A new generation flood defences: Dam with tidal power station including pumping capacity

A feasibility and optimisation study

Masters Thesis

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Horvat & Partners

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PREFACE

This report is a thesis, written to obtain the master Hydraulic Engineering at the TU Delft. While this thesis was written during strange times, a pandemic, I look back on the process with fond memories.

The thesis was carried out in cooperation with Horvat and Partners. I would like to express my gratitude to Horvat and Partners for presenting me with this fascinating topic and facilitating my research. I especially want to thank Orson Tieleman, member of my graduation committee, who guided me through this process. He read all versions of this report, even before I scrapped all non-essential parts, and provided me with sharp comments and insights along the way.

I also want to thank Bas Jonkman, chair of my graduation committee, for his valuable feedback on my report, and critical questions and out of the box ideas during the progress meetings. I want to thank Robert Lanzafame, member of my thesis committee, for his enthusiasm (especially about the San Francisco case), guidance and insightful questions during our meetings. I want to thank Antonio Jarquin Laguna for his valuable feedback during the meetings, and sharing his expertise on tidal energy from which I could benefit. I am very appreciative for all the time the committee members have spent to help me forward.

Finally, I would like to thank my parents, or as they like to be called: the first and second reviewers. I will not go into detail on who is who. I also want to thank Luka, Emilie, Floortje, the beast, and my roommates. They provided me with great support along the way.

Rotterdam, December 2020

Alexandra Rijnink

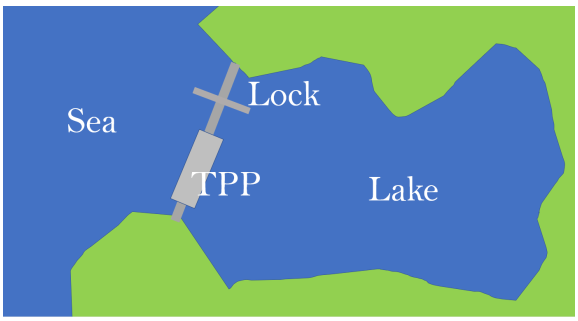
SUMMARY

Introduction

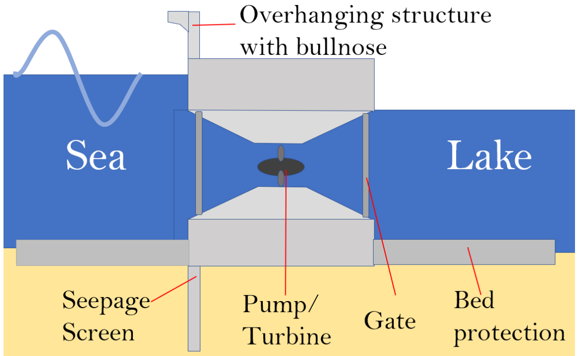
The Brouwersdam is part of the Delta works and closes of the Grevelingenlake in the Netherlands. After its closure in 1971 the Grevelingenlake transformed from an estuary to a salt water lake without tide. Since then, the water quality in the lake has deteriorated. As a sustainable measure to improve water quality the Dutch government intends to bring back a damped tide to the Grevelingenlake. The improvement of water quality will contribute to a healthy ecosystem, recreational possibilities and economic activity.

Introducing a (damped) tide to the Grevelingenlake can be realized by creating large culverts in the Brouwersdam. When the sea level rises this means that the effectiveness of the culverts decreases. The window for water to leave the basin (during low tide at sea) becomes progressively smaller with increasing sea level rise until the stage where the water is not able to leave at all. A more climate robust alternative is to integrate the culvert with a tidal power station, where the turbines can also function as pumps. The pumps can aid the outflow of water from the lake to sea when the sea level has risen

Figure 1a shows the concept of the dam with tidal power station with pumping capacity. A dam is used to close of a tidal estuary creating an artificial lake. The tidal power station with pumps is where the water can move through the barrier. A lock is included to allow ships to pass. Figure 1b shows a side view of a powerhouse in the power station with pumping capacity. Important elements are the bed protection, the gates and the pump/turbine.



(a) Overview dam tidal power station with pumping capacity



(b) Side view powerhouse tidal power station

Figure 1: Dam with tidal powerstation with pumping capacity

The proposed tidal power plant with pump capacity at the Brouwersdam represents a new generation of flood defences: an ecologically friendly dam. When a coastal area is at risk of flooding, a dam with tidal power plant including pumping capacity can be implemented as means of protection. Different from the traditional dam the effects on ecology are less disruptive as a tidal stroke is maintained in the basin for a long period. The dam is a potentially economically and ecologically more desirable solution than currently implemented measures such as heightening dikes and other coastal protection structures around the perimeter of a basin.

The export potential of the solution as considered at the Brouwersdam could potentially be large. Deltares has performed a study identifying 461 potential locations where the solution could be implemented. The study had a very broad scope as the goal was to inventory all possible locations where the solution could potentially be implemented. Different sea level rise scenarios, technical and economical feasibility have not been taken into account in this assessment. The solution also has not been compared to other alternatives such as updating coastal defences already in place. This thesis is focused on getting more insight on whether this export potential truly exists, by answering the question: Under which circumstances does a tidal power station with pump capacity become relevant and economically feasible given the uncertainty of sea level rise?

Methods

To answer this question a few locations with potential are analysed in more detail. These were selected from the dataset created by Deltares. Eight locations with high flood exposure and a large population were chosen. At these locations increased flood protection will most likely be needed as sea level rises.

One of the selected locations was the Brouwersdam, as this provides a good reference case. For the Brouwersdam a design for a tidal power station with pumps and the requirements on the minimum tidal stroke have already been researched. The minimum tidal stroke is a crucial element in the solution, as this is what ensures a standard of water quality. For the other locations in the selection a minimum tidal stroke was not yet established. The solution has the potential to not only maintain good water quality but also the intertidal character of a basin, if the stroke at the basin is kept close to the original tidal stroke. A large tidal stroke at the lake comes paired with large projects costs. Two scenarios were considered for the required tidal stroke at the basin after closure: an economic scenario in which the tidal stroke may be reduced to the point where it is just sufficient to refresh water in the deeper parts of the basin (5 - 25 % of the original stroke depending on basin bathymetry), and a nature preservation scenario in which at least 80% of the currently present tidal stroke is maintained.

To analyse the energy yield, pumping requirements, tidal stroke and lifetime of the tidal power station with pumping capacity, a hydro-energetic cost model was developed. The model makes use of a two sided energy generation scheme. This means that energy is generated both during high tide and low tide. Water flows from sea to the lake when the water level at sea is higher than at the lake, and vice versa. The water passes the turbines and energy is generated. As sea level rises pumps come into play, they aid the outflow of water from the lake to sea. The model was validated against the results of a previous study done on the Brouwersdam. The results of the hydro energetic cost model were in good agreement with previously found results, It was found that the minimum tidal stroke at the Grevelingenlake could be maintained for 42 centimeters of sea level rise with the original design. This is more than was calculated in previous studies (28 centimeters) where pumps were only used in flushing mode. Pumps in the new model are able to pump both in flushing mode and against a head difference.

A second model was developed to generate optimal designs for all locations. The model is able to generate a basic design at the lowest costs for a any climate scenario and design lifetime. The model finds an optimal combination of the required number of pump/turbines, their diameter, the design head, the starting head and stopping head difference (n, D, H_{rated}, H_{start}, H_{stop}). The first three being properties of the pump turbine, and the latter two are used to determine when to open and close the gates in order to regulate the water levels at the lake and generate energy. Through a parameter influence study a targeted optimization routine was constructed. The model was validated against the design of the Brouwersdam from the previous study. The resulting design was close to the original both in found flow area and total costs. The resulting design showed some improvements over the original design. The tidal stroke can be maintained up to 0.5 meters sea level rise and with a 9% less negative net present value.

Results and conclusions

After generating optimal designs for every location, design lifetime, climate scenario and water level management strategy of interest, the results could be analysed. It was found that the number and diameter of the pump/turbines were most influential on the energy yield, tidal stroke and costs of the structure. The number and diameter of the pump/turbines result in a certain flow area through which the water is able to pass the barrier. A minimum flow area is needed to facilitate the desired tidal stroke at the basin at zero meters sea level rise. Without using pumps the energy yield and tidal stroke at the lake reduces as sea level rises. It was found that the number of pumps used for the minimum flow area is able to maintain the tidal stroke up to approximately 0.5 meters sea level rise. Additional pumps are needed above this minimum in order to maintain the tidal stroke for a longer period and more sea level rise.

While this new solution has the potential to 'pay for itself' as energy can be generated by the tidal power station, this study has shown that this is not the case. It was found that primarily the uniformity of the tide and secondarily the amplitude of the tides were indicators of the return on investment. A large daily inequality in the tidal amplitude negatively affects the energy yield and requires more energy for pumping. The highest return on investment (55 %) was found for Asan Bay in South Korea, a location with a uniform tide with and a mean tidal range of 6 meters, and the lowest return on investment (-30%) was found for San Francisco Bay, a location with a non uniform tide with and a mean tidal range of 1.7 meters. The investment is thus only partially returned at best.

While the solution is not a positive business case in itself, the study has shown that the solution can be the most economically attractive alternative to protect a coastal zone against flooding in an ecologically friendly manner, compared to updating coastal defences or a dam with simple culvert. The two most important considerations are the expected sea level rise and the desired tidal stroke in the basin. The results for all business cases for increasing sea level rises are shown in figure 2.

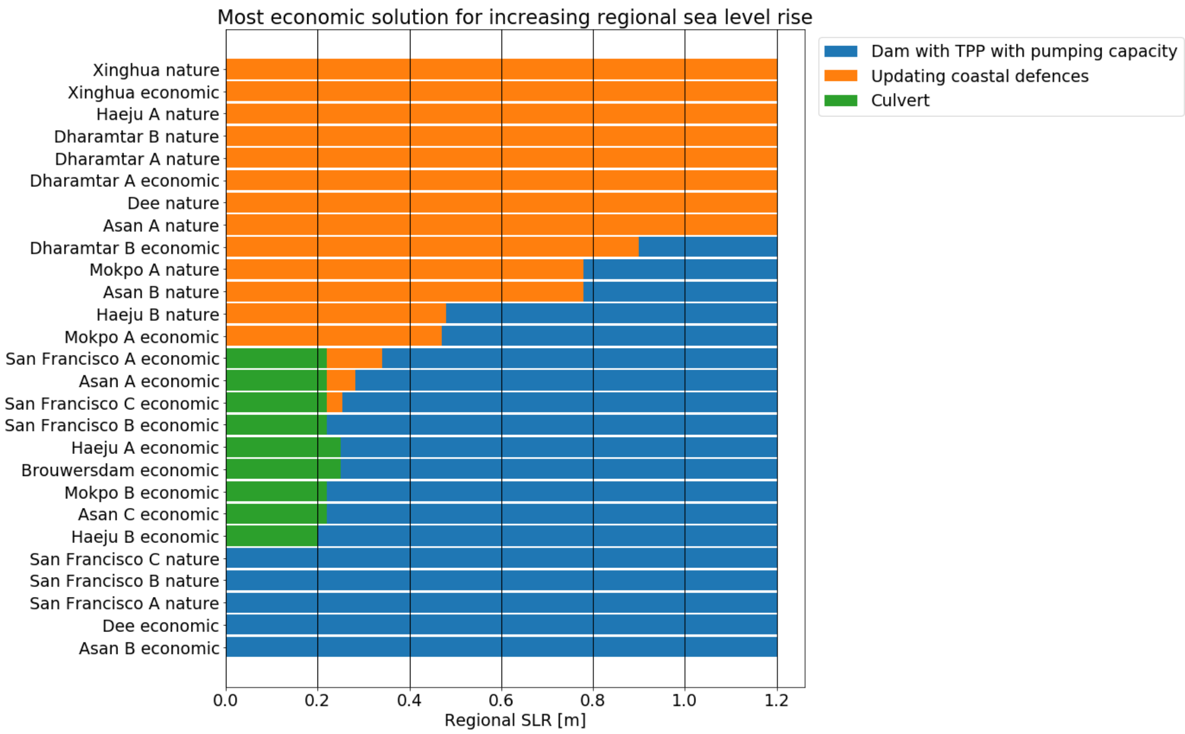


Figure 2: Results business cases for increasing regional sea level rise

Table 1 summarizes this figure in words by listing the best alternative for the cases assessed in the study.

Table 1: Best coastal defence alternative for sea level rise and tidal stroke

	Strongly reduced stroke (5 - 25% original)	**Mildly reduced stroke (80% original)**
Limited SLR (0 -25 cm)	Mixed: Dam with culvert or upgrading dikes and floodwalls	Upgrading dikes and floodwalls
Mild SLR (25 - 60 cm)	Dam with tidal power plant and pumps	Upgrading dikes and floodwalls
Severe SLR (60 - 100 cm)	Dam with tidal power plant and pumps	Dam with tidal power plant and pumps

For limited sea level rise and a strong reduction in the tide both the dam and updating dikes and floodwalls are suitable alternatives. In this situation pumps are not needed to maintain the tidal stroke, making a dam with culvert the cheaper dam option. For limited sea level rise and the original tidal stroke still mostly in tact, the most cost efficient alternative is to update the coastal defences. For mild sea level rise and and a strong reduction in the tide the dam with tidal power plant and pumps becomes the most attractive alternative. Culverts are no longer sufficient to maintain the tidal stroke over the lifetime of the structure, so some pumps are needed, but a relatively small station suffices. To maintain a larger tidal stroke for this amount of sea level rise a large station is needed, this is a very cost intensive element of the dam, in which case it is more cost efficient to adapt the existing coastal defenses. Only for the most severe climate scenarios does the investment in a large tidal power station with pumps become favourable for a large tidal stroke compared to the option to update coastal defences.

These conclusions are valid for most basins included in the analysis, however there are some basin characteristics that may lead to a different most suitable solution for a certain tidal stroke and sea level rise. High construction costs are in favour of a dam, where low construction costs are in favour of updating coastal defences. The ratio between the perimeter of the basin and the closure gap width and depth can also be of influence. If the perimeter is very large compared to the closure gap, the dam with TPP and pumps will be the favoured alternative already for small amounts of sea level rise. A large closure gap compared to basin perimeter results in the opposite: updating the coastal defences will be the preferred solution also for large amounts of sea level rise.

A more detailed case study of the San Francisco Bay supported the findings of the business cases. The case study identified added benefits of the dam such as transportation that contributed to the desirability of the alternative. A dam with tidal power station is a feasible solution for the San Francisco South Bay.

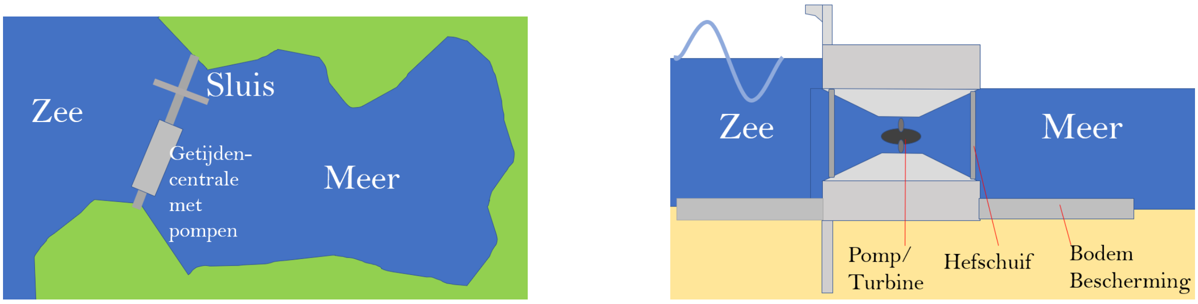
The method developed in this study was used to analyse the feasibility of a dam with tidal power station and pumping capacity for 8 potential locations. By extrapolation of the obtained results an estimation of the total amount of potential locations can be made.The study has shown that the export potential of the solution is significantly lower than initially estimated. The number of locations is dependent on the climate scenario and timescale that are used. On the short term (40 years) the number of export locations is 10 - 20 locations. On the long term (100 years) the number of location ranges from 10 - 60 locations. This is a large reduction on the 461 locations initially estimated in the Deltares study.

SAMENVATTING

Introductie

De Brouwersdam is een onderdeel van de Deltawerken en sluit het Grevelingenmeer af van de zee. Na het voltooien van de Brouwersdam transformeerde het Grevelingemeer van een estuarium naar een zoutwatermeer. Sindsdien is de waterkwaliteit sterk afgenomen. De Nederlandse overheid heeft het voornemen om dit te verbeteren door middel van de herintroductie van een gedempt getij op het Grevelingenmeer.

Afbeelding 3a laat het concept van de voorgestelde oplossing zien. Een dam sluit een estuarium af, waardoor een kunstmatig meer wordt gecreëerd. Een deel van de dam bestaat uit de getijdencentrale met pompcapaciteit. Een sluis kan worden toegepast om de passage van schepen te faciliteren. In afbeelding 3b is een zijaanzicht van een machinekamer in de centrale getoond. De belangrijkste elementen van de constructie zijn de bodembescherming, de pompturbine en de hefschuiven.



(a) Overzichtsschets

(b) Zijaanzicht machinekamer getijdencentrale met pompcapaciteit

Figure 3: Getijdencentrale met pompcapaciteit

Het concept van een dam met getijdencentrale en pompcapaciteit representeert een nieuwe generatie dam. Wanneer er in een estuarium een overstromingsrisico bestaat, kan een dam met centrale worden toegepast als bescherming. Het voordeel van een dam met getijdencentrale met pompen is dat de effecten op het ecosysteem in het afgesloten bassin minder nadelig zijn ten opzichte van de traditionele dam. Dit komt doordat de getijslag blijft bestaan. Ook wanneer de zeespiegel stijgt kan de getijslag in stand worden gehouden door middel van pompen. Dit nieuwe type dam is potentieel economisch en zelfs ecologisch aantrekkelijker dan veel gebruikte alternatieven, zoals het versterken van dijken en andere waterkerende structuren rondom het estuarium.

Het exportpotentieel van de oplossing is mogelijkerwijs groot. Een verkennende studie van Deltares identificeerde eerder 461 locaties waar de oplossingsrichting van toepassing zou kunnen zijn.Verschillende klimaatscenario's alsmede de technische en economische haalbaarheid per locatie zijn in het onderzoek buiten beschouwing gelaten. Ook is de oplossing niet vergeleken met gangbare alternatieven zoals het versterken van bestaande kustverdediging.In dit afstudeeronderzoek wordt het exportpotentieel verder uitgewerkt door ook deze aspecten te beschouwen. De centrale onderzoeksvraag luidt: "Onder welke omstandigheden is een getijdencentrale met pompcapaciteit praktisch en economisch haalbaar, gegeven de onzekerheid van zeespiegelstijging?"

Methode

Hiervoor worden enkele van deze locaties met exportpotentieel nader geanalyseerd. Uit de dataset zijn acht locaties geselecteerd met een hoog overstromingsrisico, met zowel een hoge overstromingsfrequentie als een hoge bevolkingsdichtheid. Op deze locaties is hoogstwaarschijnlijk meer bescherming tegen overstromingen nodig naarmate de zeespiegel stijgt. De Brouwersdam is geselecteerd als referentielocatie. Voor de Brouwersdam is al onderzoek gedaan naar een ontwerp voor een getijdencentrale met pompen en zijn ook de eisen aan de minimale getijslag vastgesteld. De minimale getijslag is een cruciaal element in de oplossing, omdat dit de waterkwaliteit waarborgt. Voor de overige locaties in de selectie is nog geen minimale getijslag vastgesteld. De dam met getijdencentrale met pompcapaciteit heeft het potentieel om niet alleen een goede waterkwaliteit in het meer te behouden, maar ook het intergetijden karakter. Dit laatste wordt deels behouden wanneer de getijslag in het bassin dichtbij de oorspronkelijke getijslag ligt. Een grote getijslag gaat echter gepaard met grote projectkosten. Twee peilbeheersingsscenario's zijn in deze studie verder geëvalueerd: een economisch scenario waarin de getijslag kan worden teruggebracht tot het punt waarop het net voldoende is om water te verversen in de diepere delen van het bekken (5 - 25 % van de oorspronkelijke getijslag, afhankelijk van het diepteprofiel in het bassin), en een natuurgericht scenario waarbij zoveel mogelijk natuur wordt behouden doordat minimaal 80% van de huidige getijslag wordt gehandhaafd.

Om de energieopbrengst, pompbehoefte, getijslag op het meer en levensduur van de getijdencentrale met pompcapaciteit te analyseren, is een hydro-energetisch kostenmodel ontwikkeld. Het model maakt gebruik van een tweezijdige energie generatieschema. Dit betekent dat er zowel bij eb als vloed energie wordt opgewekt. Het model is gevalideerd aan de hand van de resultaten van een eerdere studie naar de Brouwersdam. De resultaten van het hydro-energetisch kostenmodel kwamen goed overeen met deze eerder gevonden resultaten. Het bleek dat de minimale getijslag op het Grevelingenmeer met het bestaande ontwerp tot aan 42 cm zeespiegelstijging gehandhaafd kon worden. Dit is meer dan de 28 cm die werd berekend in het eerdere onderzoek. In het eerdere onderzoek werd alleen uitgegaan van spuiend pompen terwijl in het nieuwe model zowel spuiend als tegen verval gepompt kan worden.

Een tweede model is ontwikkeld om voor alle locaties een optimaal ontwerp te bepalen. Het model is in staat om een ontwerp te genereren voor elk klimaatscenario en ontwerp levensduur tegen de laagste kosten. Het model zoekt een optimale combinatie van het vereiste aantal pompturbines, hun diameter, het ontwerpverval, het startverval en het stopverval. De eerste drie zijn eigenschappen van de pompturbine, en de laatste twee worden gebruikt om het waterpeil te reguleren. Het model is gevalideerd aan de hand van het ontwerp voor de Brouwersdam uit de vorige studies. Het gevonden ontwerp kwam grotendeels overeen met het origineel, zowel wat betreft het doorstroom oppervlak als de totale project kosten. Het gevonden ontwerp vertoonde enkele verbeteringen: De getijslag kan worden gehandhaafd tot 0.5 meter zeespiegelstijging en met een 9 % minder negatieve netto contante waarde.

Resultaten en conclusies

Na het genereren van een optimaal ontwerp voor elke relevante locatie, ontwerplevensduur, klimaatscenario en peilbeheerscenario's, konden de resultaten worden geanalyseerd. Het aantal en de diameter van de pompturbines hadden de meeste invloed op de energieopbrengst, de getijslag en de kosten van de het project. Een minimaal doorstroomoppervlak is nodig om de gewenste getijslag op het meer bij nul meter zeespiegelstijging mogelijk te maken. Tot 0.5 m zeespiegelstijging blijkt de pompcapaciteit behorend by het basis doorstroomoppervlak effectief in het behoud van de getijslag. Voorbij dit punt is veelal extra pompcapaciteit nodig.

Hoewel deze nieuwe oplossing het potentieel heeft om "voor zichzelf te betalen" met de opgewekte duurzame energie, is aangetoond dat dit niet het geval is. Het bleek dat voornamelijk het onderlinge verschil in amplitude tussen twee opeenvolgende getijcycli van belang waren en op de tweede plaats de grootte van de amplitude van de getijslag. Onder ideale omstandigheden werd maximaal 55% van de investering terug verdiend.

Hoewel de geïsoleerde oplossing geen positieve business case oplevert, heeft de studie aangetoond dat de oplossing toch toepasbaar is. Wanneer de oplossing wordt vergeleken met het alternatief om bestaande kustverdediging te versterken of met een dam met doorlaat, is deze in bepaalde omstandigheden economisch het meest aantrekkelijk. De twee belangrijkste parameters zijn de verwachte zeespiegelstijging en de gewenste getijslag in het bassin. Figuur 4 toont de resultaten voor alle business cases voor toenemende zeespiegelstijging.

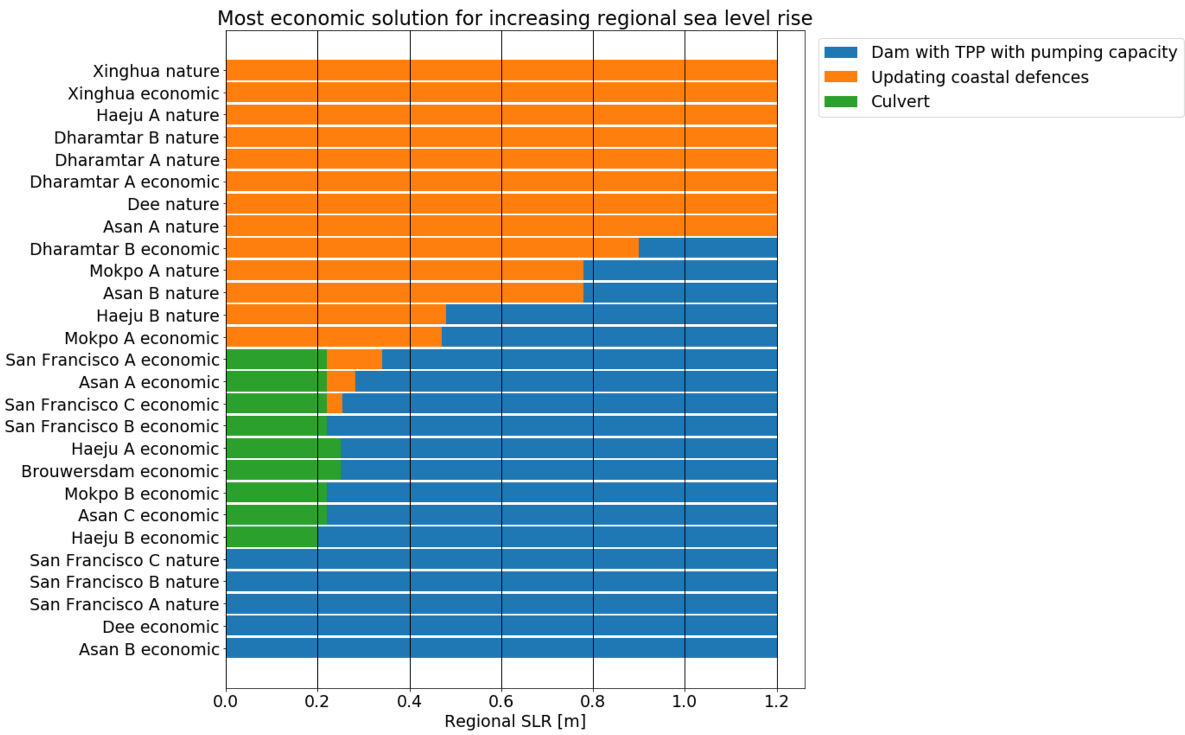


Figure 4: Results business cases for increasing regional sea level rise

Tabel 2 vat deze resultaten samen in woorden voor de beschouwde peilbeheersings en klimaat scenario's.

Table 2: Economisch meest aantrekkelijke kustverdedigingsstrategie

	Sterk gereduceerd getij (5 - 25% origineel)	**Mild gereduceerd getij (80% origineel)**
Weinig ZSS* (0 -25 cm)	Gemengd: Dam met doorlaat of versterking bestaande kustverdediging	Versterking bestaande kustverdediging
Gematigd ZSS (25 - 60 cm)	Dam met getijdencentrale inclusief pompcapaciteit	Versterking bestaande kustverdediging
Veel ZSS (60 - 100 cm)	Dam met getijdencentrale inclusief pompcapaciteit	Dam met getijdencentrale inclusief pompcapaciteit

*ZSS: zeespiegelstijging

Bij een beperkte zeespiegelstijging (0-0.25 meter) en een sterke reductie van het getij zijn zowel de dam als het versterken van bestaand kustbescherming geschikte alternatieven. In deze situatie zijn pompen in de dam nog niet nodig om de getijslag in stand te houden. Een dam met doorlaat volstaat en is goedkoper dan een dam met getijdencentrale met pompen. Om voor een zelfde hoeveelheid zeespiegelstijging een getijslag dicht bij de oorspronkelijke getijslag te bewerkstelligen, is het meest kostenefficiënte alternatief het versterken van de kustverdediging rondom het bassin.

