} h_1^{1.5} * \sqrt{2g}$$

In the empirical formula two coefficients are used, namely c_v, a correction coefficient for approaching velocity and c_d, a coefficient for the weir shape.

Because the approaching velocity is equal to zero this coefficient becomes one. The coefficient for the weir shape also becomes one if the weir is long enough and rounded.

In this situation the water height h_1 is 0.5 meter. From formulae (2) and (3) the discharge and the velocity can be determined:

Discharge	Q = 0.6 m^3/s/m
Velocity	u_2 = 1.8 m/s

Appendix I Wave height in detention areas

When detention areas are in use, a rather big water surface exists, thus wind induced waves are going to develop. The development of these waves is depending on the wind speeds, the water depth and the duration of the storm. The interaction between the developed waves and the embankment, i.e. the wave run-up can be calculated from the wave height and period.

I.1 Wave height

The development of the wave height according to the wind speed and fetch length is given in table I.1. The water depth of the detention area is at maximum 2 meters. The wave heights given in the table are given by a depth of two meters, the value of the significant wave height will be on the safe side.

Table I.1 Wave heights according to the fetch length and wind speed, depth is 2 meter

Fetch length	Windspeeds at 10 meter height				
	5	10	15	20	25
250	0.06	0.13	0.21	0.29	0.36
500	0.08	0.17	0.27	0.36	0.45
1000	0.11	0.22	0.34	0.45	0.54
2000	0.14	0.28	0.41	0.52	0.62

The effective fetch length F is determined by the weighted average of the projection $R(\alpha_w)$ on the wind direction in all directions of α_w. A sector of 45º is used on both sides, see also figure I.1. The effective fetch length is determined by the formula

$$F_e = \frac{\Sigma R(\alpha_w)\cos^2\alpha_w}{\Sigma \cos\alpha_w}$$

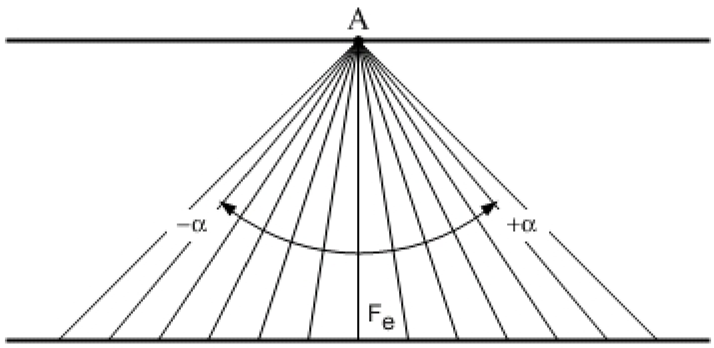

Figure I.1 Example of the determination of the effective fetch length

The effective fetch length follows from table I.2 and is 250 meter. The design wind speed of a typhoon is 50 meters per second, which is more than the maximum value of table I.1. By using the program “Cress” the significant wave height and the wave period are determined, these are respectively 0.65 meter and 2.3 seconds if the method of Bretschei is used. This wave height can be used to determine the wave run-up on the embankments.

Table I.2 Calculation of the fetch length

α_w in degrees	$\cos\alpha_w$	$\cos^2\alpha_w$	$R(\alpha_w)$ in meters	$R(\alpha_w)\cos^2\alpha_w$
-45	0.71	0.50	353.55	176.78
-42	0.74	0.55	336.41	185.79
-36	0.81	0.65	309.02	202.25
-30	0.87	0.75	288.68	216.51
-24	0.91	0.83	273.66	228.39
-18	0.95	0.90	262.87	237.76
-12	0.98	0.96	255.59	244.54
-6	0.99	0.99	251.38	248.63
0	1.00	1.00	250.00	250.00
6	0.99	0.99	251.38	248.63
12	0.98	0.96	255.59	244.54
18	0.95	0.90	262.87	237.76
24	0.91	0.83	273.66	228.39
30	0.87	0.75	288.68	216.51
36	0.81	0.65	309.02	202.25
42	0.74	0.55	336.41	185.79
45	0.71	0.50	353.55	176.78
$\Sigma\cos\alpha_w =$	14.93		$\Sigma R(\alpha_w)\cos^2\alpha_w =$	3731.28

I.2 Wave run-up on the dikes

The wave run-up on the embankments depends on the Iribarren parameter, which itself depends on the slope of the embankment and the height of the incoming waves. The run-up can be calculated with

$R = 1.75\xi H_i r_i$ if $\xi < 2.5$

$R = 3.5 H_i r_i$ if $\xi \geq 2.5$

These formulas are derived for regular waves; if they're applied to wind waves the results will be less accurate. The Iribarren parameter is defined as

$$\xi = 1.25\frac{T}{H_i}\tan\alpha$$

An adapted formula is used in the program "Cress". This results in a wave run-up of 1.39 meter when no reduction is applied. Depending on the roughness of the slope there can be a reduction applied from 0.6 for rubble mount to 0.9 for smooth slopes of placed stones.

Appendix J Calculation stiffeners

The stiffeners are calculated in the ultimate limit state, i.e. with the most unfavorable load combination possible. In this case the most unfavorable load combination is the combined hydrostatic and wave pressure on the sea side, while there is no water on the reclaimed area side.

The maximum hydrostatic pressure is calculated with the formula

$$p = \rho g h$$

The maximum wave pressure is calculated with the formula

$$p = \rho \frac{H}{2} \frac{\cosh(2\pi(d-z)L)}{\cosh(2\pi dL)}$$

The pressures are used to calculate the maximum bending moment in the gate. The resulting stresses in the material should be lower then the maximum allowable stress of 235 N/mm^2. To optimize the distance between the stiffeners an Excel-sheet is used. The optimized result can be found in table J.1.

Table J.1 Optimized calculation for distance between the stiffeners

Input

Global variables		
Ro, water	1025	kg/m3
Gravity	9.81	m/s2
Bathymetry; reference level EL		
Top outlet structure	1.82	meter
Bottom outlet structure	-0.86	meter
Bottom in front of outlet sctructure	-1.86	meter
Water levels		
HHW	3.23	meter
wind set-up	0	meter
wave-height	1.115	meter
Dimensions gate		
width	5	meter
height	3	meter

Calculation

Pressure ont the valve		
Hydrostatic pressure	26.94807	kN/m2
Wave pressure	5.531117	kN/m2
Structure		
Space between IPE200	300	mm
Point of mass	80.76923	mm
Iy of combined profile	38903151	mm^4
Moment	30.44924	kNm
Stress	141.4869	N/mm^2
With safety factor	212.2303	N/mm^2

Appendix K **Drawings**