Voor een gematigde zeespiegelstijging (0.25 m -0.6 m) en een sterke reductie van het getij wordt de dam met getijdencentrale en pompen het meest aantrekkelijke alternatief. Pompen zijn nodig nodig om de getijslag in stand te houden, maar een relatief kleine centrale voldoet. Om bij deze zeespiegelstijging een grotere getijslag te behouden is een grotere centrale nodig. Dit veroorzaakt een forse toename in de kosten van de oplossing, waardoor het versterken van de bestaande kustbescherming een goedkoper alternatief is om hetzelfde effect te bereiken. Het behouden van een grote getijslag op het meer door middel van een dam met getijdencentrale en pompen is alleen in de meest pessimistische klimaatscenario's een economisch wenselijk alternatief.

Deze conclusies zijn geldig voor de meeste locaties die in de analyse zijn opgenomen, maar er zijn enkele locatiespecifieke omstandigheden die kunnen leiden tot een afwijking van bovengenoemde resultaten. Hoge bouwkosten zullen ertoe leiden dat al bij minder zeespiegelstijging een dam economisch wenselijk wordt. Lage bouwkosten zijn in het voordeel van het versterken van de kustverdediging. Ook de verhouding tussen de omtrek van het bassin en de breedte en diepte van de monding van het bassin kunnen van belang zijn. Als de omtrek erg groot is in vergelijking met de monding, zal de dam met getijdencentrale en pompen al sneller de beste oplossing zijn voor kleine hoeveelheden zeespiegelstijging. Een tegenovergestelde verhouding zal ertoe leiden dat het versterken van de kustverdediging ook bij grote hoeveelheden zeespiegelstijging de geprefereerde oplossing is.

De resultaten van de meer gedetailleerde case study van de Baai van San Francisco waren in overeenstemming met de resultaten van de business case. De analyse toonde aan dat een dam nog verdere toegevoegde waarde heeft zoals op het gebied van transport. Een dam met getijdencentrale en pompcapaciteit is een haalbare oplossing voor de Zuid Baai van San Francisco.

Uit het onderzoek is gebleken dat het exportpotentieel van de oplossing aanzienlijk kleiner is dan aanvankelijk ingeschat. Waar Deltares 461 potentiële locaties schatte, concludeert deze studie dat dit aantal hoogstens rond de 60 locaties ligt, afhankelijk van het klimaatscenario en de tijdschaal waarop de oplossing beschouwd wordt.

CONTENTS

CHAPTER
ONE

INTRODUCTION

1.1. Evolution of the Brouwersdam project

The Brouwersdam is part of the Delta works and closes of the Grevelingenlake in the Netherlands. After its closure in 1971 the Grevelingenlake transformed from an estuary to a salt water lake without tide. Since then the water quality in the lake has deteriorated. In the summer months hypoxic zones have appeared in the deeper parts of the lake. This has visible consequences on the ecosystem and organisms that live at the bottom of the lake are harmed by this. Furthermore the poor water quality has consequences for the recreational possibilities at the lake, and the area becomes less attractive for economical developments. For these reasons it is desired to improve the water quality at the Grevelingenlake.

As a sustainable measure to improve water quality the Dutch government intends to bring back a damped tide to the Grevelingenlake. The improvement of water quality will contribute to a healthy ecosystem, recreational possibilities and economic activity.

Figure 1.1: Artist impression of tidal power station at the Brouwersdam. From 'Omroep Zeeland'

There are some requirements that need to be fulfilled when reintroducing the tide. The water level has to vary around a fixed mean water level. This is because the Grevelingenlake is located in a Natura2000 area and thus strict regulation apply to protect the nature in that area. When the water level in the lake would increase by too much this can damage the above water nature and can also lead to high mitigation costs to adjust the infrastructure.

Furthermore a minimum tidal stroke needs to be maintained at the lake. This minimum tidal stroke ensures that the water is sufficiently circulated so that also the deep parts of the basin are refreshed, preventing hypoxic zones from forming. The minimum tidal stroke is dependent on the basin geometry and has been established through models.

Introducing a (damped) tide to the Grevelingenlake can be realized by creating large culverts in the Brouwersdam. When the sea level rises this means that the effectiveness of the culverts decreases. The window for water to leave the basin (during low tide at sea) becomes progressively smaller with increasing sea level rise until the stage where the water is not able to leave at all. Consequently the average tidal stroke is reduced and the measure has a lifetime that is limited by sea level rise.

An interesting opportunity arises here to integrate the inlet with a tidal power station, where the turbines can also function as pumps. While the Brouwersdam is not the most evident location for a tidal power plant because of the relatively small tidal amplitude, it becomes interesting when taking into account sea level rise. By installing the tidal turbines with combined pump function, renewable energy can be generated while regulating the tide at the Grevelingenlake. Water can be pumped out of the lake when the head difference becomes too small to realize the desired tidal amplitude naturally. This is illustrated in figure 1.2. The orange line is the water level at the lake without pumps. During some tidal cycles the basin is not emptied to the intended minimum water level. This is caused by a decrease in the time window where the head difference allows water to flow out to sea. The green line is the water level at the lake with pumps, where the basin is emptied during every tidal cycle due to the forcing of the pump. The usage of the turbines will thus change in time while the main function, i.e. regulating the tide at the Grevelingenlake is always ensured. This way a smaller inlet can suffice for a longer duration.

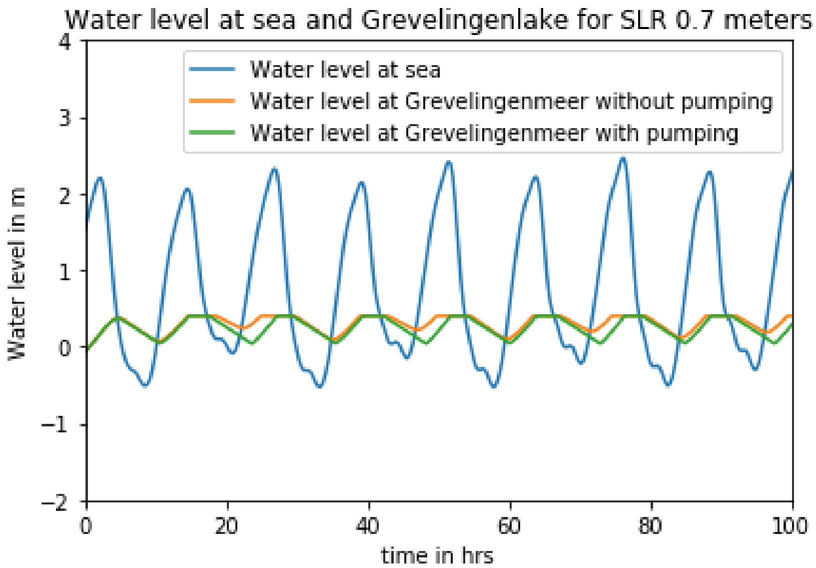


Figure 1.2: Illustration functioning pumping system to regulate water level at Grevelingenlake

The tidal power plant with pump capacity at the Brouwersdam represents a new type of solution for areas that are exposed to flood risk. When an intervention is needed such as the closure of a basin, the tidal power plant with pump capacity can be integrated in a dam to maintain a certain standard of water quality while generating green energy. Flood protection is the primary function which is provided by the dam, maintaining the water quality is the secondary function which is performed by the pump/turbines and sustainable energy generation is the tertiary function also performed by the turbines. If the tidal power plant is built in the Brouwersdam, this solution will have been applied in phases: first the closure in 1971 and later the power plant to improve water quality. At other locations this can also be applied simultaneously to provide an integral solution.

A tidal power plant with pump function has export potential. With climate change developments and the inherent sea level rise, dealing with water will become ever more important. The Brouwersdam represents a new type of solution to deal with sea level rise where closure of a basin is needed. The project could serve as a test case and Dutch knowledge can be used on similar sites around the globe. Deltares has done an exploratory study of possible sites worldwide and their potential yield and profitability to Dutch companies and the GDP. As a first estimation this export potential amounts to 678 million euros. (Deltares (2019)). This gives added value to the project of the Brouwersdam if a tidal power station with pump capacity were to be included.

1.2. Problem description

The previous study about the export potential of the Brouwersdam by Deltares had a very broad scope where the goal was to inventory all possible locations where the solution could potentially be implemented. Different sea level rise scenarios, technical and economical feasibility have not been taken into account in this assessment. The solution also has not been compared to other alternatives such as updating coastal defences already in place.

While the solution seems like a promising new type of ecologically friendly and sustainable dam, it is unclear whether the the application is as widespread as the previous study indicates.

1.3. Objective

To establish if the dam with tidal power station with pump capacity is an effective and economically feasible solution for flood prone coastal areas more insight is needed about relevant parameters in this consideration. Identifying these parameters and mapping their influence and importance will be a powerful diagnostic tool for the identification of 'export locations' as previously described by Deltares. This study investigates the feasibility of a tidal power station with pump capacity for the Brouwersdam and potential export locations.

The main research question is as follows:

Under which circumstances does a dam with tidal turbine station with pump capacity become relevant and economically feasible means of flood protection given the uncertainty of sea level rise?

To answer this question several sub questions have been posed that will together answer the main research question.

1. When does a site have potential for a tidal turbine station with pump capacity i.e. when is it effective?
2. Based on these criteria, what are the most promising sites?
3. When is a tidal turbine/pump station economically feasible ?
   (a) What physical and economical parameters are relevant for this consideration?
   (b) How does the dam with tidal powerstation including puming capacity compare to other flood protection measures?

1.4. Problem approach

To answer this main research question a phased approach, starting with a broad scope and later zooming in, is used. The phases and level of detail is illustrated in figure 1.3. In the figure it is also indicated what purpose each step of the problem approach has.

1.4.1. Literature Study

A literature study will be performed to make an inventory of components that are relevant for the feasibility analysis of the solution. Through a series of case studies of the Brouwersdam and the two largest tidal power stations insight on important components is obtained. Components under investigation are tidal power, generation schemes, sea level rise, flood risk, and economic evaluation methods. More elements are added as the literature study progresses. Insight is desired in all relevant parameters and the result of this literature study will thus not be a list of parameters but rather an overview with background information included. Insights gained during the literature study are used to define the focus of the solution (i.e. energy generation

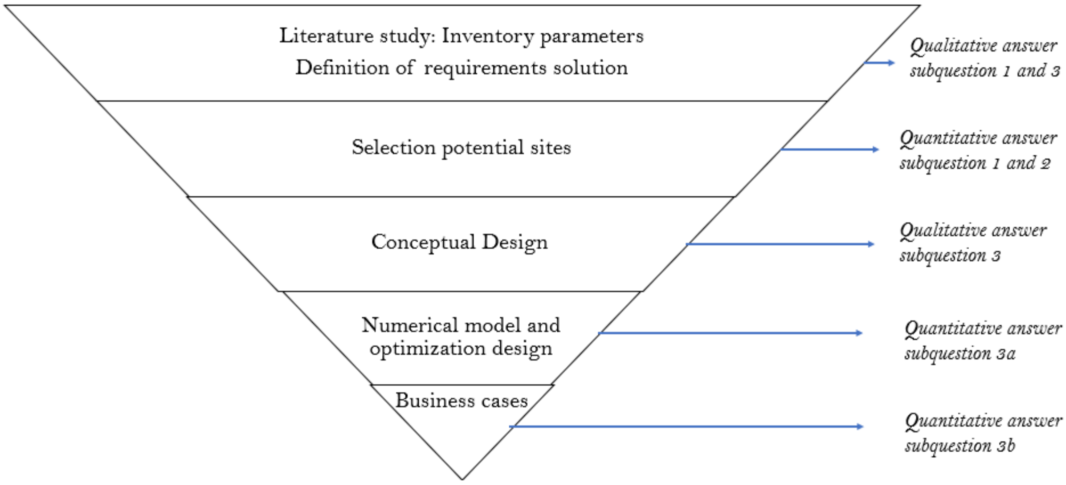


Figure 1.3: Problem approach with narrowing scope

or flood protection) and the program of requirements. As a result of the literature study a qualitative answer is be found for *sub question 1 and 3.*

1.4.2. Selection potential sites

The sites to be analysed for their export potential will be selected in this step. This entails a filtering of the initial selection as made by Deltares. First the Deltares study is evaluated to establish to what extent their selection can be used in this study. A targeted selection approach is synthesized that filters locations with the right properties. The remaining locations are visually inspected through satellite imagery. The result is a small number of potential sites. This will give a quantative answer to *sub question 1 and 2*

1.4.3. Conceptual design

In this step a conceptual design is made for a tidal power station with pumping capacity. The building blocks of the solution are identified, and rules of thumb are provided where necessary for dimensions. The conceptual design can be used as input for the numerical model. This step is used as a tool to answer subquestion 3.

1.4.4. Hydro energetic cost model

Next, a numerical model is set up in Python to investigate the economic feasibility under different rates of sea level rise. This model is used to calculate the energy yield, average annual tidal stroke at the basin, costs and lifetime of a design for different rates of sea level rise. This model is then used to optimize the design for each sea level rise scenario so that the project costs are low while the lifetime is long. This is done by first doing a sensitivity study of the design parameters to establish which parameters are the most influential. These are then varied within several iterations to yield the optimal design for each location for each sea level rise scenario. The results for each optimal design, i.e. the energy yield, average annual tidal stroke at the basin, costs and lifetime can be used in the business case which establishes the economic feasibility of a location. The model thus provides the tools to answer *sub question 3a.*

1.4.5. Business cases

In this step the results from the model described in the section above are transformed into a business case for every location. A positive business case is one where the investment is returned and added value is created. All requirements of the solution have to be met.

The solution is compared to two alternatives to protect against coastal flooding: Updating existing coastal defences and a dam with simple culvert. All three solutions are a means of protecting against flood where disadvantageous effects on ecology are minimized. The comparison can thus be done based purely on economical considerations. This step quantitatively answers *subquestion 3b*.

1.5. Readers Guide

Based on this approach this thesis is divided into five parts based on different steps in the research. These are summarized in figure 1.4 with the corresponding chapters indicated below each part.

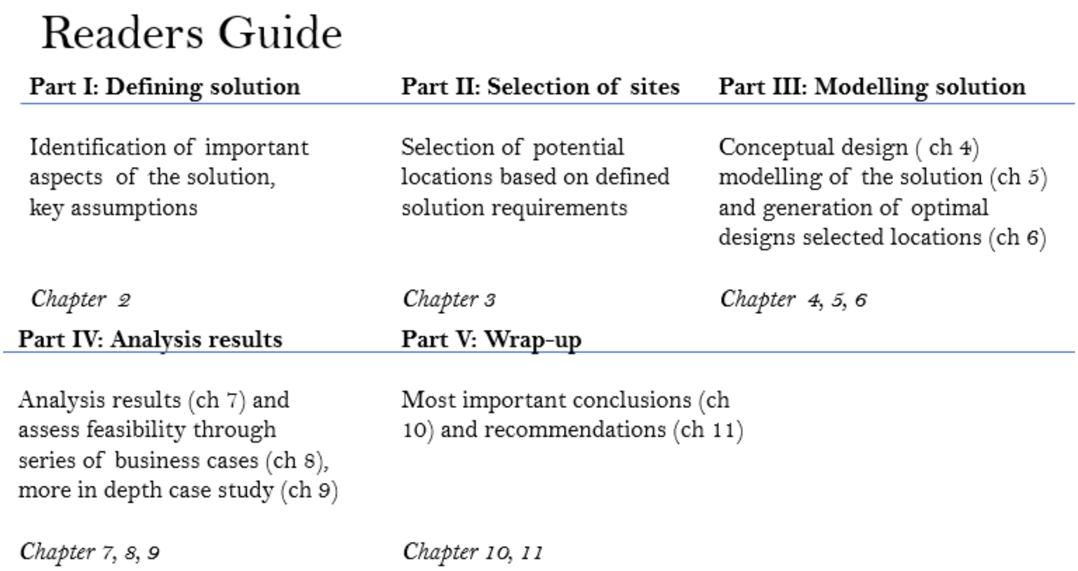


Figure 1.4: Readers guide to thesis

CHAPTER
TWO

KEY STARTING POINTS AND ASSUMPTIONS

The project of reintroducing the tide in the Grevelingenlake has led to a new solution type: a tidal power station where the turbines can be used as pumps to empty the basin when sea level rises. The solution has many functions, the dam serves for flood protection, the pumps ensure a good water quality in a basin and the turbines generate energy. Depending on which element is given most importance, the suitable locations and general design will be different. Depending on which policy is used to manage water quality and surrounding nature in an estuary the design will also be different. In this chapter the starting points are listed which are used in the thesis. In section 2.1 the functions of the solution are explored and their relative importance as assumed for this project is explained. Section 2.2 explains the water level management strategies that will be used in the thesis. These include requirements for the minimum tidal stroke at the artificial lake as well as the requirements for the minimum and maximum water level. Finally in section 2.3 it is explained that the solution will be tested against four sea level rise scenarios.

2.1. Functions of the solution

Table 2.1 summarizes the functions of the solution and their relative importance. Next to the function the element of the solution is described that fulfills this function. Lastly it is listed what parameters are determining for the necessity/success of the function. Below the table it is explained why the functions are in this order.

Table 2.1: Summary building blocks solution

Functions	Importance	Fulfilled by	Parameter of influence
Flood protection	Primary	Dam	Sea level rise
Water quality and nature preservation	Secondary	Culverts with pumps	Sea level rise and minimum tidal stroke
Energy generation	Tertiary	Turbines	Tidal range

First of all it is important to recognize that the tidal power station with pumping capacity is an integrated solution with a dam. The solution does not exist on its own. This means that a dam will already need to be in place, or be placed in order to apply a tidal power station with pumping capacity.

A traditional tidal barrage power station has the sole function of generating energy. Often this has proven to be a cost intensive way of generating renewable energy compared to wind and solar energy generation. (IRENA (2018)). Furthermore environmental concerns have caused several tidal energy projects to be canceled. Only at locations with a very large tidal range could a tidal power station potentially be profitable.

At most sites closing a basin will thus need more reason than energy generation alone. At the Grevelingenlake the Brouwersdam was placed as part of the Deltaworks, to protect the area against flooding. In this project the assumption is made that this will also be the case for any other location. The primary function of the solution will be flood protection. A dam is only constructed where it is needed to protect the area.

A tidal power station with pumping capacity is in fact an improvement of a standard dam, so that water is circulated in the created lake and the water quality consequently can be maintained at a certain standard. The tidal stroke at the lake which is introduced by the opening can furthermore maintain the intertidal character of the basin. The pumps are needed in the long term to force this circulation of water when sea level rises above levels where water can naturally flow out of the lake. This is only for the situation that the water level in the lake is not allowed to rise with the sea level, otherwise an opening in the dam would suffice. This condition is discussed in more detail in section 2.2 about the water level management strategy. From the tidal power station with pumping capacity the pumps are thus the most essential part.

2.2. Water level management strategies

The dam with tidal power station including pumping capacity has the potential to preserve some of the nature in the basin, as the tidal stroke can partially be kept in tact. A pump/turbine station can manage the natural quality by means of setting the minimum and maximum water level, and setting the minimum tidal stroke. As discussed in section 2.1 the solution is only needed if the water level needs to fluctuate around a fixed level, this is assumed for all basins under evaluation. Natural quality is more than just the quality of the water, but also includes quality of inter tidal flats and nature around the waterline. Besides water quality and surrounding nature, infrastructure and agriculture around the basin will also be affected by the choices in water levels and tidal stroke. These considerations are discussed in more detail appendix A.

The tidal stroke will be a key indicator of the performance of the structure. If the minimum tidal stroke is not met, the nature quality will drop below the set standard. The best strategy to adapt differs for a situation with a dam or without a dam and is highly dependent on local policy. The situations, with and without dam already present, are discussed separately in this section.

2.2.1. Situation with dam already present

When a dam is already present, it is desirable to keep the tidal stroke as small as possible whilst still improving the water quality. A small tidal stroke is beneficial because it has a minimum impact on the above water nature as well as on the infrastructure. In this case the following relationship can be used as an estimation of the minimum tidal stroke:

$$R_{\text{min}} = \frac{0.4}{22} d_{d_{\text{avg}}} \tag{2.1}$$

With R_{min} the minimum tidal stroke in meters
With $d_{d_{\text{avg}}}$ the average depth of the deepest parts in meters

This relationship was derived from the minimum tidal stroke at the Brouwersdam, and uses the assumptions that the required stroke is related to the depth of the deeper parts in the basin. These are the parts of the basin where hypoxic zones are formed and water quality is poorest. This is discussed in more detail in appendix A.

The considerations for the minimum and maximum water level are the same as for the minimum tidal stroke. The water level cannot drop by too much (minimum water level not too low) as this could have an impact on the sailing depths and underwater nature being exposed. The water level should also not rise too much (maximum water level not too high) as this can damage the above water nature and infrastructure and could cause a flooding problem. The minimum and maximum water level will therefore be based on the tidal stroke. The starting point is that the maximum water level will be half the minimum tidal stroke above mean water level and the minimum water level will be half the tidal stroke below mean water level. As this is very restrictive for the energy generation it can be researched for each location if a larger range is allowed. For the model it is assumed that a slightly larger range is allowed: 10% increase at each side.

2.2.2. Situation without dam already present

When there is no dam present the tidal stroke should resemble the current situation as much as possible, for the sake of water quality, nature and infrastructure. The costs of the project are reduced when a smaller tidal stroke is accepted as a smaller inlet will suffice. Depending on what the project budget is and what value is given to the intertidal character of the basin, two strategies are further explored.

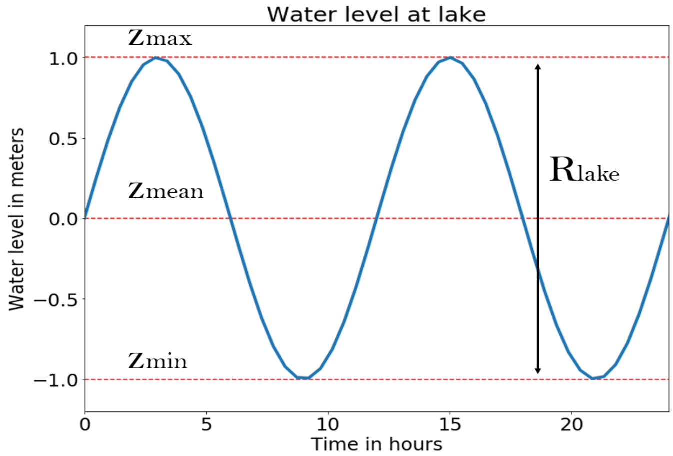


Figure 2.1: Minimum, mean and maximum water level and the tidal stroke at a lake

- **Nature conservation strategy**: Means to choose a minimum tidal stroke close to the original situation. This is assumed at 80 %. With this strategy the intertidal character of the basin is maintained as much as possible, but at high construction costs.
- **Economical strategy**: Means to choose a minimum tidal stroke that fulfills the requirement of water quality only. This results in lower construction costs and higher mitigation costs as less nature conservation occurs.

The maximum and minimum water level ($z_{min}$and z_{max}) for the two strategies will both be half the tidal stroke at the lake (R_{lake}) below and above the mean water level at the lake (z_{mean}). This is illustrated in figure 2.1.

2.3. Sea level rise scenarios

The feasibility of the proposed solution is largely dependent on the rate of sea level rise, as was explained in chapter 1. In this thesis it is chosen to test the solution against four sea level rise scenarios as defined by the Intergovernmental Panel on Climate Change (IPCC).The bases of these projection are the so called 'Representative Concentration Pathways' (RCPs). These are scenarios that lay out possible futures with different rates of greenhouse gas emission and consequently radiative forcing. They show one of the many pathways of how the future may develop due to multiple variables such as technological advancements, energy and land use, the emission of greenhouse gases, socio-economic change and climate policies. These scenarios are used across the scientific community to better compare study results. The RCPs are the newest generation of scenarios used for climate change research and assessment. (Moss et al., 2010), (van Vuuren et al., 2011)

There are four RCPs selected for further use in scientific research. This includes one very low emission scenario (RCP 2.6), two intermediate (RCP 4.5 and RCP 6) and 1 very high emission scenario (RCP 8.5). Their names include the radiative forcing target level for 2100 (W/m^2).

The most recent projections for sea level rise according to the IPCC are summarized in table 2.2. This global mean sea level rise is "likely (medium confidence) to be in the 5 to 95% range of projections from process based models. For RCP 8.5, the rise by 2100 is 0.52 to 0.98 m with a rate during 2081–2100 of 8 to 16 mm yr^{-1}." (Oppenheimer et al., 2019)

Table 2.2: Projected global mean sea level rise for the RCPs for the period 2081–2100, compared to 1986–2005 from Oppenheimer et al. (2019)

RCP	Sea level rise (2081 - 2100)
2.6	0.26 - 0.55 m
4.5	0.32 - 0.63 m
6.0	0.33 - 0.63 m
8.5	0.52 - 0.98 m

Sea level rise is not uniform across the globe but will differ from region. This is caused by many processes such as the attraction of water by large land or ice masses. (Hsu and Velicogna, 2017) (Mitrovica et al., 2001) In figure 2.2 global sea level rise is translated to regional sea level rise for the three RCPs according to these projections. This same relationship will be used to scale the sea level rise scenarios to specific locations in this thesis.

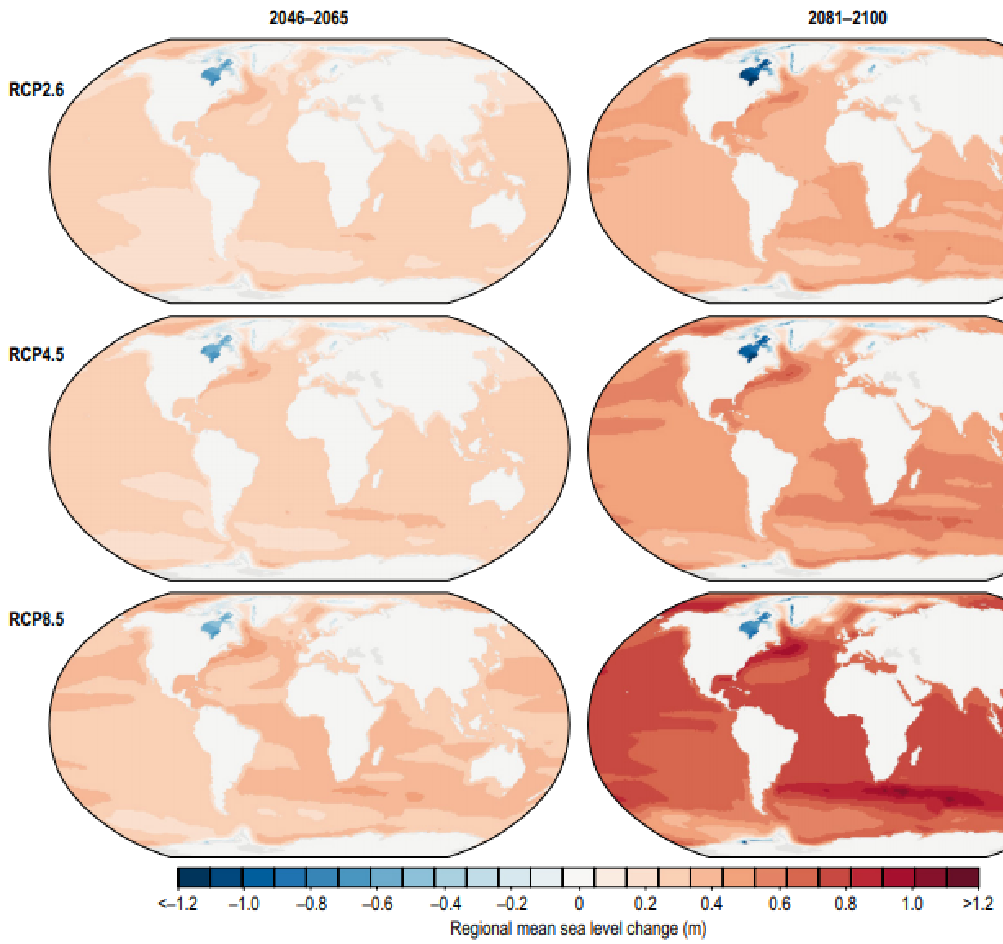


Figure 2.2: Projections of regional mean sea level rise in meters for RCP 2.6, 4.5 and 8.5. From Oppenheimer et al. (2019)

CHAPTER
THREE

LOCATION SELECTION

This chapter focuses on the selection of potential sites that will be further analyzed for feasibility. As a first step in sections 3.1 and 3.2the study done by Deltares is explored to see what methods were used and to what extent the proposed selection method can be used for this research. Properties of the dataset checked against satellite imagery. Based on the priorities of the solution established in chapter 2, a selection of locations is made that will be further analyzed in sections 3.3 and 3.4.

3.1. Study by Deltares

This section gives an overview of the Deltares study. First the original study is summarized including the method of data acquisition and scoring system used.

3.1.1. General approach

The study performed by Deltares explores the export potential of the solution strategy for the Brouwersdam to other sites. The identification of these potential sites was done based on threshold values of selection criteria. All potential locations were then given a score by assigning weighing factors to the selection criteria. This resulted in a ranked dataset of 461 potential export locations. (Deltares, 2019)

The selection criteria were established through expert opinions and discussions. Based on available data this resulted in the following list and threshold values. An overview of the used values is given in table 3.1.

Table 3.1: Selection criteria and weighing factors as used by Deltares

Criterion	Threshold Value	Weighing Factor	
Flood Risk	>0	w_f	1
Mean Tidal Range [m]	≥ 2	w_R	1
Basin Area [km^2]	50	w_S	0.5
Population	1*10^6	w_p	0.8
Elevation [m]	5	–	
Natural Areas [IUCN category]	-	w_N	0.4

As can be seen from table 3.1 and equation 3.1 flood risk and tidal range were identified as the most important criteria with the largest weighing factor. Flood risk is needed for the solution to make sense, and a tidal range is needed for the technology to work. A large population contributes to the suitability score of a location as the investment in such a dam is justified when it protects a lot of people. A larger basin will have more energy potential, but as this is not the primary function of the solution the weighting factor is lower. Furthermore if the location is in an area with high natural value, then this is also incentive to implement the solution as high value nature should also be protected.

3.1.2. Data acquisition

In this subsection the type of data used in the study is described. This is relevant because the same dataset is used to select locations in this study. This gives an idea of what type of data is dealt with and of the reliability and accuracy of the database.

Basin area
The basin areas are obtained from two different datasets (Caldwell et al., 2019) and (WWF, 2004). This was imported in QGIS, and all water bodies further than 10 km from the shoreline were removed. This resulted in all possible coastal lakes. The spatial resolution 15 arc seconds. For each location a centroid was defined in QGIS with a 50 km radius of interest. For this area the flood risk, tidal amplitude and population were determined.

Flood exposure
For the flood exposure use has been made of a global flood hazard map. (Dottori et al., 2016a). The map provides inundation levels in meters with a 20 year return period. The spatial resolution of the map is 30 arc seconds, which equals approximately 1 km. The inundation levels can be used as a measure of flood exposure. (Dottori et al., 2016b) For the exposure level the highest value in the area of interest was used as this indicates the highest risk that the area faces.

Tidal variation
The Global Tide Surge 18 Model v3.0 was used to obtain values for the tidal range. (Irazoqui et al., 2018). With this model both today's tides and future tide's can be calculated. This model uses the RCP 4.5 SLR projections from CMIP5. The tidal variation used is for the year 2083 with 40 cm sea level rise. In the model information on tidal variation is known for points approximately every 20 km along the shoreline. For the tidal variation the value was selected that was closest to the body of water.

Population size
The population size is derived from a database with the size projected for 2020. (CIESIN, 2018) The aggregated population is calculated for the area of interest.

Natural value
Protected areas are inventoried by intersecting the locations with protected areas in the IUCN database. IUCN categories were treated in section A.1. The ranking in the dataset runs from 1 to 7, where 1 is the most protected.

3.1.3. Scoring system Deltares
Equation 3.1 is used to calculate the final suitability score. With f the normalized floodrisk, t the normalized tidal variation, p the normalized population, a the normalized basin area and n the normalized nature value.

$$\text{score} = w_f f + w_R t + w_p p + w_S a + w_N n \tag{3.1}$$

3.2. Discussion study Deltares for application in this thesis
The approach used for the study by Deltares has been analysed and several aspects have led to the conclusion that the proposed selection by Deltares cannot be directly used in this research. These are highlighted in this section. The alternative selection approach that takes these factors into consideration is discussed in section 3.3.

3.2.1. Research method
Sea level rise scenarios
The effect of sea level rise is taken into account indirectly, the flood risk and tidal range values are both based on the RCP4.5 scenario with 40 cm sea level rise in 2083. This was simulated with the Global Tide Surge Model developed within Deltares. (Irazoqui et al., 2018) RCP4.5 is a moderate scenario. Most likely the dataset would look differently if a different scenario was chosen, or the most current projections are used.

The aim of this research is to evaluate multiple rates of sea level rise, while this dataset is limited to one. It is however beyond the scope of this thesis to generate these different datasets. While a scenario with more sea level rise would identify more locations with a flood risk, this will only increase the flood risk at the locations already in the dataset. And these are the locations that are most suited for the intended solution. A scenario with less sea level rise would reduce the number of locations in the dataset, but would still include the locations that are most at risk of flooding. For this reason the dataset suffices for the identification of locations at risk for flooding. The selected locations will be supplemented with additional information for different rates of sea level rise in the next stages of the analysis.

Scoring system

The scoring system applied in the study is not suitable to select the most promising sites for the intended application of this thesis. Deltares works with normalized scores for each criterion. This normalized score is based on the following equation:

$$\text{score} = \frac{\text{value} - \text{minimum}}{\text{maximum} - \text{minimum}} \tag{3.2}$$

For selection criteria with a wide range of values (especially with large outliers such as for basin area) this proves problematic. When there is a very large range this also results in a large range in the normalized scores, as these are all between 0 and 1, all locations with a low value will have a value of approximately 0 and thus not have any contribution to the final score. In figure 3.1 the cumulative distribution of basin areas is shown, the red lines indicate the point that is 100 times smaller than the largest point, 97.7 % of the locations have a basin area smaller than this location and will thus all get a value of zero.

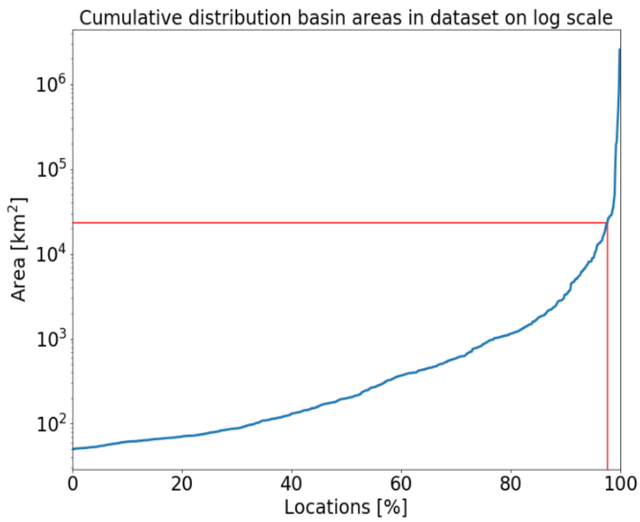


Figure 3.1: Cumulative distribution basin areas in dataset

For the mean tidal range and flood exposure the range of data is much smaller, so for these criteria all locations have a representative score. This causes the final score to be the product of mainly the tidal range and the flood risk, giving an incomplete view of the situation.

Another limitation was found in the ranking of the protected nature areas. Locations situated in a protected area are given a score 1-7, while areas that are outside a protected area are given the score zero. In the normalized scoring system where 1 is intended to yield the highest score, it is actually the locations with value zero that get the highest score.

No visual inspection

By analyzing the locations through satellite imagery it becomes clear that there is a wide variety in the locations in terms of basin size, morphology, population density, infrastructure and the presence of a dam. The full satellite image analysis can be found in appendix B. Site specific qualities can affect the suitability of a site and can be reason to exclude a site from the selection. Examples are the presence of a large port and a unsuitable basin geometry. Selection should therefore be done not only based on the previously named criteria: mean tidal range, basin area, flood exposure and population size and protected natural areas, but also based on these site specific aspects, which means visual inspection and further research after an initial filtering round.

3.2.2. Dataset

Basin Area

The analysis of locations on satellite imagery led to more insight on the reliability of the dataset. The complete analysis can be found in appendix B. The information on basin areas is unreliable. Out of a random sample of 20 locations 16 had an area significantly different than recorded in the dataset. Incorrect was defined as a deviation larger than 20%. The "largest basin" which is located in Egypt, has a size of 2.56 million km^2 according to the dataset. This is larger than Egypt itself (1 million km^2) or the neighboring red sea (0.44 million km^2). Without correction for this parameter the surface area is thus not a reliable selection criterion. For further analysis the number of locations that are selected should be limited such that all information can be verified and basin areas can be corrected if necessary for specific cases.

Mean Tidal Range

The mean tidal range (MTR) in the dataset also did not correspond with data found for example with Delft3D Toolbox. Out of a random sample of 8 locations 4 show locations with incorrect MTR's. Incorrect is defined as a deviation of more than 20% from the value in the dataset. This discrepancy can partially be explained by the fact that the tidal signal in the Deltares dataset are derived with GTSMv3.0. (The point of interest in time was chosen to be when 40 cm sea-level rise is reached with Representative Concentration Pathway (RCP) 4.5, which corresponds to 2083.) However, differences can be large, for example: Izmir bay in Turkey according to dataset has a MTR of 4.6 m, while according to Delft3D toolbox the MTR for 2020 is in the range of 0.2 m.

Doubles

From inspection of sites through satellite imagery it becomes clear that there are a number of basins where multiple points are defined. Each point represents a potential location according to the dataset. However these points would result in only one pump tidal station. This reduces the amount of potential locations.

3.2.3. Missing features in analysis

In this subsection some suggestions are listed that can be added as criteria for the dataset. Due to limited time and resources these cannot all be added during this project. Some of these points will partially be used for the targeted selection approach in section 3.3.

- Building a dam for the purpose of tidal power plant will often not result in a positive business case. An important element of the potential site selection is therefore that a dam is necessary to mitigate high flood risk.
  - To this end a parameter can be introduced: flood exposure × population × GDP, to quantify risk and thus the necessity for flood protection (dam or otherwise). This will be further discussed in section 3.3.1.
  - Whether a dam is the appropriate flood defense versus for example a dike can be first estimated with a ratio between the perimeter of the basin versus basin mouth length. Where a large ratio gives an indication that a dam could be favorable. Furthermore the depth across the basin mouth can also give an indication whether a dam is feasible.
- Important are the demands for the minimum tidal stroke. If only used to refresh water in the deeper parts of the basin, the minimum tidal stroke can be relatively small to serve this purpose. Pumping will not be needed in shallow basins for many years as only a very small tidal stroke will be needed. Features of the basin bathymetry could serve as an additional selection criterion.

3.3. Proposed selection approach

In this section the approach is laid out that will be used to select suitable locations for further analysis from the Deltares dataset.

The lessons learned from the analysis of the Deltares study are applied where possible. Parameters with a lot of incorrect data such as the basin area will not be used as a criterion. The protected areas are also not used but are taken into account in the next phases in terms of mitigation costs. Filtering is done through setting threshold values for selection criteria and inspection of satellite imagery and verifying tidal data with Delft3D-toolbox. Two new scoring parameters are introduced: flood risk and energy yield.

To establish the threshold values the data is analyzed through exploratory data analysis. This aims to inventory how the selection criteria in the dataset relate to the scoring parameters of flood risk and energy yield. The goal of this is to find optimal threshold values to get a selection with the desired properties and desired size.

3.3.1. Introduction scoring parameters

As discussed in section 2.1 the primary function of the solution is to serve as flood protection, the secondary function is ensuring water and nature quality and the tertiary function of the solution is energy generation. Ideally two scoring parameters are introduced: one for flood risk which is given most weight and one for energy generation with less weight.

There needs to be a necessity for protection and thus high flood risk needs to be present, the higher the more incentive there is to place an intervention like a dam. Equation 3.3 gives the expression for risk used in this study. The exposure is the inundation levels with a 25 year return period as provided in the dataset, and the consequences are here quantified by the population around the basin area. If a lot of people are living in an area the damages will automatically be larger. It should be noted that as mentioned in subsection 3.2.3 the GDP would be a valuable contribution to the consequences but due to lack of data this is not workable in this phase of the study. Also the equation 3.3 is not the traditional definition of risk but serves as a scoring parameter within this study.

$$\text{Risk} = \text{Exposure} \times \text{Consequence} \tag{3.3}$$

with exposure as the maximum inundation level with a 25 year return period [m]
with consequence the population size [-]

In theory the potential energy yield for a location can be calculated with equation 3.4.

$$E = \frac{1}{2}\gamma S R^2 \tag{3.4}$$

with γ the volumetric weight of water [kN/m^3]
with S the surface area of the basin [m^2]
with R the mean tidal range [m]

This is the theoretical maximum and not the most economical. However as the basin areas in the dataset is inaccurate, this parameter cannot be used in this study. Only the tidal range qualifies as a selection criterion. The tidal range thus equals the scoring parameter of energy yield within this study.

3.3.2. Exploratory data analysis

Correlation selection criteria

Before looking into other properties of the dataset the correlation between the criteria is looked into. When a strong correlation is found between two criteria this is useful for the filtering approach. If for example basin area and mean tidal range have a strong positive correlation than only one of the criteria needs to have a threshold value and it automatically follows that the larger the mean tidal range the larger the energy yield. Table 3.2 shows the Pearson correlation coefficients for the combinations of criteria.

Table 3.2: Pearson correlation coefficients of selection criteria

	Basin Area	**Flood Risk**	**Mean Tidal Range**	**Population**
Basin Area	1.00	0.00	0.04	0.03
Flood Risk	0.00	1.00	0.16	0.14
Mean Tidal Range	0.04	0.16	1.00	0.14
Population	0.031431	0.14	0.14	1.00

As can be seen the largest correlation that is found is between flood risk and the mean tidal range: 0.158. This is a weak correlation. For the evaluation of the threshold values the selection criteria will be regarded as uncorrelated.

Tidal Range

The minimum tidal range is 0.04 while the maximum tidal range is 9.30 m. The number of filtered locations with increasing threshold value for the tidal range is shown in figure 3.2

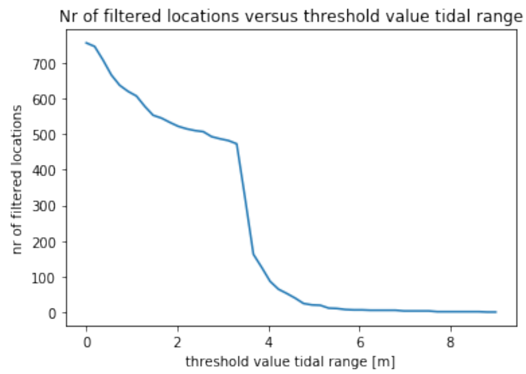


Figure 3.2: Number of filtered locations for increasing threshold value of mean tidal range

As can be seen in figure 3.2 there is a steep decline in the number of filtered locations between 3.5 and 4 meters. This means that a large number of locations have a tidal range within these bounds.

Effect threshold values flood exposure and population on flood risk

Both flood exposure and population number have an effect on the risk. Both parameters have a linear effect on the risk. As the population number can be very large and has larger variety, this parameter has the most potential to influence the risk.

Figure 3.3 shows a heatmap for the average risk per basin category. All locations are grouped by their population size and flood exposure. For each group the average risk is calculated. Because of the wide range in the data the colouring is done on a lognormal scale. A dark orange colour means a high average potential yield, the number in the box indicates how many basins are in this group. White boxes indicate a zero value. There is a large amount of locations with a small population size (x < 40 000) in every flood exposure category. The very large populations and very large risk exposure are grouped together in one category: poupulation > 1.2 million and exposure > 5 m. In the last column it can be seen that for all exposure categories there are locations with a very large population. Resulting in very large flood risk.

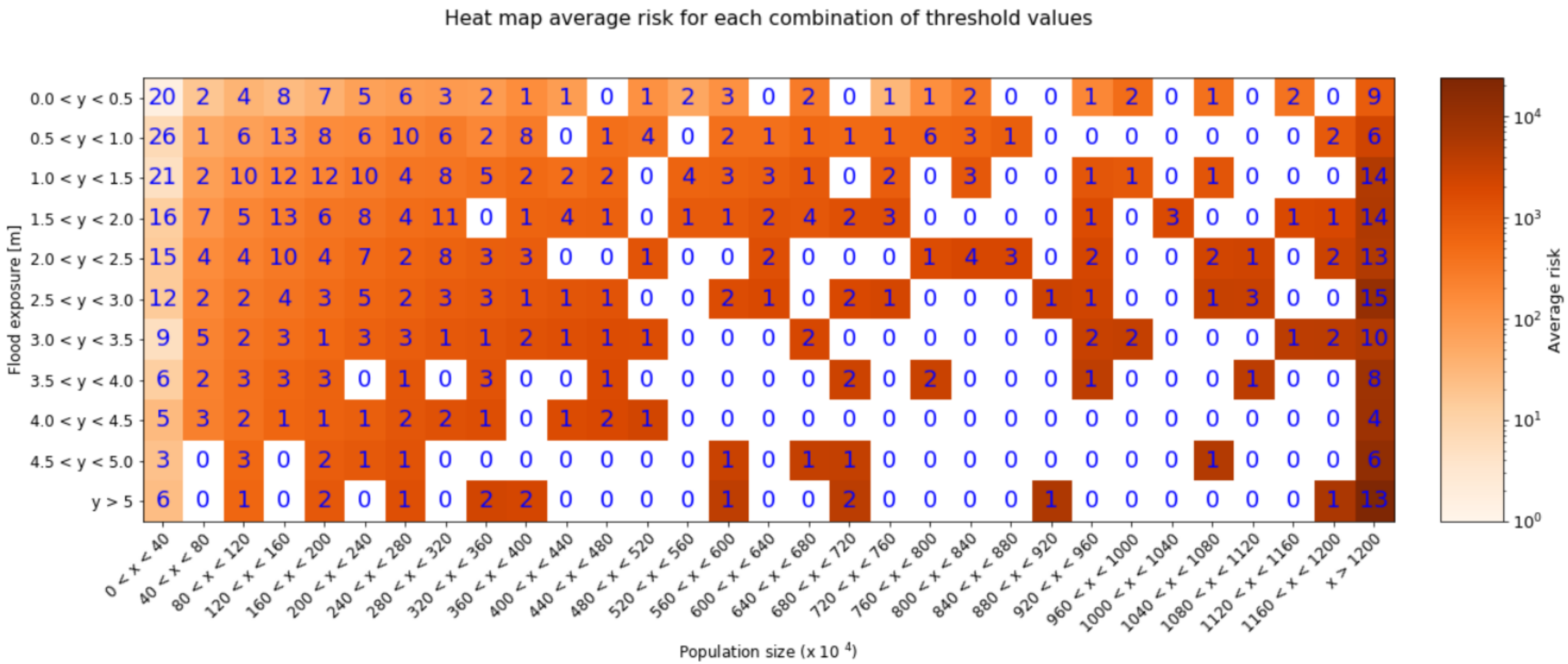


Figure 3.3: Heat map showing the average risk in intervals for increasing threshold values of population size and flood exposure. In each interval the number of locations is given.

3.4. Final selection: Flood risk and tidal range

In total 3 preliminary selections were made, this is done to give an idea of how the type of location changes as the selection criteria change. One selection focuses on energy generation, one selection focuses on flood risk, and the last is a combination flood risk and tidal range. The latter being the selection that fits best with the intended purpose of this thesis. Only the final selection is presented here, the preliminary selections can be found in appendix B. The locations in the selection are inspected through satellite imagery, and a few basic characteristic such as possible length of a barrage and the depth of the deepest part in the basin and the depth along the cross section are looked into, to give a first idea of the feasibility of a dam with tidal power station at this location.

The surface area was excluded from the selection criteria as this parameter has proven to be unreliable. Considering the two water level management strategies (section 2.2), locations are chosen with a tidal range varying from small to large. The strategies under evaluation are: nature strategy with tidal stroke as close to the original tidal range as possible and the economic strategy with the tidal stroke large enough to refresh the water (generally a much smaller stroke than the nature strategy). A large tidal range at sea and a small tidal stroke at the basin means a large time window for water to flow out naturally. As the tidal range decreases so does the time window for natural outflow. Therefore, locations with a small to medium tidal range are more likely to need pumps if an economical management strategy is applied. When a nature conservation strategy is chosen for all tidal ranges pumps will likely be needed. The energy potential for large tidal ranges is bigger. To assess under what circumstances a tidal power station with pumping capacity is feasible a selection is chosen where all these cases are represented.

From the heat map for flood risk (figure 3.3) it can be seen that very high risks occur for populations larger than 480 000 and flood exposure of more than 2.5 m. This leaves 81 locations with high flood risk. 9 suitable locations are manually selected from this selection to have the desired spread in tidal ranges and also seen as suitable based on visual inspection of satellite images. The Brouwersdam is included as this is an interesting reference case, there are two other locations in South Korea where a dam is already present in the basin, which are interesting to include.

The 8 locations are shown in figure 3.4. Upon closer inspection of the locations it becomes clear that for several locations there are multiple cross sections where a dam could be placed. Some locations thus result in multiple cases, from this point on indicated as 'location name + letter'. A satellite image of each location is shown in figure 3.5, where the cross sections are indicated where a dam could be placed. Table 3.3 gives an overview of all relevant, verified, data of each location ordered by tidal range.Mokpo B and Asan Bay C are both pre-existing dams.

Figure 3.4: Top 9 location selected based on high flood risk and high energy potential

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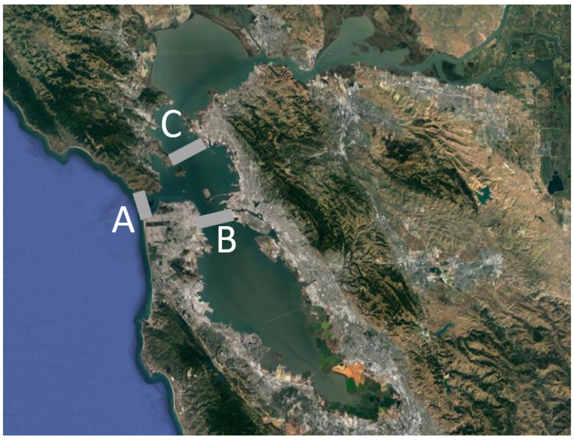
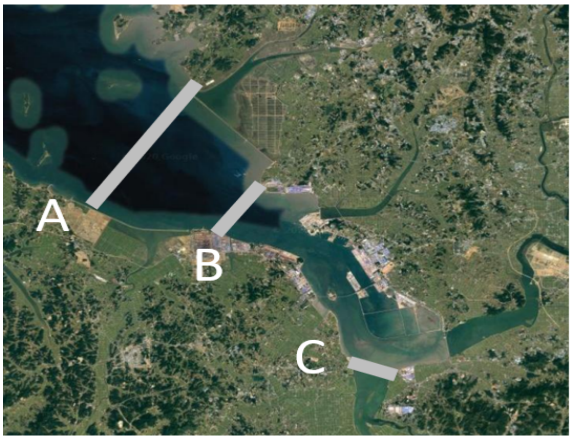
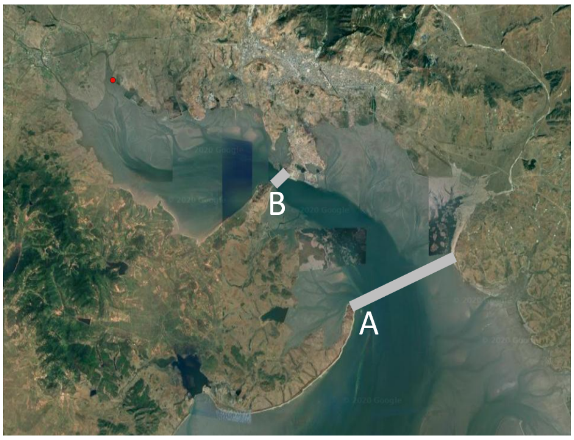


(a) San Francisco Bay, Unites States

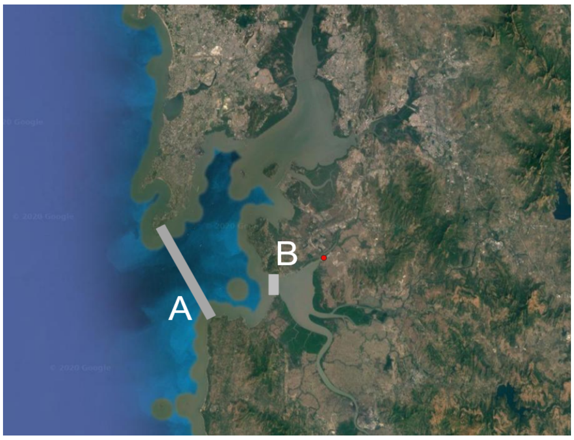
(b) Mokpo, South Korea



(c) Asan Bay, South Korea



(d) Haeju Bay, North Korea



(e) Dharamtar Creek, India

(f) Brouwersdam, Netherlands

(g) Xinghua Bay, China

(h) Dee Estuary, United Kingdom

Figure 3.5: Final selection (images on different scales)

Table 3.3: Overview relevant data final selection

Location	**MTR [m]**	**Area [km ^{2}]**	**Length cross section [km^2]**	**Perimeter [km]**	**Deepest point basin [m]**	**Average depth cross section [m]**
San Francisco A	1.7	950	7	214	50	40
San Francisco B	1.7	500	6.5	125	19	15
San Francisco C	1.7	300	6.5	85	23	10
Brouwersdam	2.3	110	7	60	22	n.a
Mokpo A	3	50	5	42	26	15
Mokpo B	3	15	2	22	17	n.a
Dharamtar Creek A	3.2	280	11	97	10	10
Dharamtar Creek B	3.2	20	3	18	6	6
Asan Bay A	4	250	17	95	4	9
Asan Bay B	4	100	5.5	55	4	17
Asan Bay C	4	15	4	22	4	n.a
Xinghua Bay	4.2	480	14	120	25	22
Haeju Bay A	5.8	110	5	65	10	6
Haeju Bay B	5.8	45	1	40	10	6
Dee Estuary	6	90	8	43	15	5

CHAPTER
FOUR

CONCEPTUAL DESIGN

Before starting the selection of locations and models it is important to visualize the solution. In this chapter a conceptual design is given of a dam with tidal power plant with pumping capacity. This design is used as the basis of the hydro energetic cost model in chapter 5. A second conceptual design is added without a tidal power plant but with culverts to allow flow between the sea and lake. This design will serve as a means of comparison in the business cases of chapter 8. An overview is given of the designs with some general aspects.

4.1. Design dam with tidal power station

In figure 8.1a an overview can be seen of the conceptual design. A dam is used to close of a tidal estuary creating an artificial lake. The tidal power station with pumping capacity is incorporated in the dam. A lock is included to allow ships to pass. A road can be built over the dam. The latter two elements being optional, i.e. not essential for the solution to work.

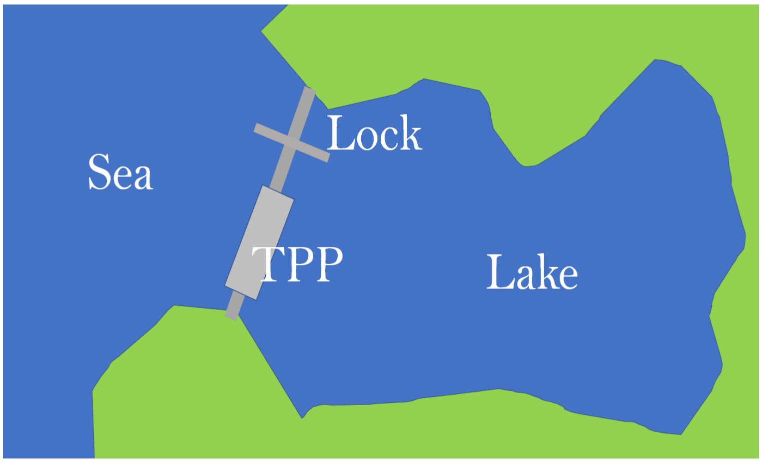
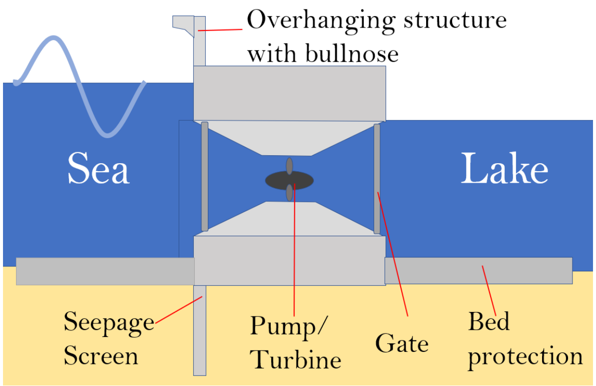


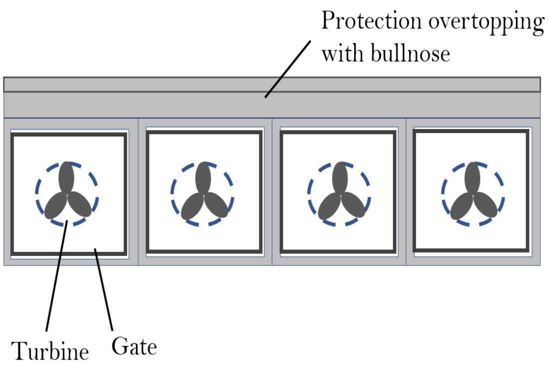
Figure 4.1: Overview conceptual design Dam with tidal power plant

The pump/turbines are housed in so called powerhouses. Figure 4.2a shows the side view of the powerhouse, and figure 4.2b shows a frontview. The powerhouse has an in/outlet at both sides, the pump turbine is located in the center. Gates are included at both sides, these are used to regulate the water level. Two gates are needed for maintenance which is performed when the turbine is dry. The empty unused spaces in the caisson can be filled with sediment for stability. At the sea side, the caisson is locally higher and has an overhanging structure. This is included to reduce overtopping of waves. This is especially important when a road is built over the dam. A seepage screen is included at the bottom. Bed protection is very important to prevent instability of the structure as flow is locally altered due to the in and outflow through a constricted area.

The part of the dam without any important equipment can be designed in several ways and is largely



(a) Side view powerhouse tidal power station

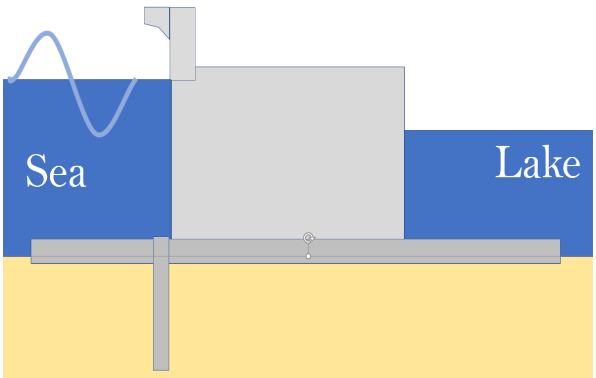


(b) Front view conceptual design powerhouse tidal power station

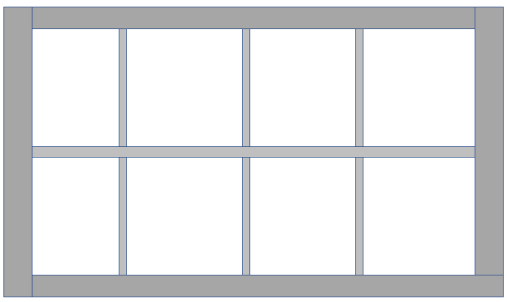
Figure 4.2: Conceptual design powerhouse

dependent on the closure technique required. As the lake is closed, very high flow velocities will occur. The dam can be constructed from large rocks that are dumped to create the closure, with a cover layer on top. Alternatively, caissons can be used to close the dam These can be filled with sediment for stability. For the closure of the Brouwersdam both techniques were used. Part is constructed with dumped material while another part has the first layer consisting of coarse material and caissons resting on top of this. It will be very location dependent what the best method is. However in order to be able to compare locations it is desirable to choose one general design. In this thesis it is chosen to use caissons to close the barrage. This is seen as a realistic option due to the fact that caissons will have to be constructed for the powerhouse. Using caissons to close the gap will increase the repetition factor for the formwork and dry dock and will reduce the costs per caisson. An additional advantage of this method is that the closure of the dam can be done relatively quickly once the caissons are finished. The closure can then be planned during optimal weather conditions.

The caisson is hollow on the inside with ribs for structural stability. The caisson can be filled with ballast when it is sunk to its final location. Figures 4.3a and 4.3b give an impression of what the caissons used for this part of the dam will look like. Since there is a head difference across the structure water can flow underneath the structure and piping can occur. A seepage screen is installed to prevents this by creating a longer seepage length. The interaction between waves and the dam can cause a disturbed wave field and result in a scour hole in front of the structure. Bottom protection should be placed to prevent this. Again, an elevated wall with overhanging structure is implemented to reduce overtopping.



(a) Side view dam closure with caisson



(b) Top view of caisson with internal ribs, used for closure of dam

Figure 4.3: Conceptual design closure dam

4.2. Design dam with culverts

In figure 4.4 an overview can be seen of the conceptual design. A dam is used to close of a tidal estuary creating an artificial lake. Part of the dam has culverts. A lock is included to allow ships to pass. A road can be built over the dam. The latter two elements being optional.

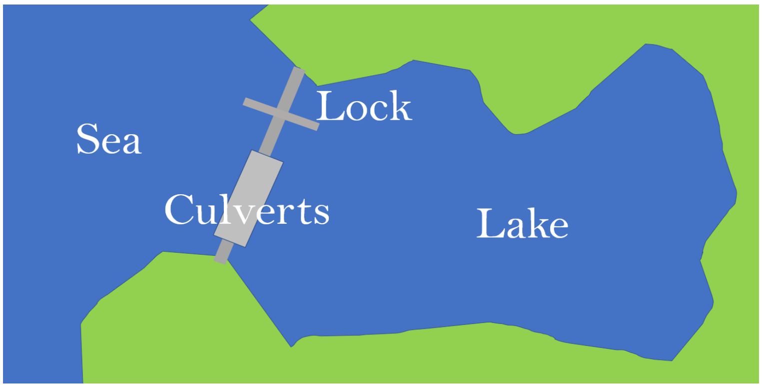
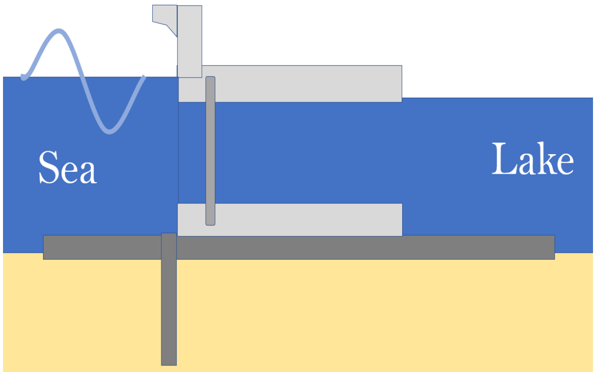
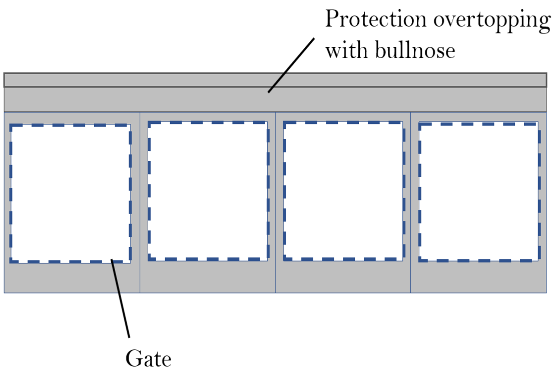


Figure 4.4: Overview conceptual design Dam with culverts

Figure 8.2b shows the side view of the culverts, and figure 4.5b shows a frontview. For the culvert only one gate is needed to regulate the water, as no maintenance is needed within the culvert. It is important to note that the flow area is larger because there are no obstacles. In the powerhouse the inlet cross section narrows towards the pump turbine, whereas here the cross section remains unchanged. This means that for the same outer dimensions, the discharge through the culverts will be larger. Again the measure against overtopping is included. Other important elements are the bed protection and seepage screen.



(a) Side view culvert



(b) Front view culvert

Figure 4.5: Conceptual design culverts

The part of the dam without any important equipment has the same design as the alternative with tidal power plant of section 4.1.

CHAPTER
FIVE

MODEL TIDAL POWER STATION WITH PUMPING CAPACITY

In this chapter the hydro energetic cost model is discussed. The model is used to calculate the energy yield, pumping needs, resulting tidal stroke at the lake, costs and lifetime of a tidal power station with pumping capacity. The model uses the configuration of the conceptual design in chapter 4 as its starting point. In the next chapter the model is incorporated in an optimization model to generate designs for all locations. In section 5.1 the set-up of the model is explained, starting with the basic principles and concluding with the workflow of the model. In section 5.2 the building blocks of the model are discussed more in depth. It is explained how each element is schematized and the relevant mathematical relations are presented.

5.1. Model set-up

The model makes use of a two sided energy generation scheme. This means that energy is generated both during high tide and low tide. Figure 5.1 illustrates how this scheme works.

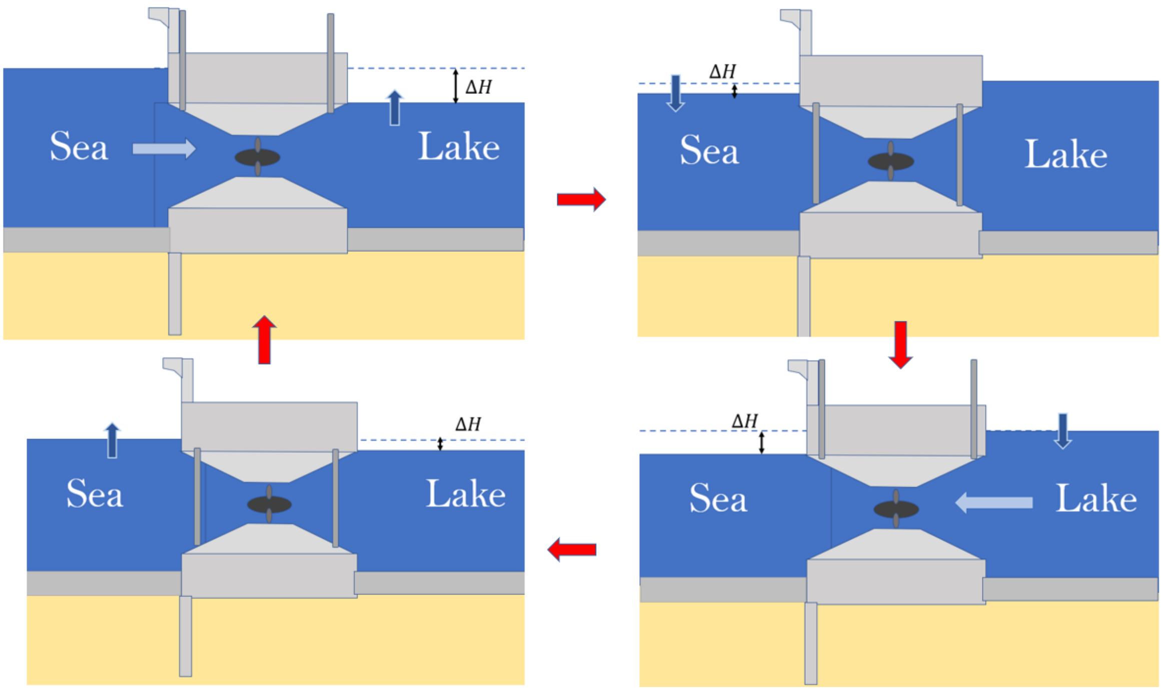


Figure 5.1: Two sided energy generation scheme

The light blue arrows symbolize the flow direction of the water, the dark blue arrow symbolize the change in water level and the red arrows indicate the transition from one phase to the next. When the water level in at sea is higher than at the lake, water can flow from sea through the turbines into the lake, generating energy. When the water level at the lake reaches a certain point, the gates are closed. While the gates are closed the water level in the lake is kept constant and the water level at sea decreases. When the water level at the lake is higher than at sea, the gates are opened and water flows from the turbines towards sea. As sea level rises pumps come into play, they aid the outflow of water from the lake to sea during this second phase. In the following paragraphs it is explained how this scheme is translated to a computer model.

In general terms the model consists of three elements, the sea, the tidal power station and the lake. The main elements of the model and the output are illustrated in figure 5.2.

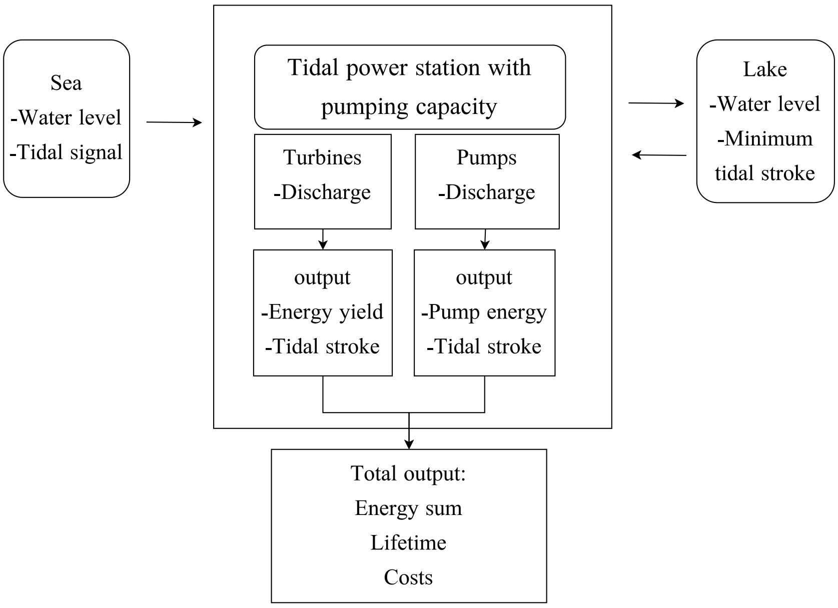


Figure 5.2: Main elements model

The power station gets both input in the form of water level at sea and at the lake. The station decides based on these water levels whether to:

- Let water flow from the sea to the lake while turbining
- Let water flow from the lake to the sea while turbining
- Let water flow from the lake to the sea while pumping
- Hold the water level at the lake by closing the gates, while the water level at sea changes

The lake receives information from the power station by discharging water in or out of the lake, changing the water level, or by holding it at the same level when the gates are closed. The water level at sea is not affected by the power station because of its much larger volume. The water level at sea is only changed by the tide, the tidal signal is thus the primary input.

5.1.1. Tidal power station without pumps

For the hydro energy model first a situation without pumps is explored. The basis of this model without pumps was produced by Royal Haskoning. Haskoning DHV (2010). The principle is to create a head difference between sea and the lake (by keeping the gates closed) and letting water flow through the turbines only during a specific range of head differences by opening the gates (from $H_{start}$to H_{stop}). The head difference is defined as:

$$H = z - h_{sea} \tag{5.1}$$

With z the water level at the lake in meters
With h_{sea} the water level at sea in meters

When the minimum or maximum allowable water level at the lake is reached (z_{min} and z_{max}), the gates are closed, regulating the water level at the lake. As the water flows from one side to the other, the turbines are set into motion and energy is generated.

This process can be translated into four phases, where the next phase is initiated when the transition conditions mentioned in the previous paragraph are met. This is summarized in table 5.1. Figure 5.3 shows how the water level at sea and at the lake change over time. The phases as defined in table 5.1 are indicated in the figure between time intervals.

Table 5.1: Two sided energy generation scheme, no pump

Phase	Operation	Turbines	Action at end operation	Transition conditions	Next Phase
1	Fill basin	Running	Close turbines	$z = z_{max}$ or $H > H_{stop}$	2
2	Hold water in basin	Closed	Open turbines	$H > H_{start}$	3
3	Empty basin	Running	Close turbines	$z = z_{min}$ or $H < H_{stop}$	4
4	Hold water in basin	Closed	Open turbines	$H < H_{start}$	1

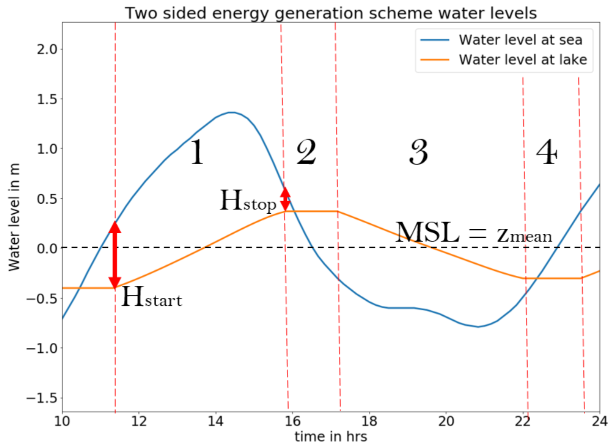


Figure 5.3: Two sided energy generation scheme, no pump

5.1.2. Tidal power station including pumps

When sea level rises the time window for emptying the basin is decreased, this is because mean low water level at sea is moved up, closer to the mean water level of the lake. The head difference during the interval where the basin is supposed to be emptied is thus smaller, and H_{start} is reached later or not at all, while H_{stop} is reached sooner. This means that the basin is not fully emptied and thus the tidal stroke at the lake becomes smaller.

To counter this effect pumps can be used. The pumps are initiated when the water level at sea reaches a critical value at low tide (h_{ebb}) and the basin is not emptied naturally. If during pumping the head difference between sea and lake is such that the lake can be emptied through natural outflow, the pumps are switched to turbines. Pumping stops either when the target minimum water level at the lake is reached or when the water level at the sea rises beyond a critical point (h_{flood}). This is done to prevent the pumps from pumping during high tide, where they become less efficient and the tidal stroke at the lake becomes out of phase with the tide at sea. This would lead to irregular filling and emptying of the lake, reducing the number of tides occurring at the lake annually.

If during emptying of the lake through natural outflow H_{stop} is reached while the minimum target water level at the lake has not yet been reached and the critical water level at sea (h_{flood}) has also not been reached, the turbines switch to pumping.

Table 5.2 summarizes the phases and transition conditions for this situation where pumps are needed because of a rise in sea level. Figure 5.4 shows three lines, the water level at sea and the water level in the lake with and without pumps. It can clearly be observed how for the orange line (no pump) phase two, waiting, is long (about 4 hours) and phase 3, emptying, is shorter (2 hours). In figure 5.3 phase 2, waiting, lasted 2 hours while phase 3, emptying, lasted 4 hours. As a result the minimum water level reached by the orange line is higher during this cycle. The green line represents the scheme when pumping is applied, and the different phases as defined by table 5.2 are indicated between the red line intervals and relate only to the green line.

Table 5.2: Two sided energy generation scheme, with pump

Phase	Operation	Turbines/ Pumps	Action at end operation	Transition conditions	Next phase
1	Fill basin	Turbine Running	Close gate stop turbine	$z = z_{max}$ or $H > H_{stop}$	2
2	Hold water in basin	Closed	Open gate start turbine Open gate start pump	$H > H_{start}$ $h < h_{ebb}$	3 5
3	Empty basin	Turbine Running	Close gate stop turbine Stop turbine start pumping	$z = z_{min}$ or $H < H_{stop}$ and $h > h_{flood}$ $H < H_{stop}$ and $h < h_{flood}$	4 5
4	Hold water in basin	Closed	Open gate start turbine	$H < H_{start}$	1
5	Empty basin with pumps	Pump Running	Stop pump start turbine Close gate stop pump	$H > H_{start}$ $z = z_{min}$ or $h > h_{flood}$	3 4

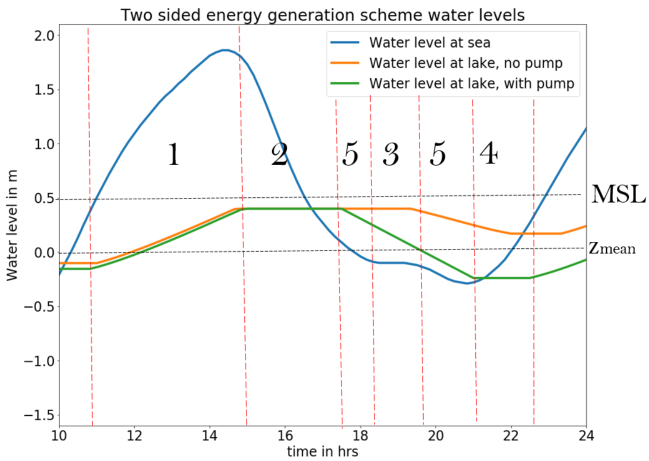


Figure 5.4: Two sided energy generation scheme, with pump

5.1.3. Workflow hydro energetic cost model

By running the model for increasing sea level rise, the annual tidal stroke, the sum of energy (+ turbine and - pump), the lifetime of the station and the costs can be determined. The moment the station is no longer able to maintain the minimum tidal stroke is the end of the lifetime of the solution. The cost are made up from the cost of the civil works including labour, costs of the pump/turbines, operation and maintenance costs and the profit from the energy yield and the cost for the pump energy. The costs can thus only be calculated when the other elements (energy sum and lifetime) are known.

Figure 5.5 illustrates the workflow of the computer model. Multiple tidal signals are created by adding sea level rise. "Loop I: no pumps" loops through the information and calculates the tidal stroke and energy yield for each tidal signal, using the scheme of table 5.1. The annual tidal stroke for each tidal signal with sea level rise is then compared to the minimum tidal stroke. If the minimum tidal stroke is met, the energy yield and tidal stroke for that level of sea level rise are stored in the final results. For signals where the tidal stroke is less than the minimum, "Loop II, pumps" is initiated, which loops through those signals using the scheme of table 5.2, and these final results are stored. The energy yield, pumping energy and tidal stroke for a range of sea level rise is now known. For a desired sea level rise scenario (e.g. RCP26) the lifetime of the solution is calculated. Using all this information the costs are calculated.

The choice to run both schemes with and without pumps is made to give insight into the added value of the pumps. The increase in lifetime and energy yield can easily be determined by comparing the results from the two loops. To prevent unnecessary extra calculations, tidal signals where no pumps are needed are skipped in the second loop.

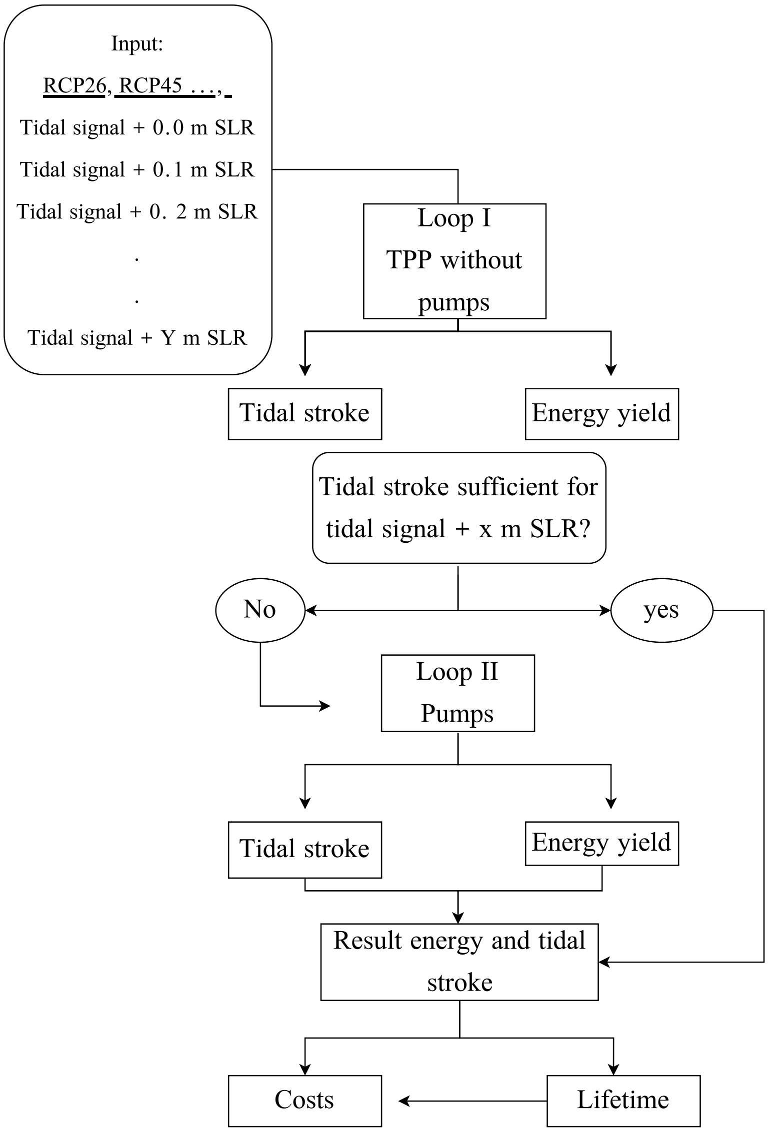


Figure 5.5: Workflow computer model

5.2. Schematization

In this section the schematization and relations used in the model are described. These are needed to describe how the water level in the lake changes, what the discharge through the turbines and pumps will be at a certain head difference and what the consequent energy yield will be. It is also described how the cost are related to a few basic parameters of the design such as turbine size.

5.2.1. Tidal signal

The tidal signal used in the model is derived from the Delft3D toolbox tide database. As input the the tidal signal from 2019 is used. The water level is recorded every 10 minutes, the signal consists of 365 * 24 * 6 = 52560 data points.

It is assumed that the tidal signal is not altered by the placement of the dam. This is a simplification as in reality the tidal signal is in part shaped by the geometry of a coastline or basin.

By shifting the tidal signal upwards with x meters, sea level rise is modelled. This is also a simplification as the shape of the tidal signal will also be partly modified by the rise of sea level. Pickering et al. (2017)

5.2.2. Lake

It is assumed that the water level in the lake rises uniformly. Without the presence of a barrage this assumption would be correct for 'short basins' i.e. their length being much smaller than $\frac{1}{4}$ of the tidal wave. The presence of a tidal power station this causes constriction of the flow. It is important to check for each location if this assumption is valid. When the water level rises uniformly in the basin, equation 5.2 can be used to establish the relationship between discharge, surface area and water level change.

$$Q = S\frac{dz}{dt} \tag{5.2}$$

With Q the discharge in m^3/s
With S the surface area of the lake in m^2

The yearly tidal stroke at the lake is calculated by the model with equation 5.3. The model calculates for each tidal cycle the difference between high and low water at the lake, and divides this by the number of tides at sea. When the number of tides at the lake reduce, the amplitude of the tidal stroke at the lake for a certain tidal cycle can remain the same while the yearly tidal stroke at the lake reduces.

$$R_{\text{lake}} = \frac{1}{N_{\text{sea}}}\sum_{n=1}^{N_{\text{lake}}}(z_{\text{high}\,n} - z_{\text{low}\,n}) \tag{5.3}$$

With R_{lake} the yearly average tidal stroke at the lake in meters
With N_{sea} the yearly number of tides at sea
With $z_{\text{high}\,n}$ the highest water level in meters at lake during tidal cycle n
With $z_{\text{low}\,n}$ the lowest water level in meters at lake during tidal cycle n

5.2.3. Turbines

Efficiency

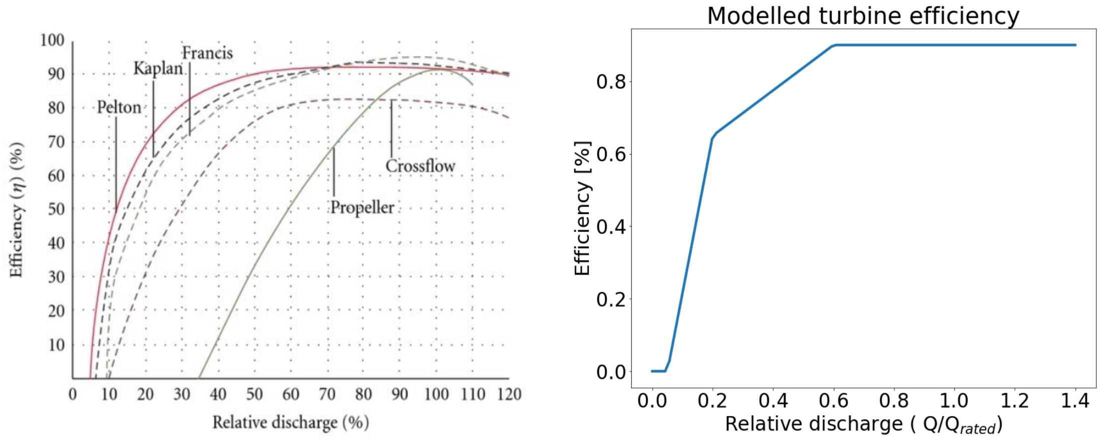
As the water flows through the turbines, the blades start running and electricity is generated by storing this kinetic energy. Turbines are not 100% efficient in harvesting all kinetic energy from the water. Instead some energy will get dissipated during the process. A hydro turbine works best at its design discharge (Q_{rated}). When the discharge is lower the efficiency also decreases. This is illustrated in figure 5.6a.

At the tidal power station the discharge will not be constant over one cycle as the head difference changes. Efficiency therefore will also not be constant. Turbines normally used for this type of application are bulb turbines, a special type of Kaplan turbine. More information on turbine types is found in appendix C. The curve for the Kaplan turbine will be used as a basis for the schematization of the turbine efficiency. In figure 5.6a it can be seen that the efficiency has a constant value of around 90% between a relative discharge (Q/Q_{rated}) of 0.7 and 1.2 below a relative discharge 0.7 the efficiency decreases. It is chosen to schematize the efficiency of the turbine by dividing the efficiency into three linear domains, this is illustrated in figure 5.6b.

Design head and design discharge

The relationship for the rated head and rated discharge were derived using data the river central at Alphen aan de Maas (Haskoning DHV, 2010). With the scaling law in equation 5.4 the turbine design discharge can be determined. With Q_1 = 100m^3/s, D_1 = 4 meters and H_{rated}= 4 meters, the expression of the design discharge is given in equation 5.5.

$$\frac{Q_{\text{rated}_1}}{Q_{\text{rated}_2}} = \frac{\sqrt{H_{\text{rated}_1}}}{\sqrt{H_{\text{rated}_2}}}\frac{D_1^2}{D_2^2} \tag{5.4}$$



(a) Efficiency turbine types for relative discharge. (From Kaunda et al. (2012))

(b) Schematized turbine efficiency for relative discharge

The rated head (H_{rated}) and the rated discharge (Q_{rated}) are coupled with equation 5.5.

$$Q_{\text{rated}} = 3.125\sqrt{H_{\text{rated}}}D^2 \tag{5.5}$$

With D the runner diameter in meters

Discharge through turbines

The discharge through the turbines is not constant over the cycle, and can be divided into two domains. If H < H_{rated} equation 5.6 applies. If H > H_{rated} then equation 5.7 applies. Haskoning DHV (2010)

$$Q = nQ_d\sqrt{\frac{H}{H_{\text{rated}}}} \tag{5.6}$$

$$Q = nQ_{\text{rated}} * \frac{H_{\text{rated}}}{H} \tag{5.7}$$

With Q the discharge in m^3/s
With n the number of turbines
With η the turbine efficiency

This is illustrated in figure 5.6. It can be seen that the discharge is maximum at the rated discharge. When the head difference is below the $H_{\text{start}}$the discharge will be zero. In the figure this is illustrated by the zero discharge until $H/H_{\text{rated}} = 0.4$.

Energy generation

The maximum (installed) capacity of the turbines is at its design discharge with maximum efficiency. This can be calculated with equation 5.8

$$P_{\text{max}} = \eta_{\text{max}}\rho g Q_{\text{rated}} H_{\text{rated}} \tag{5.8}$$

The discharge at any point in time and the total energy yield can be calculated with the equations below. It becomes clear that there are two regimes depending on the ratio of $H_{\text{rated}}$and H.

$$P(t) = \begin{cases} n\eta\rho g Q(t)H(t) & \text{if H < H}_{\text{rated}} \\ nP_{\text{max}} & \text{if H > H}_{\text{rated}} \end{cases}$$

The energy yield is found with equation 5.9.

$$E = \int P(t)dt \tag{5.9}$$

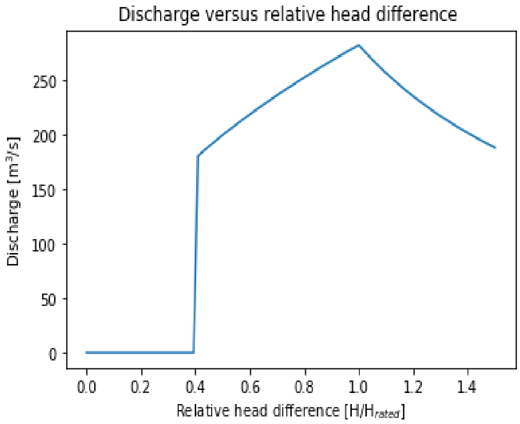


Figure 5.6: Discharge versus relative head, maximum is at $H/H_{rated} = 1$

With $P_{max}$the power at the maximum installed turbine capacity in W
with P(t) the total power at a point in time in W
with E the energy yield in J
With η the turbine efficiency [-]
With g the gravitational constant in m^2/s
With Q(t) discharge in m^3/s in time
With H(t) the head difference in meters in time

5.2.4. Pumps

The discharge through the pump can be described by the performance curve and system curve.

A pump has specific properties that can be described by a performance curve. This curve describes the maximum discharge that can be achieved by the pump for a certain head difference. This curve limits the maximum amount of work that can be done at each moment by the pump. The performance curve is normally supplied by the manufacturer.

Very high pump discharges are needed to regulate the water level at the basin. No reference projects were found for this type of high-discharge pump and thus no performance curves can be used directly. An analysis of performance curves and other pump properties are given in appendix C. The schematization of the pumps is described in the following paragraphs.

The amount of water that the pump can move is limited by its capacity. The capacity of the pump is related to the turbine capacity, as it uses the same mechanical driving mechanisms. Using the turbine capacity as the pump capacity will be the starting point. The discharge at each head difference can then be calculated with equation 5.10.

$$Q = \frac{P_p \eta}{\rho g H} \tag{5.10}$$

With P_p the capacity of the pumps

In theory as the head difference approaches zero an infinite discharge can be achieved, this is not realistic in real life. A maximum discharge is used as given in equation 5.11.

$$Q_{\text{pump}\max} = 1.2 Q_{\text{rated}} \tag{5.11}$$

The installed capacity of the turbines is the maximum capacity at its efficiency point. For a large part of the cycle this optimal capacity will not be achieved. The same is true for the pumps. In the schematization of the pump it is chosen to use an averaged value for the pump capacity and this is 20% lower than the maximum capacity of the turbines. The efficiency also has an average value, as the influence of a varying efficiency is minimal on the performance curve (appendix C). An efficiency of 75% is used.

The system curve describes the properties of the whole system in which the pump is installed. As the water is pumped through the system energy is lost due to dissipation. With increasing flow rate more energy is dissipated. The head difference that needs to be overcome by the pump to move the water is increased due to this dissipated energy, this is called the dynamic head. Appendix C gives more information about the dynamic head. By calculating the dynamic head for a range of discharges the system curve is determined. This curve is unique to the properties of the system under evaluation and shows the dynamic head at a discharge rate.

The intersection of the system curve with the performance curve of the pump gives the discharge of the pump. This is illustrated in figure C.1e

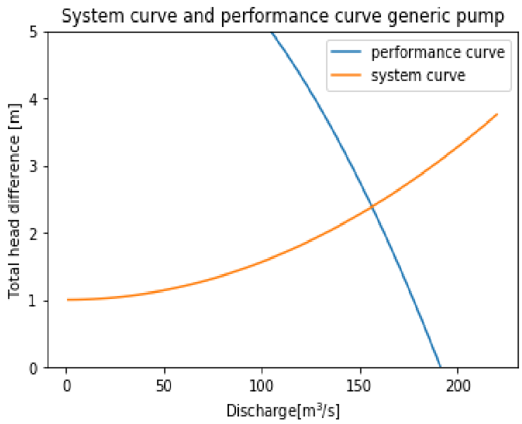


Figure 5.7: System and performance curve for generic system and pump

At a tidal power plant the tide is continuously changing and with it the head difference also changes. This means that the static head changes, but the system curve remains the same. This gives a range of discharges under which the pump can operate, the operational range.

The power that is needed to pump water follows directly from the intersection point of the system curve and performance curve. The total pump energy used can be calculated with equation 5.9.

5.2.5. Costs

The costs of the project are made up of many elements, and run on different time scales. The model can calculate the costs of the tidal power station with pumping capacity. The other elements of the solution i.e. the rest of the dam and the lock are not included in the hydro energetic cost model. These are determined separately in chapter 8. Below the elements taken into consideration in the cost model are listed.

- Location
- Cost of materials
  - Concrete powerhouse
  - Reinforcement steel powerhouse
  - Steel Gates
  - Bed protection
- Pump/turbines
- Construction costs including labour and overhead costs
- Maintenance and operational costs
- Energy yield profit

Location index

The location cost index relates construction costs in different country to a fixed frame of reference. In this way the costs of a construction project in China can be 'translated' to Dutch prices with such an index. Arcadis provides such numbers each year in a study which compares construction for many cities to London as a baseline. (Arcadis, 2019). The table below gives the values that are relevant for the chosen locations. Some of the cities are included in the list, in which case the index will be used directly. Some cities are not included in the list, in which case the value for a city in the same country is chosen. For North and South Korea the value

Table 5.3: Location index for comparing international construction costs

Location	Location index
Netherlands: Rotterdam	1
UK: Liverpool	1.14
US: San Francisco	1.64
North/South Korea: Seoul	1
China: Shenzen	0.43
India: Mumbai	0.36

for Seoul is chosen, as there is no information on North Korea. All values are transformed so that Rotterdam (closest to the Brouwersdam) is the new baseline.

It is assumed that while the construction costs will vary per location, the costs of the turbines will not due to the limited amount of manufacturers.

Cost of materials

The volume of concrete needed for the powerhouse is given in equation 5.12 and was derived by Fay and Smachlo (1983). They assume that the length of the powerhouse (in flow direction) is proportional to the tidal range, the width (perpendicular to flow direction) is proportional to the turbine flow area, and the number of turbines. The height of the dam is proportional to both. Concrete used for the powerhouse was assumed to be a fixed fraction of the total volume of the powerhouse. This fixed fraction was derived by studying several reference projects.

$$V_p = 42nRD_t^2 \tag{5.12}$$

With V_p the volume of concrete in the powerhouse
With n the number of turbines
With R the tidal range
With D_t the diameter of the turbine

The costs of the concrete for the powerhouse are then calculated with equation

$$C_p = V_p C_{\text{concrete}} \tag{5.13}$$

With C_p the cost of concrete in the powerhouse in euros
With C_{concrete} the price of concrete in euros

It is assumed that the cost of the reinforcement steel in the powerhouse, the gates and bed protection are proportional to the cost of the concrete of the powerhouse. This is a reasonable assumption as the reinforcement steel is a percentage of the concrete in the powerhouse, the size of the powerhouse is determined by the number of turbines, as is the number of gates, furthermore the size of the gates and the powerhouse will be linked. Lastly, the size of the powerhouse is related to the currents and waves that it is exposed to, which relates to the bed protection. The cost of the reinforcement steel is estimated to be approximately the same as the cost of the concrete. The costs of the gates is estimated to be approximately 3 times as large. The cost of the bed protection was estimated at approximately 1.5 times the cost of concrete of the powerhouse. The total material costs are given in equation 5.2.5.

$$C_{\text{mat}} = C_p + C_r + C_g + C_b = 7.5Cp \ = 7.5(42nRD_t^2 C_{\text{concrete}})$$

(5.14)

With C_r the cost of the reinforcement in euros
With C_g the cost of the gates in euros
With C_b the the cost of the bed protection in euros
With C_{mat} the total material cost in euros

Costs of pump turbines
Swane (2007) developed a expression to estimate the costs for bulb turbines in a tidal power station. The costs according to Swane are proportional to the head difference and turbine size. This relationship is shown in equation 5.15 The relationship was established by using reference projects in China and South Korea and shows good back casting potential.

$$C_t = 0.7(5.5 \times 10^6 + 118500 H^{0.18} n D_t^2) \tag{5.15}$$

With C_t the cost of the turbines in euros
With n the number of turbines

Gilles et al. (2008) also developed a relation to estimate the costs of turbines, and related this to the installed capacity and rated head. For turbines with a larger capacity he found the relationship as described in equation 5.16

$$C_t = 10646 n P_t^{0.7} H^{-0.26} \tag{5.16}$$

With P_t the capacity of the turbine in kW

These relations for turbine costs are compared to the estimated costs for the Brouwersdam turbines. (Grevelingen, 2019). A correction for inflation needs to be added to convert the value in 2008 to 2019, the year of the cost estimation for the turbines in the Brouwersdam. The inflation rate during this period was approximately 14%. The results are given in table 5.4. It can be seen that the relation found with Swane is very close to the predicted value for the turbine costs of the Brouwersdam. The relationship found by Swane will be used to predict the turbine cost.

Table 5.4: Predictive capacity two cost estimation methods for Brouwersdam case study

	Swane	**Gilles**	**Estimated**
Price (million euros)	73.2	39.7	67.8
Error [%]	3	44.6	-

Construction costs
The additional costs of the project include overhead costs, construction equipment, man hours and maintenance work, these are proportional to the size of the project. It is assumed that the cost of the costs of concrete, reinforcement, gates and hydraulic equipment (excluding pump/turbines) was 50% of the total project costs.

$$C_{\text{add}} = C_{\text{mat}} \tag{5.17}$$

With C_{add} the additional project cost in euros

Operational and Maintenance cost
An assumption has to be made about the operational and maintenance costs in order to establish the annual cash flows. It is chosen to express the operational and maintenance as a percentage of the investment costs of the structure. The investment costs are split up into civil works and pump/turbines. For the civil engineering structure annual operational and maintenance costs are estimated as 1.2%. For the pump turbines this is 2.5%.

The total maintenance cost over the lifetime is then:

$$C_{\text{mo,c}} = \sum_{t=1}^{T_{life}} 0.012 C_{\text{init}} \tag{5.18}$$

$$C_{\text{mo,t}} = \sum_{t=1}^{T_{life}} 0.025 C_{\text{init}} \tag{5.19}$$

With $C_{\text{mo,c}}$ the maintenance and operational costs of the civil works in euros
With $C_{\text{mo,t}}$ the maintenace and operation costs of the turbines in euros
With C_{init} the costs of the initial investment in euros

Energy yield profit

The price of energy is expressed as a unit price per GWh.

$$C_e = 51 \times 10^3 E \tag{5.20}$$

With C_e the energy cost or profit in euros
With E the amount of energy in GWh

Mitigation costs

The mitigation costs are related to the compensation for damages to nature (equation 5.21 and infrastructure (equation 5.22. The derivation of the mitigation costs are discussed in more detail in appendix A. The mitigation costs for nature are based on the cost estimation for the Brouwersdam, and are linked to the change in the prism of water moving through the barrier.

$$C_{\text{nature}} = |R_1 - R_2| S 50000 \tag{5.21}$$

With C_{nature} the compensation cost for nature in euros
With R_1 the initial tidal stroke at the lake in meters
With R_2 the altered tidal stroke at the lake in meters
With S the surface area of the lake in m^2

The mitigation costs for infrastructure around the basin were also based on the Brouwersdam cost estimation and is linked to the surface area around the perimeter of the basin that is affected by the new tidal stroke.

$$C_{\text{infra}} = |R_1 - R_2| p600 \tag{5.22}$$

With C_{infra} the compensation cost for infrastructure in euros
With p the perimeter of the lake in meters

Total costs

The total costs are a sum of all costs, scaled to the location index. The turbine costs are not scaled, this is because of the limited amount of manufacturers worldwide.

$$C_{\text{total}} = (2C_{\text{mat}} + C_{\text{mo, c}} + C_{\text{mo, t}} + C_{\text{infra}} + C_{\text{nature}}) I + C_{\text{t}} \tag{5.23}$$

with I the location index [-]

5.3. Validation model with current Brouwersdam design

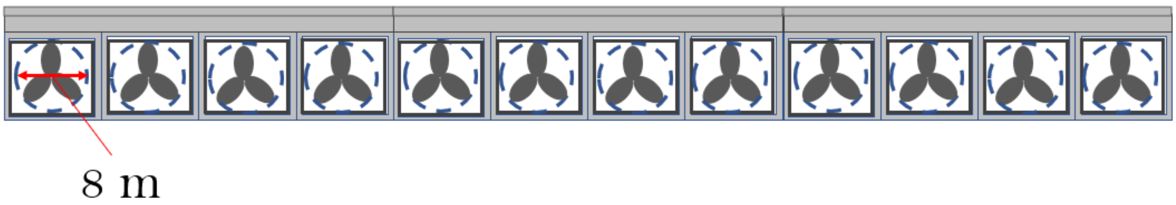


Figure 5.8: Design Brouwersdam: 12 pump/turbines with 8 meters diameter

The document Grevelingen (2019) is used to compare the results from the model to previous studies about the tidal/pump station.

Table 5.5: Input model Brouwersdam

Input Brouwersdam			
Area lake [km2]	110	Number of turbines [-]	12
MTR [m]	2.3	Turbine diameter [m]	8
zmin [m]	-0.2	Hrated [m]	1.5
zmax [m]	0.2	Hstart [m]	0.45
Minimum tidal stroke [m]	0.35	Hstop [m]	0.05

5.3.1. Tidal stroke and lifetime

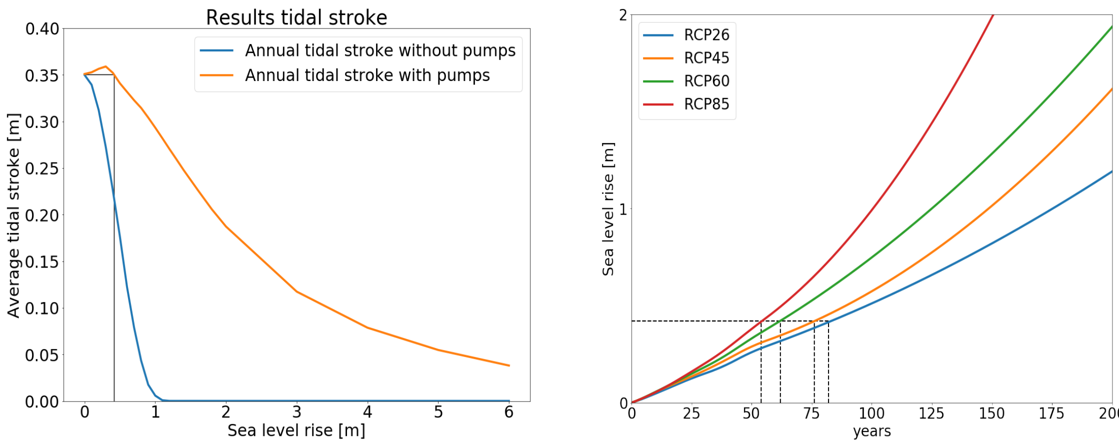
Results model

Figure 5.9a shows the result of the annual tidal stroke at the Grevelingenlake for increasing sea level rise as calculated by the model. The situation without pumps results in an initial tidal stroke of 0.345 m, which is very close to the target value. Considering the accuracy of the model it is said that both with and without pumps the minimum tidal stroke is initially met. The point of failure for the situation with pumps occurs between 0 and 5 cm sea level rise. As sea level rise increases, the stroke rapidly decreases for the situation without pumps.

For the situation with pumps, the decrease of the tidal stroke with increasing sea level is slowed down. The solution 'fails' when the tidal stroke can no longer be maintained. If the tidal stroke drops below the minimum the water quality is no longer of a sufficient standard. The point of failure, when the average tidal stroke drops below 0.35 m is after 42 centimeters of sea level rise. Depending on which sea level rise scenario is looked at this corresponds to different lifetimes. This is illustrated by figure 5.9b. The respective lifetimes are given in table 5.6. The pumps are effective in increasing the lifetime of the structure, although all lifetimes are far below 100 years.

Table 5.6

	Lifetime without pumps [years]	With pumps [years]
RCP26	0 - 12	83
RCP45	0 - 12	76
RCP60	0 - 12	62
RCP85	0 - 12	54



(a) Average annual tidal stroke at Grevelingenlake with and without pump for increasing sea level rise)

(b) Discharge through pump/turbines at 0 meter sea level rise

Figure 5.9: Tidal stroke and lifetime Brouwersdam

Table 5.7: Results previous study Grevelingen (2019) versus model results

	Previous study	Hydro energetic model
SLR at failure without pumps [cm]	7	0-5
SLR at failure with pumps [cm]	28	42

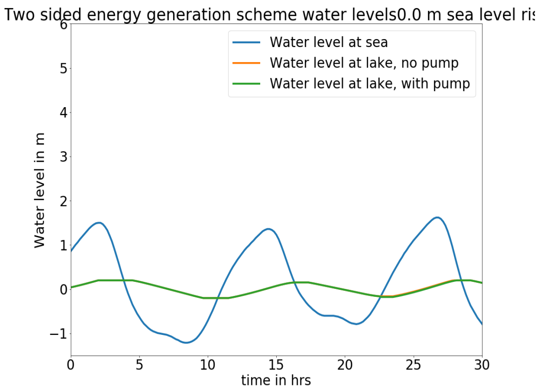
Comparison Brouwersdam project

Table 5.7 summarizes the results from the hydro energetic cost model and the results according to the previous study on the Brouwersdam Grevelingen (2019). For the situation without pumps the results are in relative agreement, while for the situation with pumps the hydro energetic cost model is more optimistic than the previous study. Most likely the difference can be explained by the fact that the previous study only takes into account pumps used for flushing (i.e. the water level is lower in the direction the pumps are pumping the water), while the model takes into account both flushing and pumping against a head difference. With this limitation removed the lifetime is increased.

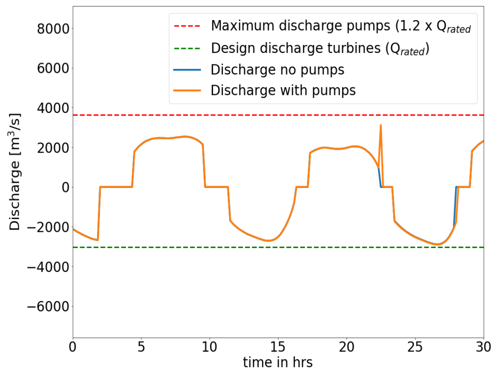
5.3.2. Water levels and discharge

Results model

For a few points from figure 5.9a the water levels and discharge through the pump and turbines are shown to illustrate the working of the model with and without pumps. The situations range from the current situation to extreme levels: 0.0 m, 0.3 m, 1 m and 4 meters.



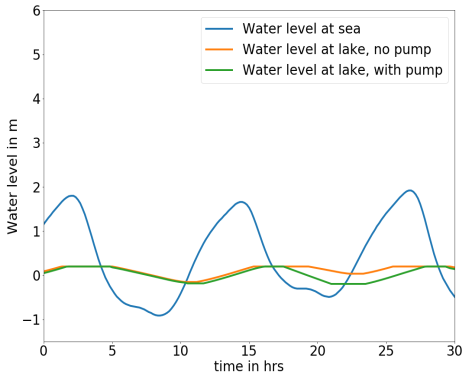
(a) Water level at sea and at Grevelingenlake at 0 meter sea level rise)



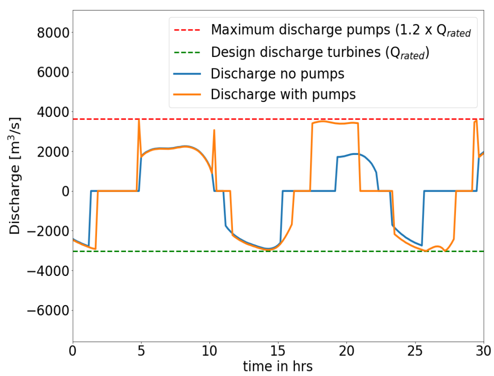
(b) Discharge through pump/turbines at 0 meter sea level rise

Figure 5.10a show the water level change at the Grevelingenlake for 0 meters sea level rise. The lines representing the situation with and without pump almost completely coincide, meaning almost no pumping is necessary. Figure 5.10b shows the discharge through the pump/turbines over this period of time. A positive discharge means emptying of the basin (water flows from the lake to the sea) and a negative discharge means filling of the basin (water flows from the sea to the lake). The maximum discharge that can be achieved in pumping mode is indicated in the figure as well as the maximum discharge that can be achieved in turbining mode. A little peak is observed at the end of emptying the basin on the second cycle, this means that for a short period of time the pumps were used to reach the target water level at the lake. In table 5.2 this means a switch from phase 3 to phase 5, with the corresponding transition conditions occurring.

Figure 5.10c show the water level change at the Grevelingenlake for 0.3 meters sea level rise. Here a difference between the situation with and without pumps is observable. Where the orange line, without pumps, skips a tidal cycle. Figure 5.10d shows the discharge during this period of time. The basin is emptied partly by pumps and partly by natural outflow. This can be nicely observed in the first tidal cycle where first pumps are used, when the head difference is large enough to allow natural outflow the orange and blue line overlap, at the end of the cycles the pumps are again used to empty the basin until the desired level. For the second cycle the basin is emptied by the pumps alone. In figure 5.9a, it can be seen that for 0.3 meters sea level rise, the tidal stroke is slightly higher than for 0 meters sea level rise, which can be explained by the

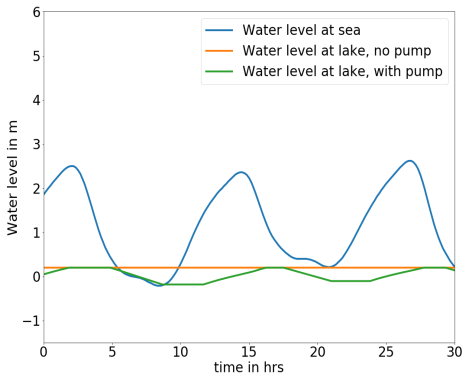


(c) Water level at sea and at Grevelingenlake at 0.3 meter sea level rise)

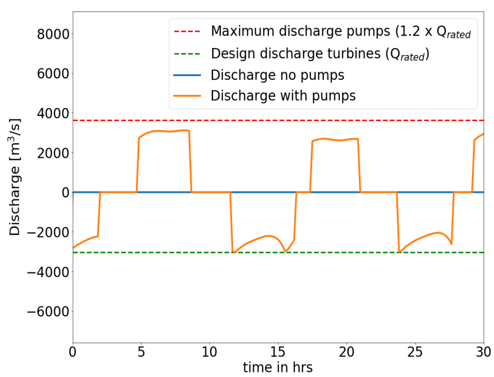


(d) Discharge through pump/turbines at 0.3 meter sea level rise

higher discharges during both in and outflow. The transition condition to continue to the next cycle is then more often that the minimum or maximum water level is reached instead of the head differences no longer being sufficient or a critical water level at sea has been reached. (see table 5.2)



(e) Water level at sea and at Grevelingenlake at 1 meter sea level rise)



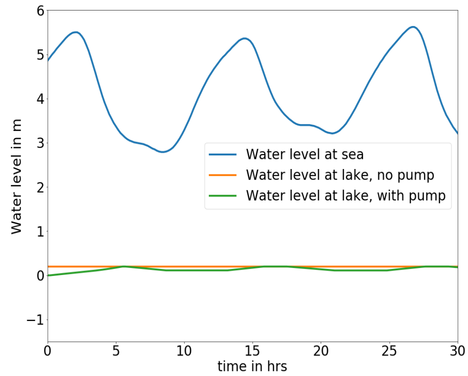
(f) Discharge through pump/turbines at 1 meter sea level rise

Figure 5.10e show the water level change at the Grevelingenlake for 1 meter sea level rise. Without pumps there is no tidal stroke at the lake, which can also be seen in figure 5.9a. With pumps the a tidal stroke is observed. Figure 5.10f shows the discharge during this period of time. The basin is now emptied by the pumps alone, the head difference at this point is such that the pumps can still achieve their maximum discharge. As was explained in subsection 5.2.3, when the relative head difference (H/H_{rated}) exceeds 1 the discharge through the turbines decreases, which is also demonstrated in the figure. 1 meter sea level rise corresponds to the end of the plateau observed in figure 5.9a.

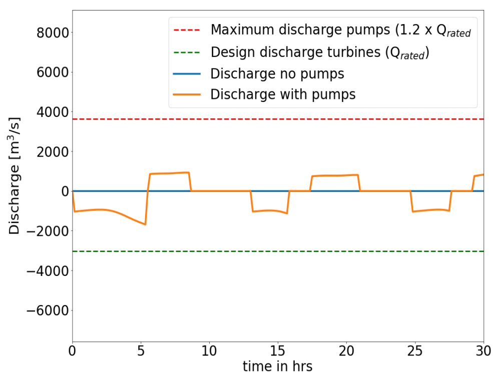
Figure 5.10g show the water level change at the Grevelingenlake for 4 meters sea level rise. With pumps a very small tidal stroke is observed. Figure 5.10h shows the discharge during this period of time. The basin is emptied by the pumps alone, the head difference during low water at sea at this point is such that the pumps can no longer achieve their maximum discharge. The basin is only partly emptied during the cycle. The relative head difference is also large resulting in a low discharge also during filling of the basin.

Comparison to Brouwersdam Project

As in the Brouwersdam project specified in Grevelingen (2019) the water level fluctuates between the minimum and maximum allowable water level. No numbers on discharges through turbines where specified in the document.



(g) Water level at sea and at Grevelingenlake at 4 meter sea level rise)



(h) Discharge through pump/turbines at 4 meter sea level rise

Figure 5.10: Water level and discharge Brouwersdam

5.3.3. Energy yield and pump energy required

Results model

Figure 5.11a shows the energy yield with and without pump, and the energy needed to drive the pumps. It can be seen that the energy with and without pumps starts at the same yield. When no pumping is applied the yield rapidly decreases as the sea level rises, similar to the tidal stroke. When discharging occurs over less tidal cycles and for a shorter period of time, naturally the energy yield will be less.

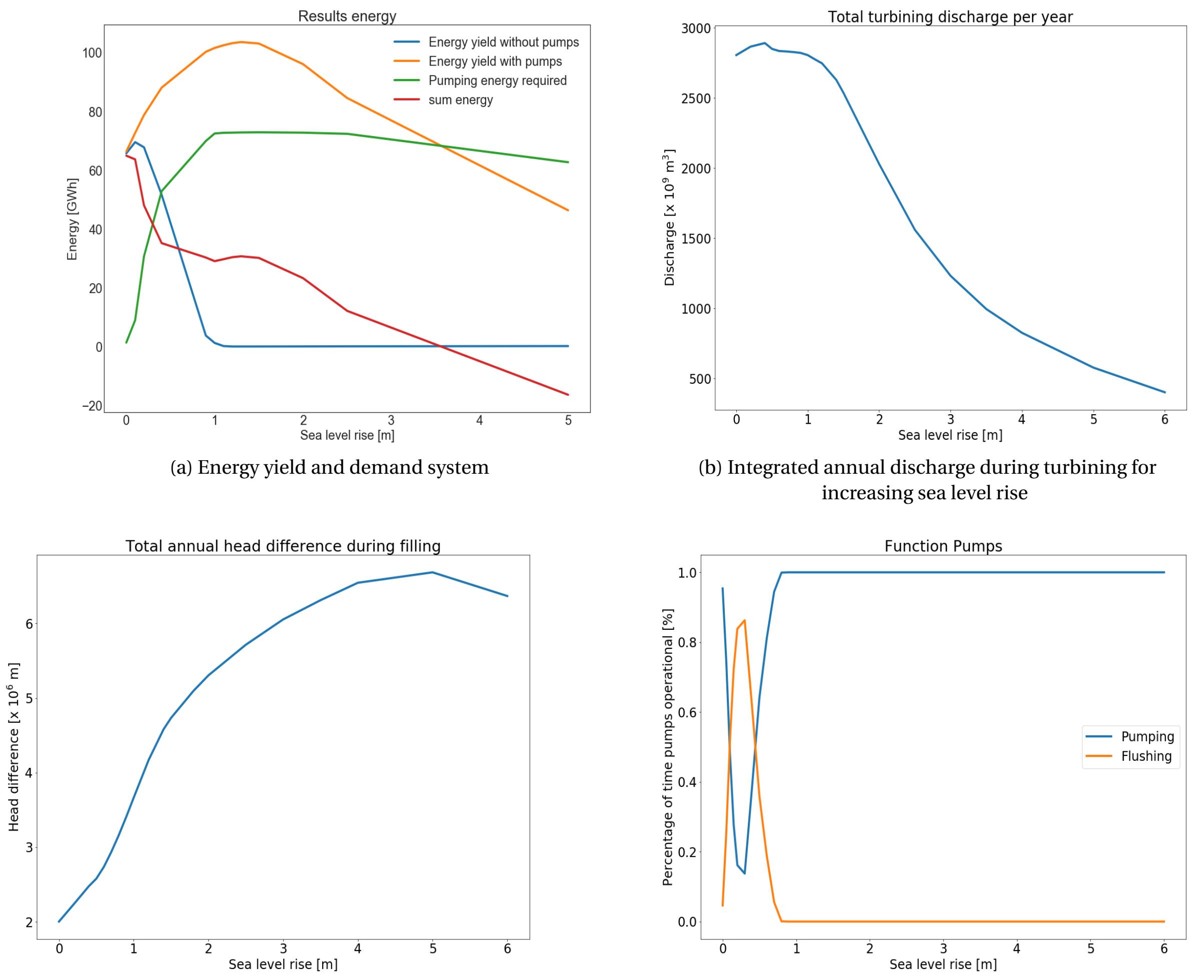
The energy yield when pumping is applied initially increases as sea level rises. Figure 5.11b shows the result of integrating the discharge over time for an entire year, for increasing rate of sea level rise. From 0 to 1.3 meters the total annual discharge through the turbines is relatively constant and high, after which it starts to decrease. Beyond this point the relative head difference is such that the discharge through the turbines decreases. Figure 5.11c shows the result of integrating the head difference during turbining during one year for increasing rate of sea level rise. As to be expected with increasing sea level rise the head difference also increases, the rate of increase is not constant due to the fact that for larger levels of sea level rise during less cycles the basin is filled. As the energy yield is proportional to $Q \times H$ the combination of figure 5.11b and 5.11b results in the shape of the energy yield curve with pumps observed in figure 5.11a.

The pumping energy required increases until approximately 1 meter. The slope is not constant during this interval, and this can be explained be explained by the transition from flushing to pumping mode. Initially the pumps are only used for a very small portion of time to achieve the desired water level (the spike in figure 5.10b). Then as the pumps are used for a larger part of the cycle, they are used for flushing (i.e. the water level is lower in the direction the pumps are pumping the water). After a certain level of sea level rise the water level at sea is always above the water level at the lake and only pumping occurs. This is illustrated in figure 5.11d. The pumps are operating at maximum capacity after this point, which is why there is no further increase in the pump energy. A slight decrease in energy is caused by the fact that the basin takes an increasingly long time to fill up, leaving less time for pumping. Similarly to the energy yield the pumping energy required is also proportional to the total discharge and head difference, resulting in the green curve. Up to 3 meters sea level rise there is a positive energy balance, i.e. the system produces more energy than it takes in.

Table 5.8 summarizes the average annual energy yield over the lifetime with pumps and the average energy required by the pumps over the lifetime.

Table 5.8

	Average annual energy yield over lifetime [GWh]	**Average annual pumping energy over lifetime [GWh]**
RCP26	78.7	28.5
RCP45	79.0	29.0
RCP60	77.9	26.4
RCP85	77.5	25.4



(a) Energy yield and demand system

(b) Integrated annual discharge during turbining for increasing sea level rise

(c) Integrated annual head difference during turbining for increasing sea level rise

(d) Function of pumps

Figure 5.11: Energy in system

Comparison to Brouwersdam project

According to Grevelingen (2019) the average annual energy yield over the lifetime is 50 GWh. This is smaller than the values represented in table 5.8.

The interval over which the average is taken is not the same. The previous study uses a different scenario than the RCP's, namely KNMI-G, constructed by the Royal Dutch Meteorological Institute specifically for the Netherlands. (van den Hurk et al., 2014) It is most similar to RCP45. The lifetime evaluated in the previous study ends at 0.28 cm sea level rise, while the lifetime of the system in the model ends at 0.42 meters sea level rise. As the energy yield increases annually over this period (as can be seen in figure 5.11a). This raises the average yield. However even if RCP45 is evaluated up to 0.28 meters sea level rise the average is 74.8 GWh. It can be considered to adjust the efficiency of the turbine to match the result from the report.

The difference can be explained by the definition of the discharge through the turbines. In the previous study this was done by means of an energy balance and an adjustable resistance of the turbines, while in the hydro energetic cost model the empirical relationship is used derived from data from Alphen aan de Maas. It can be considered to adjust the efficiency of the turbine to match the results of the previous study.

5.3.4. Costs

An overview of the costs for both the previous study and the hydro energetic cost model are given in table 5.9. The results for the investment costs are very similar. This makes sense since the cost calculation in the model is based on cost estimations from the study Grevelingen (2019).

The net present value (NPV) was calculated with a discount rate of 4.5%. The EEA is the NPV made independent of the project duration, thus enabling a comparison between projects of unequal lengths. The difference in results can be attributed to the different energy yields that were calculated in the report and in the model. Furthermore Grevelingen (2019) also includes subsidies as an income, which is omitted in the hydro energetic cost model.

Table 5.9

RCP45	**Previous study**	**Hydro energetic cost model**
Cost of civil structure [million euros]	108	111.5
Cost of turbines [million euros]	71.8	73.2

CHAPTER
SIX

GENERATION OF OPTIMAL DESIGNS

In chapter 4 a conceptual design was presented. While this describes the main elements of the design, it still has many degrees of freedom, such as the number of turbines and their sizes. In chapter 5, section 5.2 some turbine properties were introduced such as the design discharge H_{rated} and the starting and stopping heads ($H_{start}$and H_{stop}). In the same chapter the costs and profits of the project are related to these parameters. The parameters listed here will be of influence on the water/nature quality, energy generation and project costs. Only for the Brouwersdam has research been done on a design of these parameters. For all other locations this needs to be generated. This chapter will elaborate on the method used to synthesize an optimal design for each location. To define optimal in this context, reference is made to chapter 2 where the relative importance of the functions of the solution is listed. Priority is thus given to the water/nature quality. An optimal design is thus defined as:

> A solution that fulfills the requirements of the minimum tidal stroke during the design lifetime at the lowest costs.

In total three design approaches will be used: short term, long term and adaptive. The short term approach has a design lifetime of 40 years and is based on the normal lifetime of the machinery. The long term approach has a design lifetime of 100 years, and thus the machinery might need replacement during its lifetime. The adaptive approach also has a design lifetime of a 100 years, but as a is able to maintain the tidal stroke without pumps in the first 15 years. This allows for the structure to be built without installing the pumps turbines right away, but rather to wait with the phasing of the pumps based on the observed rate of sea level rise. For every variant 12 designs are made: 3 desing approaches and within that design approach four climate scenarios. With a total of 27 variants this adds up to a total 324 designs.

The design is defined as the optimal combination of the number of pump/turbines, size of the pump/turbines, design discharge, $H_{start}$and H_{stop}. To this end first the influence of the parameters on the model is assessed in section 6.1. Because of the computational time of the basic model it is not possible to calculate all possible combination for every design variant. Through the parameter influence strategy insight is gained on the behaviour of the parameters and a more efficient strategy is sought. In section 6.2.2 the optimisation workflow is discussed. In the next chapter the results of the optimisation loops are analysed.

6.1. Parameter influence assessment

6.1.1. Approach

The influence of a parameter can change from one location to the next. For example a good tuning of $H_{start}$and H_{stop}can be more important at a location with a small tidal range than at a large tidal range. The number of turbines can also have more influence on the tidal stroke in a small lake versus a larger lake. For this reason first the locations are divided into distinct categories based on tidal range and lake size. Then for a representative location from each category the influence of the parameters is established.

The influence study also has a secondary goal: to gain insight on the two water management strategies 'Nature strategy' and 'Economic strategy'. Since the nature strategy will require a larger station the construction costs will be large and the mitigation costs will be small. For the economic strategy the opposite is true. By means of the analysis it can be looked into whether there are situations where the nature strategy is in fact the most economic strategy. To this end the situation both with and without mitigation costs are analyzed.

The influence study is performed by varying one parameter at a time, while the others are kept constant. This way the changes observed in the results can be attributed to the influence of the varied parameter. The input each loop are the characteristics of the representative category location, a range of values of the parameter under investigation and multiple tidal signals that simulate sea level rise. For every value of the investigated parameter the energy yield, tidal stroke and costs are calculated. The analysis of these results give insight into the behavior of the parameters in the model, their relative influence and can help in constructing an optimisation strategy. The workflow of the influence study is illustrated in figure 6.1

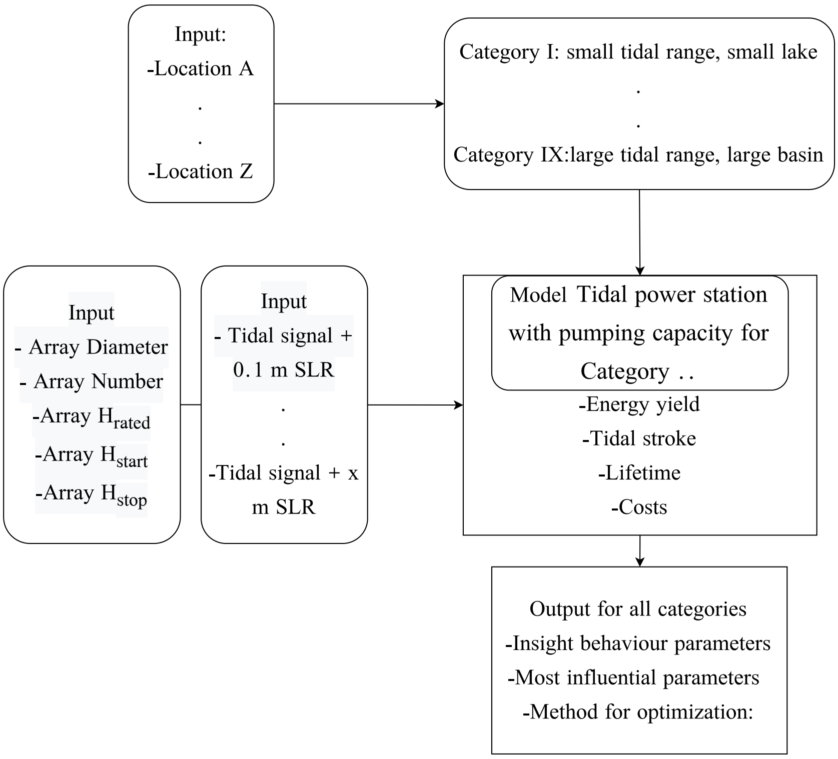


Figure 6.1: Workflow influence study

6.1.2. Results

Table 6.1 shows the nine different categories and the locations belonging to them.

The results are presented in a condensed manner. Energy yield and pumping energy are closely related, in this section only the influence on the energy yield will be discussed, as this also gives sufficient insight on pumping energy. Since the lifetime of the structure depends on what boundary is set for the minimum tidal stroke and the chosen rate of sea level rise (RCP2.6 etc) the influence of each parameter cannot directly be established. As the lifetime follows from these boundaries in combination with the tidal stroke, establishing the influence for each parameter on the tidal stroke will give sufficient insight. The influence of a parameter on the costs is given both with and without mitigation costs (nature and infrastructure as discussed in chapter 2).

Turbine number and diameter

The number of pump-turbines in combination with the pump turbine diameter turns out to be of great influence on the energy yield and tidal stroke at all locations. Together they make up the flow area, through which water can pass the barrier. The definition is given in equation 6.1.

Table 6.1: Categorization of all locations based on tidal range and basin size

Category	Tidal range [a]	Basin size [b]	Locations
I	small	small	-
II	small	medium	Dharamtar Creek B, San Francisco C, Brouwersdam
III	small	large	Delaware, San Francisco (A, B)
IV	medium	small	Mokpo (A, B), Asan Bay C
V	medium	medium	Dharamtar Creek A, Asan Bay (A, B),
VI	medium	large	Xinghua Bay
VII	large	small	Dee Estuary, Haeju Bay B
VIII	large	medium	Haeju Bay A
IX	large	large	-

[a] Tidal range: small = 0 - 2.5 m, medium = 2.5- 5 m, large = 4.5 m <
[b] Basin size: small = 0 - 100 km^2, medium = 100 - 400 km^2, large = 400 km^2 <

$$A_{\text{flow}} = \frac{\pi}{4}\pi n D^2 \tag{6.1}$$

When analysing the behaviour of the parameters n and D, it is observed that the influence of the parameters on the energy yield and tidal stroke decreases after a certain point. Both energy yield and the tidal stroke have a 'plateau phase' where a further increase in n or D does not enhance the results. This is illustrated with the Brouwersdam for 0 meters sea level rise. The turbine diameter is kept constant at 7 meters and the number is varied. With increasing number the energy yield and tidal stroke also increase. The energy yield reaches a maximum after which it starts decreasing again, this is illustrated in figure 6.2a. The tidal stroke increases until a maximum is reached, after which a plateau sets in, the flow area is large enough for the water to fluctuate between the minimum and maximum water level each tidal cycle. This is illustrated in figure 6.2b. The plateau phase is initiated for both the energy and stroke at around 17 turbines (flow area 650 m^2) When a smaller pump turbine diameter of 4 meters is used (not shown in a figure) this plateau phase is reached after 50 turbines (flow area 625 m^2). This indicates that it is not the specific value of n or D that causes the plateau phase, but the flow area. For all analysed locations this behaviour is observed, but the flow area at which a plateau phase has a large spread.

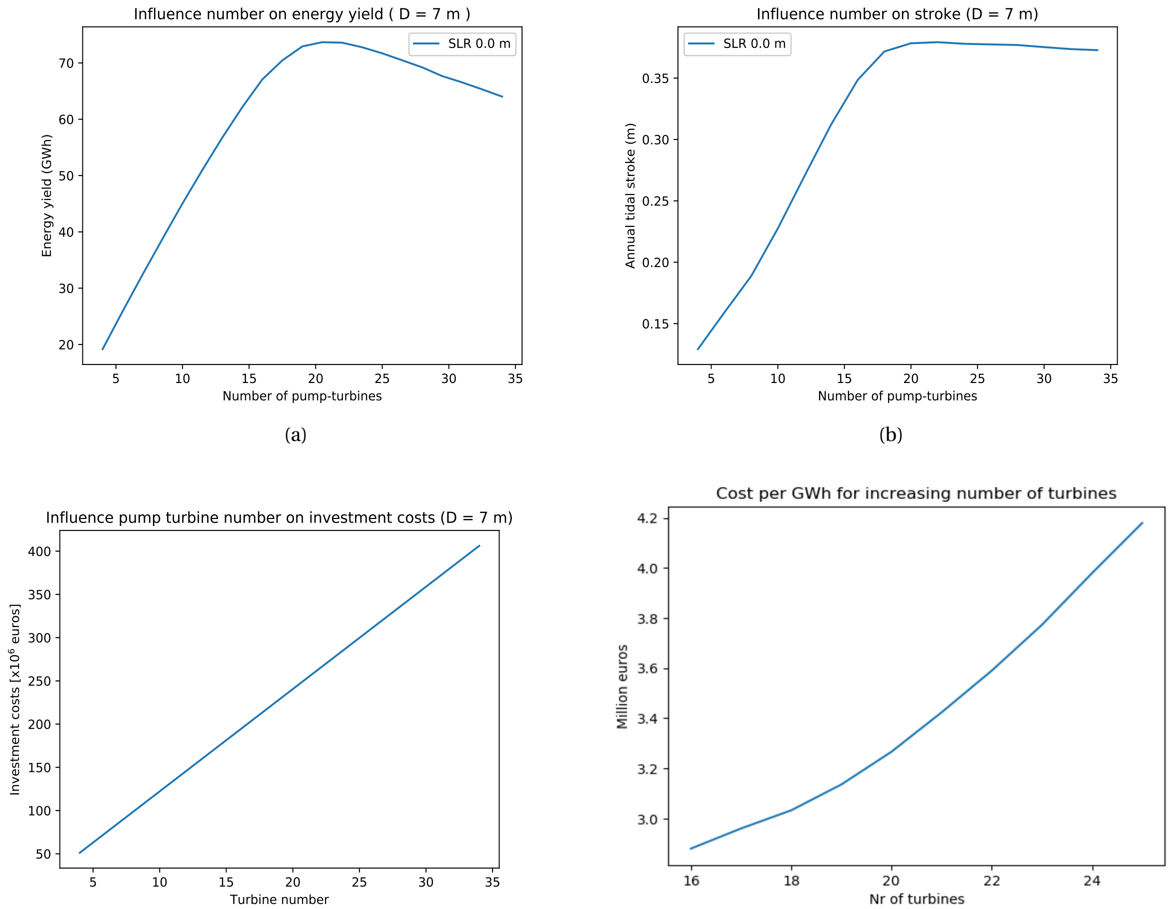
The costs of the structure is proportional to the flow area. An increase in the number of turbines with a constant diameter is illustrated in figure 6.2c. It can be seen that the project costs are very strongly linked to the number of turbines as a doubling in the amount also doubles the total costs. The increase in the diameter has a quadratic influence on the investment costs. However as the previously discussed plateau phase sets in around a certain flow area, the number of pump turbines needed reduces as the turbine diameter increases.

For the Brouwersdam the minimum tidal stroke equals 0.35 meters. This condition is met from 16 turbines and up. When plotting the costs per GWh it can be seen that the cost/GWh are lowest for 16 turbines, this is illustrated in figure 6.2d Increasing the size of the power station beyond the minimum required size will thus not lead to a higher return on the investment. The energy yield does not increase at the same rate as the costs do. It should be noted that for different locations the energy yield does not increase at the same rate, but for all locations the general trend is observed that the additional yield does not outweigh the additional investment costs.

Rated head

The influence of H_{rated}on the energy yield and tidal stroke is similar to the previously observed behaviour of n and D. For both metrics a plateau phase is observed after a certain point. No figures are included in the report as they are very similar to figure 6.2a and 6.2b. A further increase in H_{rated}will not significantly increase the tidal stroke or the energy yield. The relative influence on the cost of the project is smaller for the rated head compared to n and D. This is illustrated in figure 6.3.

Where for the pumpturbine number and diameter a large spread in possible values is found in different basins, this spread is much smaller for H_{rated}. As $H_{\text{rated}}$is the design head difference at which the pumpturbine operates at maximum efficiency, it makes sense that this should be a head difference that occurs often. For all locations it is observed that the plateau influence phase for both energy yield and the tidal stroke sets in at about 4/5th of the mean tidal range.



(a) (b) (c) (d)

Figure 6.2: Results sensitivity analysis turbine number, Brouwersdam

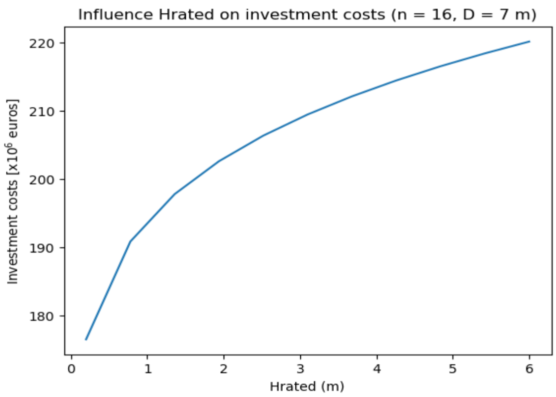


Figure 6.3: Influence H_{rated} on costs for Brouwersdam

H_{start} and H_{stop}

The influence of H_{start} and H_{stop} on the energy yield, tidal stroke and investment costs are very similar to H_{rated}. Both also show a plateau influence phase beyond a certain value at every location. For H_{start} this sets in at about 1/5th of the mean tidal range. For H_{stop} it is found that this value should be close to 0 meters.

The investment costs are not directly affected by the choice of H_{start} and H_{stop}. However as the choice for H_{start} and H_{stop} does impact the energy yield it will affect how much of the investment is earned back.

6.1.3. Conclusion sensitivity analysis

From the influence analysis it was shown that all design parameters portray the same behaviour. The influence of the parameters on the energy yield and tidal stroke decreases, the energy and tidal stroke both know a plateau phase: they cannot be increased beyond that point. The investment costs It was shown that the increase in energy yield does not keep in phase with the increase in construction costs, the costs per GWh increase as the size of the structure increases.

The pumpturbine number showed a large spread in values for different locations, and furthermore it was shown that the results for energy yield and the tidal stroke are also highly dependent on the combination of these two parameters. The spread in H_{rated}, $H_{start}$and $H_{stop}$was smaller and showed a clear link to the mean tidal range. The influence on costs for these parameters is relatively small compared to the the pumpturbine number and diameter.

Finally the influence study showed that the behaviour of the parameters are the same for all the identified location types. While the point where the influence of a parameter decreases is different for all location types, the phenomenon is observed for all location types.

6.2. optimisation model

6.2.1. Strategy

The insights gained from the parameter influence study can be used to construct an optimisation strategy. As it was established that the behaviour of the parameters is the same at all locations, only one optimisation strategy is needed.

While both the energy yield and the tidal stroke show a maximum, it was shown that this does not necessarily coincide with the lowest costs. The design needs to be able to maintain the tidal stroke throughout the lifetime, which should be given the most weight in the selection of a suitable design.

The pumpturbine number and diameter had the largest influence on the costs, stroke and energy yield. The analysis showed that it was the combination of the two, the flow area that is the most important. The range of values where the minimum tidal stroke is met, showed a large spread amongst the different basin types for n and D, but not for H_{rated}, $H_{start}$and H_{stop}.

For this reason a phased optimisation approach will be adapted. An initial guess is used for H_{rated}, $H_{start}$and H_{stop}based on the mean tidal range. With these values a optimal combination of n and D is found. The turbine number can only change with discrete values. It is assumed that the turbine diameter can only increase by 0.5 meters and the diameter has an upper limit. This limit is related to both the depth of the closure gap and technology. It is assumed that the pumpturbine diameter cannot exceed 12 meters. It is chosen not to combine the parameters n and D in one new parameter of flow area, but to treat them separately. This is because the diameter of the pumpturbine is of influence on the friction losses in the system. A large pumpturbine diameter has less friction losses, in the hydro energetic cost model (chapter 5) this is explicitly taken into account for the pumping energy. Taking this into account, the turbine diameter should be as large as possible. However, as the diameter and amount op pumpturbines can only change in discrete steps, the 'ideal flow area' can be more accurately be approximated by a larger number of small pumpturbines. By treating n and D as two parameters the model can find an optimal balance between these two principles.

Through a brute force grid search the best combination of n and D is found. This combination satisfies the minimum tidal stroke during the intended lifetime at maximum NPV. The net present value accounts for all cashflows during the lifetime: Investment costs, energy yield, pumping costs and maintenance and operational costs.

In the next phase the best value for $H_{rated}$is found, where the range of values is determined by the tidal range. This process is repeated for $H_{start}$and H_{stop}, where the range of values for $H_{stop}$is determined by H_{start}.

A certain minimum head difference is needed to set the turbine blades into motion to start generating energy. A larger turbine will require a larger starting head difference. The head difference is limited by the tidal range. The exact relationship between the minimum head difference and turbine size is not known. In the model for this reason the minimum $H_{start}$is set to 0.3 m. A minimum value for $H_{stop}$of 0.1 meters is set, for the same reason as that a minimum was set for H_{start}. The minimum is lower as it is estimated that the head difference can decrease once the blades are set into motion. Refinement of the relationship between the H_{start}, $H_{stop}$and the turbine diameter would be a good improvement on the model.

This process repeats itself iteratively replacing the initial guesses, until the found results do not change. Based on the behaviour of the parameters it is estimated that this optimisation sequence will most efficiently lead to the optimal design. The optimisation is performed for every climate scenario. This means that every design (location X designlifetime Y) alternative has four designs. At the start initial guesses are used for the parameters to be optimised, which will be replaced as the optimisation process progresses. First the optimal design for RCP26 is found, as this loop is finished, the found values are used as initial guesses in the loop for RCP45. The model thus gets smarter as the optimisation loop progresses, reducing time to convergence for the progressive climate scenarios. This is illustrated in figure 6.4.

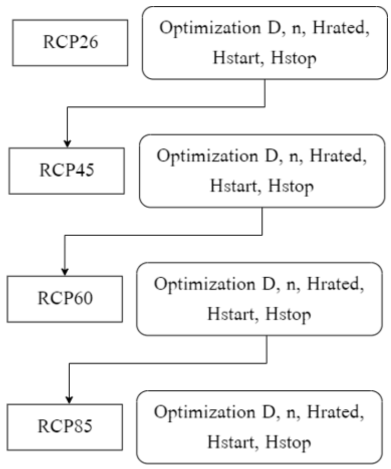


Figure 6.4: Optimisation approach climate scenarios

6.2.2. Model workflow

The workflow of the model is illustrated in figure 6.5.first iteration starts with a matrix of turbine diameters and numbers. The same model is run as in chapter 5, but now for each combination of turbine diameter and number. From the first iteration the best combination is chosen, meaning the combination with the best lifetime and the lowest costs. The best value for the diameter is the mean for a new array of diameters, with a smaller range than the previous loop, and the same is done for the number of turbines. The loop ends when the variation range is sufficiently small.

The next loop is initiated to calculate the best rated head for this combination of turbine diameter and number. Again the same model is run as in chapter 5 but now for multiple values of H_{rated}. The value of $H_{rated}$with the lowest cost is chosen, and the next loop is initiated to calculate $H_{start}$with the previously calculated values.

As the parameters for H_{rated}, $H_{start}$and H_{stop}were initially guessed, the process is repeated with the found values as starting values. The loop is exited once convergence has been reached, this means that the best values for n, D, H_{rated}, $H_{start}$and H_{stop}no longer change. When all parameters are known this results in the optimal design for that sea level rise scenario.

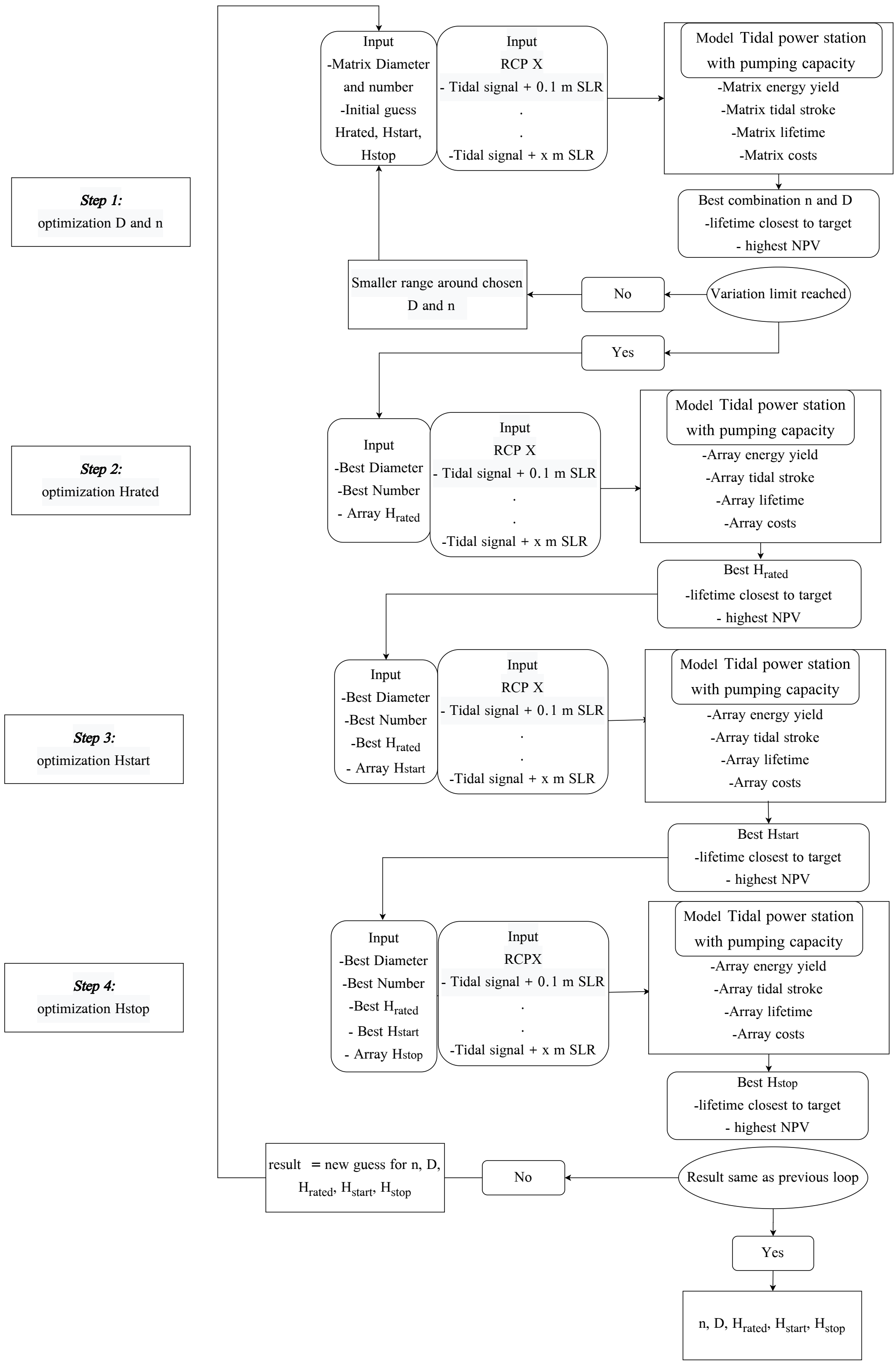


Figure 6.5: Workflow computer model optimisation

CHAPTER
SEVEN

RESULTS OPTIMAL DESIGNS

This chapter treats the results of the optimisation model.A total of 324 has been generated corresponding to the various locations, water level management strategies, climate scenarios and lifetimes. All results are included in appendix D. The results in this chapter will be discussed first by analysing one representative location: The Brouwersdam. This analysis focuses on the cashflows over the lifetime, how the design changes due to sea level rise and due to different design lifetimes. These observations hold for all locations. Next, the locations are compared and it is looked into what location specific factors influence the size, costs and profits of the power station with pumps. To conclude the chapter, the difference between the two water level management strategies is discussed.

7.1. Design Brouwersdam

The optimisation model yields a different optimal design depending on the amount of sea level rise that is to be expected during the lifetime of the structure. A minimum flow area is needed in order to facilitate the minimum tidal stroke at zero meters sea level rise. It was found that the pump capacity that corresponds to this basis flow area is effective in maintaining the tidal stroke up to around 0.5 meters sea level rise. Beyond this point additional pumping capacity is needed. Within 40 years, the expected global sea level rise for the four climate scenarios is relatively close together: 0.2 meters for RCP26 ranging to 0.3 m for RCP85. The optimal design does not vary for different climate scenarios. When looking at a design lifetime of a 100 years the scenarios show more differentiation. This is illustrated in figure 8.10. Since the sensitivity to sea level rise is one of the interesting aspects of the design, it is chosen to analyse the results based on the design lifetime 100 years. How the designs for design lifetime 100 years compare to the design lifetime 40 years and 100 years adaptive approaches will be discussed in more detail in subsection 7.1.3.

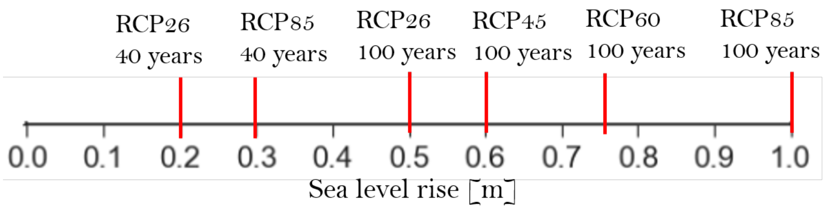


Figure 7.1: Link between Sea level rise and climate scenarios and lifetime

The analysis of the Brouwersdam results will be done based on the design lifetime 100 years. As the within the design lifetime of 40 years These are given in table 7.1. How the designs change because of the the design lifetime is discussed in subsection 7.1.3.

The flow area (A), design discharge (Q_d) and maximum pump capacity (P_t) directly follow from the found design parameters. Key results for each of the designs are also given in the table, the sea level rise at failure, the investment costs, the net present value and the return on investment. The return on investment (ROI) is defined as the portion of the investment that is returned over the lifetime. The definition is given in equation 7.1. It should be noted that the investment has a negative value.

Table 7.1: Optimal designs Brouwersdam

	SLR failure (m)	**n (-)**	**D (m)**	**Hrated (m)**	**Hstart (m)**	**Hstop (m)**	**Area (m2)**	**Pt (MW)**	**Investment (me)**	**NPV (mE)**	**ROI (-)**
Original design from previous studies: lifetime 60 years											
KNMI G	0.42	12	8	1.5	0.45	0.05	603	48	184	-183	0.01
optimised design by model: lifetime 100 years											
RCP26	0.5	47	4	1.7	0.4	0.1	590	51	181	-164	0.09
RCP45	0.6	63	3.5	1.7	0.4	0.1	606	52	186	-170	0.09
RCP60	0.75	14	8	1.7	0.4	0.1	703	61	215	-207	0.04
RCP85	1	14	8.5	1.7	0.3	0.1	794	69	242	-246	-0.02

$$ROI = \frac{NPV - C_i}{C_i} \tag{7.1}$$

With ROI the return on investment []
With NPV the net present value of the project in million euros
With C_i the investment costs in million euros

ROI larger than 1 means that the investment is earned back in full, and additional profits are made. A negative ROI smaller than 1 but larger than zero means only part of the investment is earned back, and a negative ROI means no money is earned back and the project has additional costs over the lifetime.

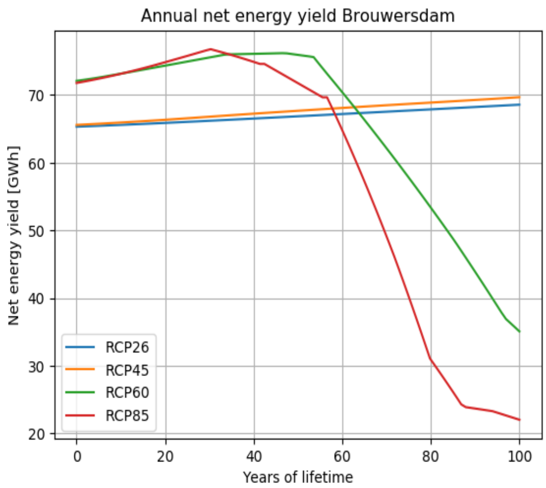
7.1.1. Comparison original design

The original design consist of 12 pumpturbines with 8 meters diameter and is able to maintain the tidal stroke up to 0.42 meters. The optimised design that most closely resembles this design is for RCP26. This design is able to maintain the tidal stroke up to 0.5 meters sea level rise. This improvement is achieved by the better tuning of both $H_{rated}$and H_{start}. The design has 47 pumpturbines of 4 meters diameter, resulting in a slightly smaller flow area. The pumping capacity is slightly higher, this is due to the larger design head difference. By opening the gates at an earlier moment (H_{start}smaller) the tidal stroke can be maintained for a longer period of time. The resulting design has similar investment costs but a larger return on the investment. The model thus successfully finds a more economically desirable configuration of the design parameters.

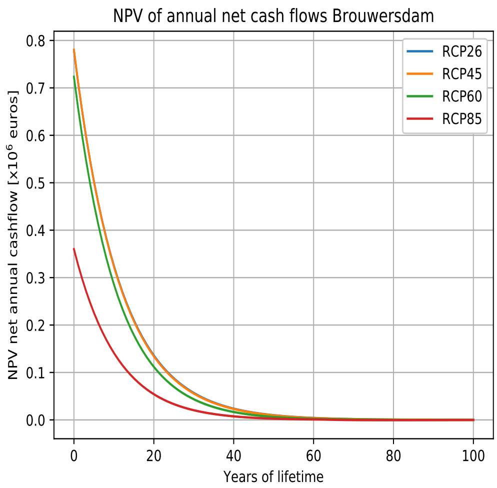
7.1.2. Effects sea level rise on design

It can be seen that the results of the model for RCP26 and RCP45 are very similar. This corresponds to 0.5-0.6 meters sea level rise (see figure 2.2). Beyond this amount of sea level rise the optimal design changes. More pumping capacity is needed in order to maintain the tidal stroke over the lifetime. Firstly it is observed that the flow area is increased as the climate scenario becomes more pessimistic, because a larger amount of sea level rise is expected. More pumping capacity is needed in order to maintain the minimum tidal stroke. An increase in flow area affects the costs of the structure in a few ways: A higher pumping capacity results in higher investment, maintenance and pumping energy costs. In contrast, the energy yield does increase as well. Because more pumps are required to maintain the tidal stroke in the more severe climate scenarios, also more pumping energy is used. Figure 7.2a shows the net energy yield (energy yield - pumping energy) over the lifetime of the four Brouwersdam designs for lifetime 100 years. It can be seen that the more severe climate scenarios have an higher initial net energy yield due to the higher number of pump turbines installed, but this drops as more pumping energy is used.

Due to the discount rate the maintenance and pumping costs during the first 50 years of the lifetime of the structure are governing for the net present value and design. Figure 7.2b shows the net present value of the annual cashflows over the lifetime of the four designs. The cashflows are made up from the net energy yield and the maintenance costs. The impact of higher maintenance costs on the cashflow can be seen when observing the difference between the net energy yield and net cash flows of RCP60 and RCP85. Both scenarios have a high initial net energy yield. However the initial net cashflow for RCP60 is about the same as for RCP26 and RCP45, which both have a lower net energy yield. The initial net cashflow for RCP85 is significantly lower than the other scenarios. These effects are attributed to the increasing maintenance costs as the size of the



(a)



(b)

Figure 7.2: Energy yield and cashflows over lifetime Brouwersdam

structure increases. It can be seen that after 50 years the cashflows for all designs converge, this is because of the used discount rate of 4.5%. One euro in 50 years is worth approximately 10 cents today. Cashflows in the far future thus have a smaller effect on the net present value than cash flows in the present. The negative effects on the NPV due to the increased pumping costs towards the end of the lifetime of the structures is therefore also limited.

7.1.3. Effect design lifetime on design

So far the results of the Brouwersdam have been discussed using the outcome of the optimised design with design lifetime 100 years. The difference between this design, and the designs for design lifetime 40 years and adaptive design 100 years will be discussed in this paragraph. Table 7.2 gives the design results for design lifetime 40 years and design lifetime 100 years adaptive.

Table 7.2: Designs Brouwersdam 40 years and 100 years adaptive

	SLR failure (m)	n (-)	D (m)	Hrated (m)	Hstart (m)	Hstop (m)	Area (m2)	Pt (MW)	Investment (m€)	NPV (m€)	ROI (-)
40 RCP26 - RCP85	0.5	47	4	1.7	0.4	0.1	590	51	181	-172	0.05
100 ad RCP26	0.6	40	4.5	1.7	0.5	0.1	636	56	195	-195	0.00
100 ad RCP45	0.6	40	4.5	1.7	0.5	0.1	636	56	195	-213	-0.09
100 ad RCP60	0.75	25	8	1.2	0.3	0.1	1256	66	360	-391	-0.09
100 ad RCP85	1	42	6.5	1.2	0.3	0.1	1393	73	400	-445	-0.11

The resulting optimal designs for 40 years and RCP26 100 years are the same. The amount of sea level rise expected over the lifetime is determining in the design, not the lifetime itself. The resulting net present values of the structure however are different for the different lifetimes, as the station is operational for a longer time when the design lifetime is 100 years instead of 40. But as becomes clear from figure 7.2b, the first 40 - 60 years of the structure is when the largest return on the investment is achieved. The resulting net present values for a structure operated for 40 years and 100 years are therefore rather close together. (NPV -172 and -164 million euros respectively)

The adaptive design has a design lifetime of 100 years, but should be able to function without pumps for the first 15 years, this way, the machinery can be installed at a later instance depending on how fast sea level rises. The results show that the adaptive designs have a larger flow area. The designs for RCP26 and RCP45 are the same, after which the flow area increases. The net present value of the is more negative and the return on investment becomes smaller. This is to be expected as during the first 15 years no energy is generated so no income is generated, furthermore the higher investment costs equal higher maintenance costs. When after this period of 15 years all the empty culverts are filled with pump/turbines, more pumps would be installed than are necessary during the lifetime of a 100 years. It could be chosen to leave some of the culverts empty. An assessment should be made whether the extra investment in empty culverts is justified in order to have an adaptive design. This design as is, does not represent an optimal design and is not taken into account in further steps of the analysis.

7.1.4. Conclusions Brouwersdam

The optimisation model has shown that an improvement can be made upon the original Brouwersdam design. The optimisation model found a different optimal design from the original. The new design has a slightly smaller flow area and different values for the design head and starting head. The tidal stroke can be maintained up to 0.5 meters sea level rise and with a 9% less negative net present value.

From the analysis of the design results for the Brouwersdam a few important conclusions can be made that apply to all locations. The ratio between the energy yield and maintenance costs are determining for the return on investment. As a more severe climate scenario requires a larger structure and more maintenance costs, this leads to a reduction in the return on investment. The increase in maintenance costs is dominant over the increase in energy yield. This highlights the importance of the ratio between investment costs and energy yield, which can differ per location. This will be further discussed in section 7.2.

Furthermore it was found that the cashflows during the first years that the station is operational have the largest influence on the net present value of the structure. As towards the end of the lifetime the sea level has risen and more pumping energy is used, the cashflows reduces or can become negative. However, the effect on the net present value is very small. This also means that when the same design is used for 40 years or a 100 years, the difference in net present value will be relatively small. It was found that the adaptive design was not efficient, its net present value is significantly more negative.

These conclusions also hold for the results of the other locations. The return on investment, change in flow area due to sea level rise and change in area due to design lifetime are governed by the same principles at these locations. These are not discussed in further detail. All results can be found in appendix D.

7.2. Comparison locations

In the previous section the design results of the optimisation model for the Brouwersdam were analysed. The baseline design is taken as the economic strategy, RCP26, designlifetime 100 years. The found effects of sea level rise on the results that hold for the Brouwersdam also hold for the other designs. However, there are differences observed between the results in the different locations, which will be discussed in this section. To this end location specific factors of influence will be analysed, after which the size of the power station and the costs and profits are discussed.

7.2.1. Location specific factors

While there are some general trends that can be observed in the results of the design model, the different locations all have different results. This can be attributed to location specific factors that influence the costs and profits at a location. The most important location specific factors that were taken into account in the model are the mean tidal range, the relative daily inequality, the regional sea level rise index and the construction cost index at the locations. The influence of these parameters will be elaborated on and are used in the comparison between locations.

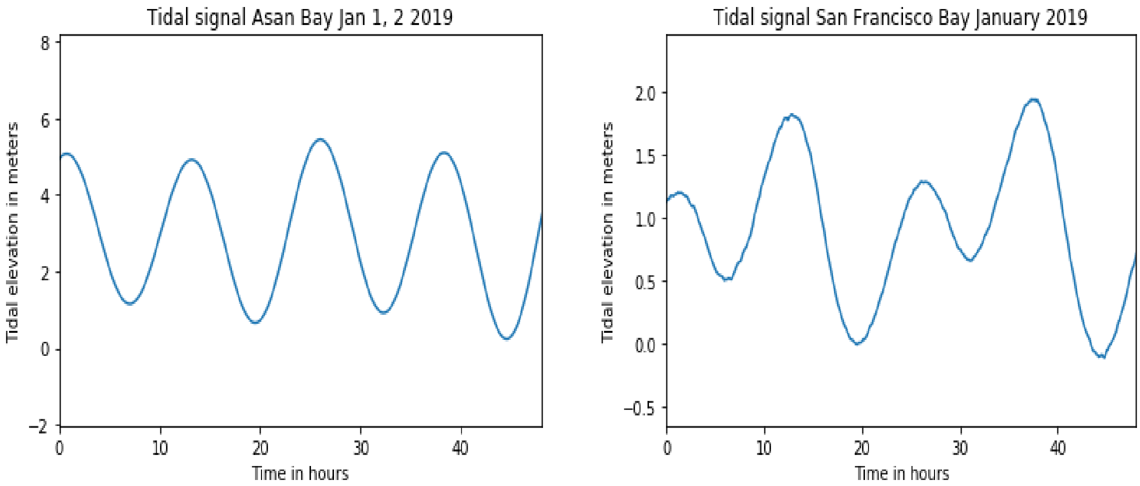
The mean tidal range is a measure of the tidal amplitude, in general a large tidal amplitude means high energy generation. An increase in the head difference causes a quadratic increase in the energy yield. A location with a large tidal range will thus have a larger energy yield for a station of comparable size with a smaller tidal range.

The relative daily inequality relates the difference in tidal amplitude between two subsequent tidal cycles. All locations have a semi diurnal tide, meaning there are two high tides and two low tides every day, when these are not equal in amplitude, there is a daily inequality. The relative daily inequality is calculated as follows:

$$DI_r = \frac{\sum_1^{365} A_{\mathrm{min,1,T}} / A_{\mathrm{min,2,T}}}{T} \tag{7.2}$$

With DI_r the relative daily inequality [-]
With $A_{\mathrm{max,1,T}}$ the first low tide on day T
With $A_{\mathrm{max,2,T}}$ the second low tide on day T
With T the day in a year

A relative daily inequality close to 1 means a very uniform tide, with no daily inequality. Figures 7.3a and 7.3b illustrate a signal with a relatively small daily inequality, and a tidal signal with a relatively large daily inequality.



(a) Relative daily inequality 1.003

(b) Relative daily inequality 1.434

Figure 7.3: Daily inequality of tidal signal

A large daily inequality has an impact on the energy balance at a location. The design discharge, starting head difference and stopping head difference found by the model are a compromise between the two amplitudes. During the low amplitude tidal cycle pumping is applied already earlier in the lifetime of the structure, to maintain the tidal stroke. A large daily inequality will thus cause less energy to be generated and calls for more pumping capacity.

The regional sea level rise index relates how the sea level is expected to rise in relation to the projected global sea level rise. This was discussed in section 2.3. In general, a location where more sea level rise occurs will need more pumps and is likely to be more expensive in comparison to locations where less sea level rise is expected. The costs of other coastal defence measures however will also be dependent on the amount of sea level rise. In chapter 8 these effects will be quantified and it is shown how this affects the economic feasibility of the dam with tidal power station including pumping capacity.

The construction cost index was discussed in section 5.2.5. A country with high construction costs leads to a high initial investment and will have a less positive energy yield to maintenance cost ratio. The return on investment is then lower.

Table 7.3 gives an overview of these factors for each of the locations.

7.2.2. Size power station

The different locations vary in tidal range, basin size and desired minimum tidal stroke. This leads to distinct designs for each locations. In this section the size of the power station at the locations is discussed, as this is a direct measure of the investment costs that can be expected. The size of a powerstation is largely dependent on the number and diameter of the pump turbines. The flow area is a combination of these two parameters, the expression is given in equation 7.3.

Table 7.3: Location specific factors used in model

Location	MTR (m)	Relative daily inequality (-)	Regional SLR index (-)	Construction cost index (-)
Asan Bay	6	1.003	0.9	1
Brouwersdam	2.3	1.059	1	1
Dee estuary	6	1.004	0.75	1.14
Dharamtar Creek	3.3	1.223	0.9	0.36
Haeju Bay	5	1.057	0.8	1
Mokpo	3	1.321	0.9	1
San Francisco	1.7	1.434	0.9	1.6
Xinghua Bay	4	1.057	1	0.43

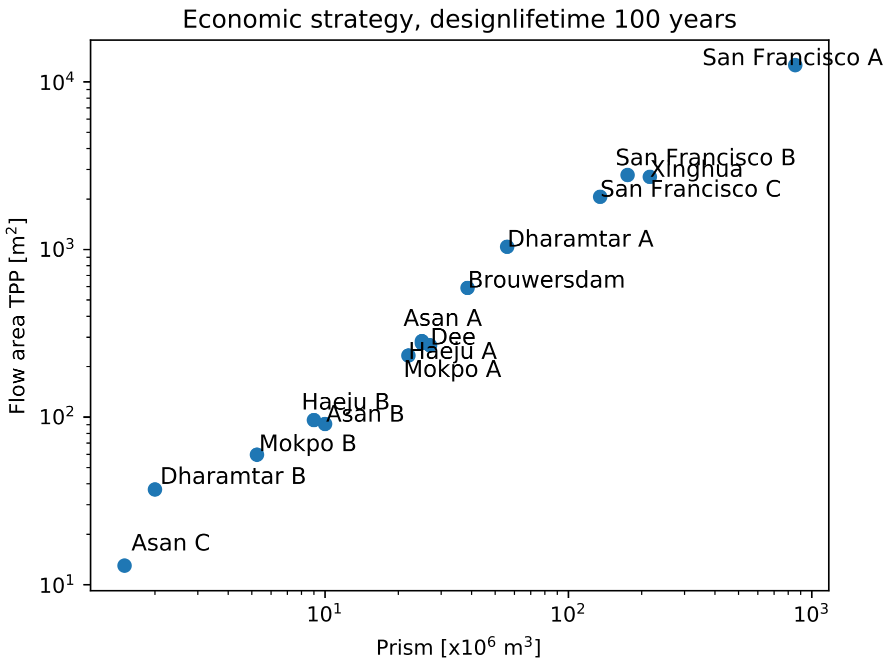


Figure 7.4: Prism versus flow area

$$A_{\text{flow}} = \frac{\pi}{4}\pi n D^2 \tag{7.3}$$

The flow area needed is in large part dependent on the volume of water that needs to move through the power station per tidal cycle. This is the tidal prism, the definition is given in equation 7.4.

$$P = SR_l \tag{7.4}$$

With P the prism in m^3
With S the basin area in m^2
With R_l the minimum tidal stroke at the lake in m

Figure 7.4 shows the prism versus the flow area of the designs. Because of the large different in scale of the different stations it is plotted on a double logarithmic scale.

It can be seen that there is a large difference in scale, the station at San Francisco A is many times larger than the other stations. An almost linear trend is observed for the increase of prism versus the increase of flow area. However as can be seen for example when looking at Asan A and Mokpo A, the same prism will not lead to the exact same flow area for two different locations. The main reason for this is the amount of pumping

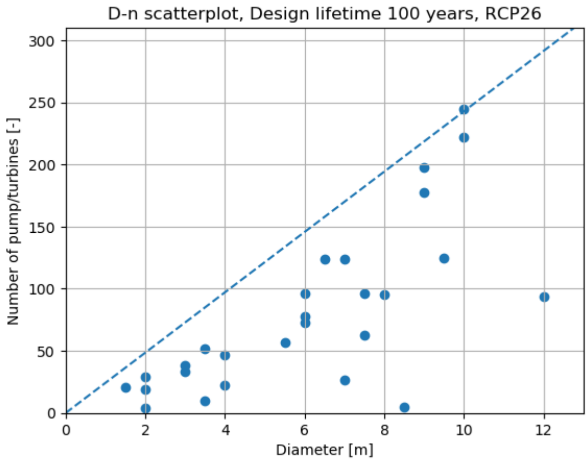


Figure 7.5: Optimised design D-n, designs below diagonal

energy that is required at a location. If more pumping capacity is needed to maintain the tidal stroke, the flow area increases as more pumps are installed.

There are a few reasons why pumping energy at a certain location can be more or less than at another location. If the tidal signal is has a large daily inequality, more pumping is needed to maintain the regular tidal stroke at the lake. This explains the difference in flow area between for example Asan C (small daily inequality) and Dharamtar B (large daily inequality) in figure 7.4. Furthermore, locations with relatively more regional sea level rise will need more pumping energy. For example at Dee Estuary the regional sea level rise is only 75 % of the global level, this location requires little pumping energy. Finally the ratio between the minimum tidal stroke and the mean tidal range can also be an indicator of pump energy required. If the minimum tidal stroke is relatively close to the original tidal stroke, pumps will need to be implemented already earlier in the lifetime to maintain the tidal stroke. This is the case for San Francisco A, which also has a large daily inequality. This location thus requires a lot of pumps.

Figure 7.5 shows how the flow area is made up out of different combinations of D and n. In the figure it is observed that the model chooses designs below a certain diagonal. There are no cases where the model finds an optimal design being a very large number of small pumps. This can be explained by the trade-off between less friction losses for a larger pump turbine diameter and a more accurate approximation of the ideal flow area if a large number of small pump turbines are used. This effect was previously predicted in section 6.2.1.

7.2.3. Return on Investment

As the solution includes a tidal power station, it has the potential to 'pay for itself'. In this case the return on investment over the lifetime would be at least 1. In the next chapter the business case of a dam with tidal power station including pumping capacity will be compared to two other alternatives, being a dam with culvert and updating coastal defences. The return on investment is a useful metric when comparing the dam with culvert to the dam with TPP and pumping capacity. As the culvert alternative has a zero return on investment but in most cases has a lower investment costs, the ROI of the TPP with pumping capacity will be key in determining which solution will be favourable. This will be discussed in more detail in chapter 8. Figure 7.6 shows the plot of the flow area versus the net present value of the designs. Because of the large difference in scale in both A and NPV, the figure has been plotted on double logarithmic axes. The net present value is negative for all locations.

From the figure it becomes clear that there are a few locations where part of the investment is earned back (green dots) and other locations where an additional loss over the lifetime occurs (red dots). The locations with the highest profits are Asan Bay, Dee Estuary and Xinghua Bay. It can be seen that for these three locations 60% - 50% of the investment costs are earned back. Both Asan Bay and Dee estuary have a large mean tidal range and a relatively small daily inequality. Asan Bay has a slightly higher return on investment than Dee Estuary. The construction labour costs are higher at Dee estuary, as can be seen in table 7.3, this explains the lower return on investment. The tidal range at Xinghua Bay is smaller, and the daily inequality is also slightly larger. However at this location the construction costs are relatively low, this also leads to a favourable ratio between energy yield and maintenance costs. Both Haeju Bay and the

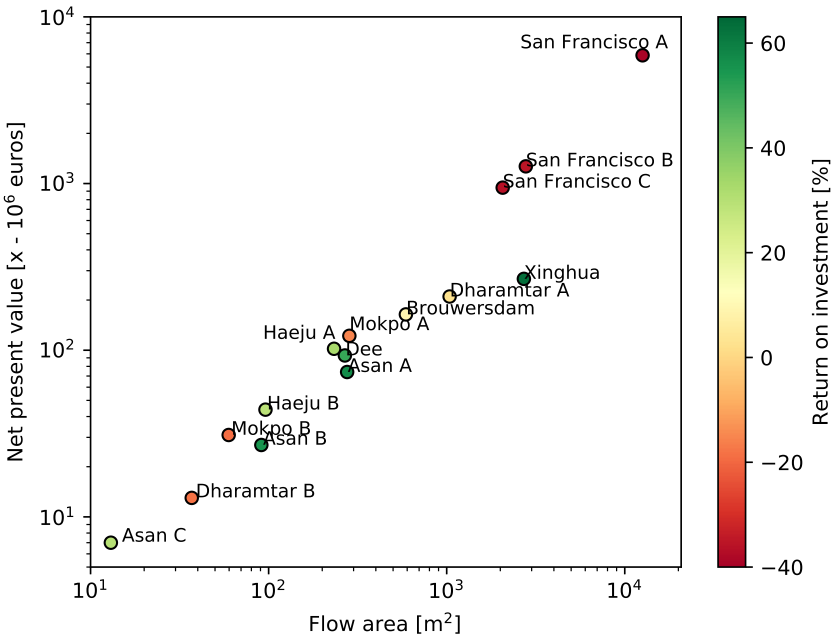


Figure 7.6: Flow area versus Net Present Value

Brouwersdam have about the same daily inequality and the same construction costs index. As the tidal range at Haeju Bay is larger than at Brouwersdam, here more energy is generated and the location has a larger return on investment.

Locations where a net loss over the lifetime is experienced are Dharamtar Creek, Mokpo and San Francisco. At these locations the structure costs more money during the lifetime than it earns. These locations have in common that they do not have a homogeneous tidal signal, there is a large daily inequality in tidal amplitudes. San Francisco has an especially negative return on investment, this is because of the high construction costs at the location. The ratio between energy yield and maintenance costs becomes less favourable. The net present value of the cash flows in San Francisco A design lifetime 100 years is shown in figure 7.7 to illustrate this. It can be seen that for all designs the cashflows are negative from the start of the lifetime, meaning an annual loss.

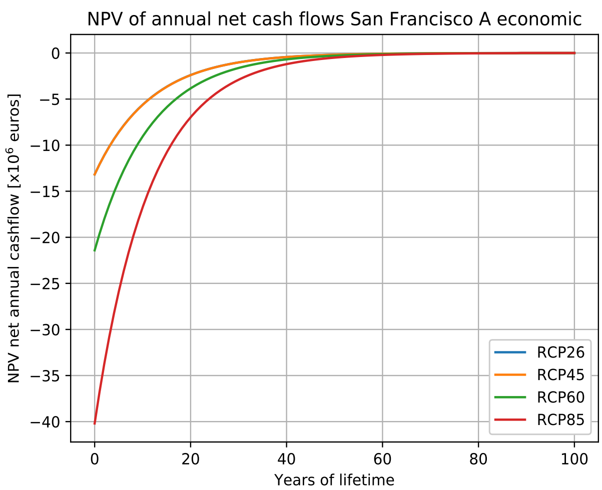


Figure 7.7: Net present value of annual cashflows at San Francisco variant A

From figure 7.6 it can be seen that all net present values are negative. The least negative are locations with a small power station and a relatively high return on investment. This combination implies a minimum investment and a maximum return. As was discussed in section 7.2.2 the main factor of influence on the size is the prism. Factors that influence the return on investment were discussed in the previous section: homogenuity of the tidal signal, the tidal range and the location specific construction costs. A location with a small prism, large tidal range and a regular signal will thus have the least negative business case. For the analysed locations this is Asan Bay C. In the next chapter the business case of the tidal power station with pumping capacity is compared to two alternatives: updating existing coastal defences, and a dam with culvert. The net present value of these alternatives will also be negative, as the cashflows consists only of the investment and maintenance costs. In each situation the least negative business case out of the three alternatives will be the best solution.

7.3. Comparison strategies: nature vs economic

Two water level management strategies were adapted to determine the minimum tidal stroke: the nature strategy and the economic strategy (section 2.2). The baseline solution that has been discussed so far was the economic strategy for all locations, where a certain standard of water quality is ensured. The nature strategy is characterized by a minimum tidal stroke close to the original tidal stroke at sea. The prism thus increases, as do the pumping needs. The size of the station and the corresponding construction costs increase accordingly.

This leads to a less favourable ratio of energy yield versus maintenance costs, the return on investment is thus lower for the nature strategy than the economic strategy. An example of this is given for one of the locations: Dee estuary, Table 7.4 gives the designs for both strategies for design lifetime 100 years RCP26. Both the flow area and investment costs increase with roughly a factor 22. The net present value however decreases with approximately a factor 39, this is because the nature strategy has a much lower return on investment. This effect is observed at all locations where the two strategies are applied. It should be noted that both the economic and nature strategies are extreme ends of the spectrum, and the adapted water level management strategy will likely be an intermediate value based on political choices. Furthermore while the NPV of the nature value is much more negative, the size of the station is also a lot bigger. This means that it takes up a larger part of the dam, meaning that the residual costs to construct the dam will be lower in comparison to the economic strategy. In this chapter looks at the isolated tidal power station with pumping capacity, in the next chapter the integral solution with dam is assessed. The differences in costs between the economic strategy and the nature strategy will then be more nuanced.

Table 7.4: Dee Estuary design lifetime 100 years RCP26

	Prism ($x10^6$ m^3)	**n (-)**	**D (m)**	**A (m^2)**	**NPV (m€)**	**Investment (m€)**	**ROI (%)**
Economic	27	38	3	268	-86	176	0.51
Nature	450	67	10.5	5799	-3375	3907	0.14

While the economic strategy has lower investment costs and a larger return on the investment, the effects on ecology can be substantial. A standard of Water quality is guaranteed over the lifetime but the intertidal character of the basin will change significantly when the tidal stroke in a a basin is strongly reduced. This can be an incentive to choose the more expensive alternative of the nature strategy. The nature strategy has less of the ecological drawbacks as the tidal stroke is only mildly reduced. Compared to conventional flood protection measures such as dikes and floodwalls the effects on ecology are assumed to be similar. It will be dependent on the valuation of the nature in the area and the local policy what strategy is best to be adapted.

CHAPTER
EIGHT

BUSINESS CASES

In this chapter the feasibility of the tidal power station with pumping capacity is assessed, based on a series of analyses and business cases. The solution of the dam integrated with a tidal power station is compared to two alternatives. A dam with simple culvert and a traditional alternative: updating the coastal defence structures already in place. All three alternatives serve the same purpose: increasing flood safety at a location with a minimal ecological impact.

From the previous chapter it became clear that the tidal power station with pumps in itself does not lead to a positive business case. Even though the station does not earn back its investment, it could still be the most favorable alternative to protect an estuary. This can be because of the integrated cost of the dam and tidal pumping station are lower than that of adapting the coastal defense structures. Where traditional dams are often not considered due to disadvantageous effects on ecology, this integrated dam solution does not have this limitation. First the main features of the three alternatives are discussed and the associated costs are discussed. Next, the business cases of the alternatives at each location are compared.

8.1. The three alternatives

The three considered alternatives are: the dam with tidal power station and pumping capacity, the dam with culvert, and updating coastal defences. A solution fulfills the basic requirements, when it protects against flooding for the duration of the lifetime, with a minimal ecological impact by maintaining a certain tidal stroke in the basin. If for a certain situation one of the alternatives does not fulfill these requirements, it is left out of the consideration. This section discusses the costs (and profits) of each of these solutions, which is input to the comparison of the business cases in next section.

8.1.1. Dam with tidal power station with pumping capacity

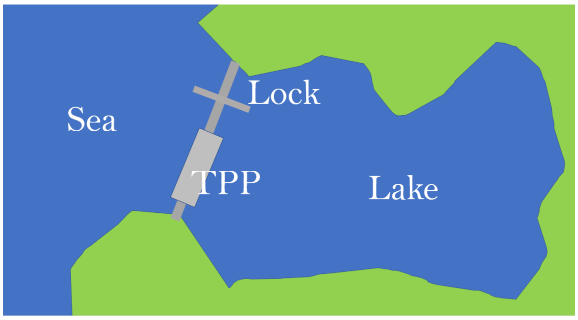
The solution of the tidal power station with pumping capacity was discussed in chapter 4. The costs corresponding to the power station with pumping capacity were discussed in section 5.2.5. In this section a brief summary is given. Figure 8.1a shows the concept of the dam with tidal power station with pumping capacity. A dam is used to close of a tidal estuary creating an artificial lake. The tidal power station with pumps is where the water can move through the barrier. A lock is included to allow ships to pass. Figure 8.1b shows a side view of a powerhouse in the power station with pumping capacity. Important elements are the bed protection, the gates and the pump/turbine.

Costs power station

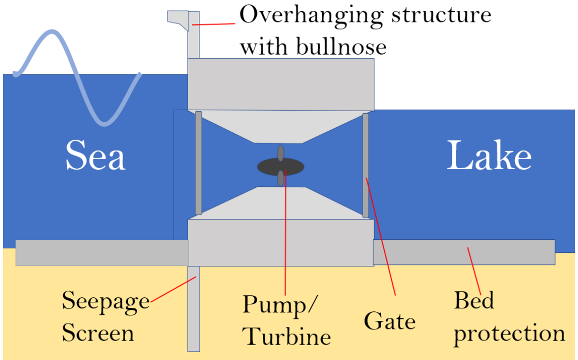
The costs of the power station with pumping capacity were calculated with the hydro-energetic cost model. The costs for the designs were discussed in chapter 7. A complete overview of all designs and costs can be found in appendix D.

Costs dam

The most recent large scale dam built is the 33 km long Saemangeum seawall in South Korea with a total cost of 3.6 billion US dollars in 2020 or 2.9 billion euros. This cost will be used to derive an expression for the cost



(a) Overview dam tidal power station with pumping capacity



(b) Side view powerhouse tidal power station

Figure 8.1: Dam with tidal powerstation with pumping capacity

of a dam per km and per meter depth. The average depth along the Saemangeum seawall was 6 meters. This leads to a cost per m/km of 14.5 million euros. The location indices that were used to scale construction costs in chapter 5 are also applicable in this context. Equation 8.1 gives the expression for the costs of the dam.

$$C_{\text{Dam}} = 14.5hLI \tag{8.1}$$

with C_{Dam} the cost of the dam in million euros
with L the length of the dam in km
With h the average water depth in meters
With I the location index [-]

In the Saemangeum seawall two navigational locks are included. Their respective dimensions are 62 m x 16 m x 12.5 m and 30 m x 16 m x 12.5 m. The costs of these navigational locks is included in the total project costs of 2.9 billion euros. These are relatively small locks compared to the largest lock in the world at Ijmuiden. The lock is 500 meters long, has a width of 70 meters and a depth of 18 meters. The total project costs of this lock are approximately 1 billion euros.

The price estimation of the dam thus already includes the costs for small scale locks. For locations where an important navigational route is behind the barrier, additional costs will be taken into account as it is expected that the lock will need more capacity. A unit cost of 50% of the sealock of Ijmuiden, scaled to location by the location index, will in that case be added to the total project costs. This is the case for all variants in San Francisco, Dharamtar A and Mokpo A.

The estimation of the lock costs is quite crude: it is now linked to the length and depth of the closure gap for all locations, and additional costs have been taken into account for a few locations. It is assumed that for the purposes of the comparison of the business case the costs estimates are accurate enough, as the costs of the locks will be relatively small compared to the total costs of the solution. In further studies the costs of the locks should be considered in more detail.

Table 8.1 gives the costs of the dam per location for the base scenario of zero meters sea level rise and without any power station or culvert. The costs change as it is integrated with either a power station or culverts. Furthermore the water depth increases as sea level rises, this will impact the costs of the dam. Both factors are taken into account in the business cases. The span of the tidal power station is subtracted from the dam length, and the dam height adapts to sea level rise by taking it into account in the water depth. When assuming that wave breaking is depth limited the height of the dam will need to increase more than the absolute amount of sea level rise. This relationship is discussed in more detail in subsection 8.1.3.

8.1.2. Dam with culvert

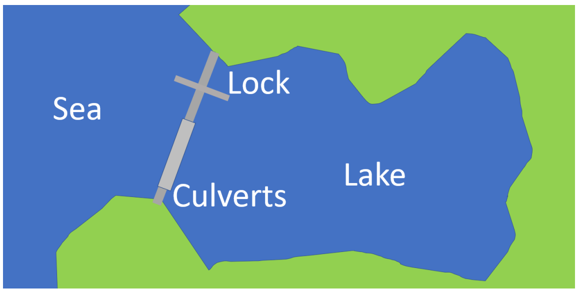
This solution shows a lot of similarities with the previous solution, the only difference being the power station is replaced by a series of culverts. This is shown in figure 8.2a. Figure 8.2b shows the side view of the culverts. Only one gate is needed to regulate the water, as no maintenance is needed within the culvert.

The costs of the culvert are calculated using the same relations as the tidal power station with pumping capacity. Different from the station, the amount of gates is halved. Furthermore a 5% reduction on the total

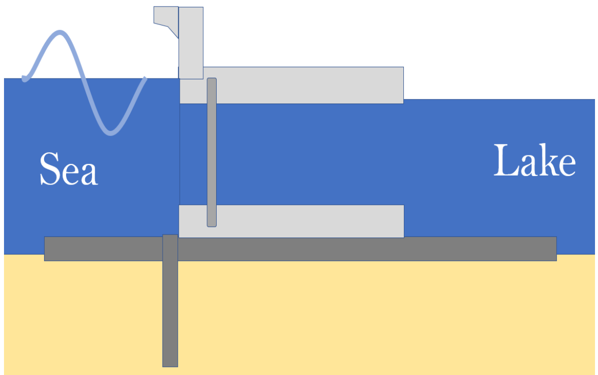
Table 8.1: Costs dam without power station and zero meters sea level rise

	Width gap (km)	**Depth gap (m)**	**Index (-)**	**Cost dam (me)**
Asan A	17	4	1	986
Asan B	5.5	4	1	319
Asan C*	n.a	n.a	n.a	n.a.
Brouwersdam*	n.a	n.a	n.a	n.a.
Dee Estuary	8	5	1.14	661
Dharamtar Creek A	11	10	0.36	574
Dharamtar Creek B	3	6	0.36	94
Haeju Bay A	5	6	1	435
Haeju Bay B	1	6	1	87
Mokpo A	5	15	1	1088
Mokpo B*	n.a.	n.a.	n.a.	n.a.
San Francisco A	7	40	1.6	6496
San Francisco B	7	15	1.6	2436
San Francisco C	7	10	1.6	1624
Xinghua Bay	14	22	0.43	1920

* Asan C, Brouwersdam and Mokpo B are three locations where a dam is already present, the cost calculation of the dam is therefore not applicable at these locations



(a) Front view culvert



(b) Side view culvert

Figure 8.2: Conceptual design culverts

material costs of the structure are applied to account for the fact a powerhouse caisson requires a more complex geometry than a culvert caisson. The cost of the pump turbines and their maintenance is also excluded from the relationship. The adapted expressions are given in equations 8.2 and 8.3, respectively.

$$C_{\text{mat}} = 0.95(C_p + C_r + \frac{1}{2}C_g + C_b) \tag{8.2}$$

With C_{mat} the total material cost in euros
With C_p the cost of the concrete of the powerhouse in euros
With C_r the cost of the concrete reinforcement in euros
With C_g the cost of the gates in euros
With C_b the the cost of the bed protection in euros

$$C_{\text{total}} = (2C_{\text{mat}} + C_{\text{mo, c}} + C_{\text{infra}} + C_{\text{nature}})I \tag{8.3}$$

With C_{mat} the total material cost in euros
With $C_{mo,c}$ the maintenance costs of the civil structure in million euros
With C_{infra} the compensation costs paid for infrastructure in million euros

WithC_{nature} the compensation costs paid for nature in million euros
with I the location index [-]

The unit costs of the dam and lock are the same as for the dam with tidal power station and pumping capacity. Since for the culverts in most cases a larger flow area is required than when pumps are used, the caissons will cover a larger part of the dam. This can cause the cost of the dam to differ between the two variants.A complete table with all results is included in appendix E.

It should be noted that for some locations it was not possible to find a flow area for the culvert that would meet all the requirements. While the minimum tidal stroke initially could be achieved, it could not be maintained for the entire lifetime, as the mean water level within the lake would have to rise. This was mostly the case for locations with a large daily inequality. For these locations the alternative with a culvert is thus not feasible.

In other cases the monthly variation in tidal amplitude caused a complication. While during most tidal cycles the minimum tidal stroke was met, during neap tide at sea there is no stroke at the lake which can persist for several days. When this is a monthly occurrence, this will have a disruptive effect on the inter tidal areas. This happens at locations where there is a pronounced difference in the amplitude during spring and neap tide. In this case the requirements of the minimum tidal stroke are also not met and thus the alternative with pumps automatically becomes the best alternative in these cases as well.

Table 8.2 shows the results for the total flow area for each location and strategy for RCP85, design lifetime 40 years, The results for all climate scenarios and lifetimes are included in appendix E.

Table 8.2: Results culverts, design lifetime 40 years RCP85

	Strategy			
	Economic		**Nature**	
Location	**Area (m2)**	**NPV (me)**	**Area (m2)**	**NPV (me)**
Asan A	292	-73	9643	-2398
Asan B	119	-30	3915	-974
Asan C	17	-4	n.a. *	
Brouwersdam	869	-124	n.a. *	
Dee Estuary	251	-106	5628	-2364
Dharamtar A	587	-49	n.a. **	
Dharamtar B	21	-2	n.a. **	
Haeju A	289	-104	n.a. **	
Haeju B	123	-44	n.a. **	
Mokpo A	751	-140	n.a. **	
Mokpo B	123	-23	n.a.*	
San Francisco A	27415	-5064	n.a.**	
San Francisco B	5820	-3546	n.a.**	
San Francisco C	3278	-982	n.a.**	
Xinghua	2501	-318	21538	-2737

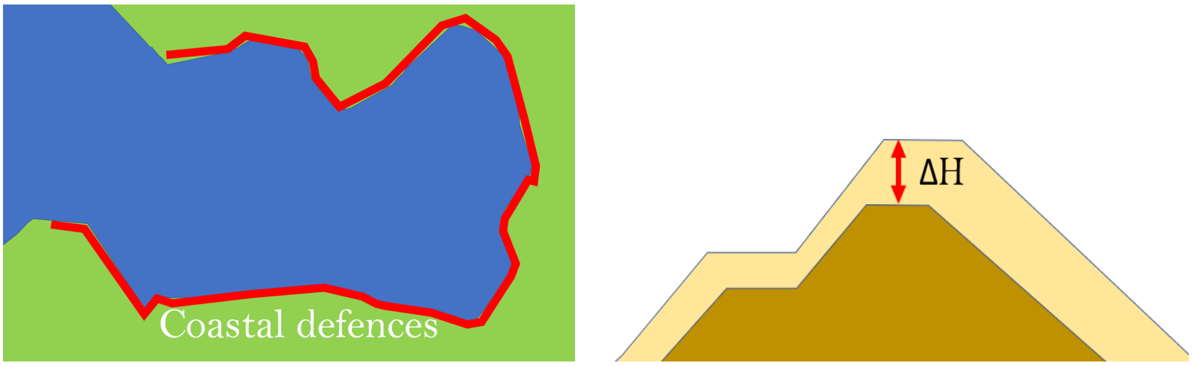
* Dam already present, only economic strategy assessed
** Tidal stroke not sufficient over lifetime

8.1.3. Updating coastal defences

An alternative measure to protect a basin of increased flooding due to sea level rise is strengthening and heightening of coastal defences around the perimeter of the basin. This is illustrated in figure 8.3a. Figure 8.3b illustrates the heightening of a coastal dike, where it can be seen that to heighten a dike and maintain the same slope, the dike also increases in dimensions in lateral direction.

The costs are dependent on many factors, such as whether there are already defences present and the construction costs that are applicable in the region. The costs of raising dikes is dependent on several factors, such as degree of urbanisation and the needed height and dike slope. Since it is beyond the scope of the study to inventory all present and required coastal defences for every location some assumptions need to be made.

Firstly, because all the selected locations are in a densely populated area with high flood risk it is assumed that there are already coastal defences in place. To withstand sea level rise the defences will thus only have to



(a) Updating coastal defences around perimeter

(b) Heightening dike

Figure 8.3: Conceptual design culverts

be adapted, not newly constructed. Two types of coastal defences will be considered: floodwalls and dikes. The assumption is thus that these are the most prominent coastal defences for all locations.

Secondly, the type of coastal defences have to be linked to the perimeter of the basin. It is not likely that the entire perimeter of the basin is equally well protected against flooding. The assumption is made that a certain portion of the perimeter is protected by dikes, a certain portion is protected by floodwalls while a third portion is either not protected or protected by measures not considered here such as berms. As a reference case the San Francisco South Bay is used, which is treated in the case study in chapter 9. San Francisco Estuary Institute-Aquatic Science Centre (2016) mapped all coastal defense structures and shoreline along the San Francisco Bay, this is illustrated in figure 8.4. It can be seen that the defense structures extend beyond strictly the perimeter of the bay, and are also around rivers and waterways that flow to the Bay. The smoothed perimeter of the Bay was estimated at 400 kilometers, the total distance mapped by the SFEI is 4820 kilometers, of which 272 kilometers consists of dikes and 280 kilometers of floodwalls. If the two considered flood defenses are scaled to the smoothed perimeter, this is approximately 0.7 km of floodwall and 0.7 km of dike per smoothed kilometer. The assumption is made that this is a reasonable estimation for all locations. Because all the locations are highly urbanised this large portion of flood wall is likely, since it is space efficient. Evidently these ratios will differ per location, but suffice to make an initial estimation of the costs of updating coastal defences around the perimeter of the basin.

For the unit cost of updating the coastal structure, the relationships as described by Jonkman et al. (2013) are used. The study provides unit cost of coastal defence measures for 3 cases: The Netherlands, The United States (New Orleans) and Vietnam (Hai Phong). The relevant data for the defence measures considered are given in table 8.3. As can be seen the range in cost estimation is still quite considerable.

It is chosen to use the dike heightening prices from Kok et al. (2008) (21.5 - 25.6 million euros/km/m) and to raise flood walls the range from of Engineers (2009) are chosen (5.6 - 13.6 million euros /km/m). Both will be scaled to location with the location index. (The prices for the floodwalls are for location New Orleans, which has location factor 0.9 (Arcadis, 2019) (nearest city Dallas). For the analysis in this chapter the mean values of the unit prices are used.

Table 8.3: Unit costs of coastal defence measures, converted to 2020 price levels, and sources. Jonkman et al. (2013)

	Cost intervention (M€/km/m)
Netherlands	Dike raising urban
	21.5 - 25.6 Kok et al. (2008)
	17.8 ARCADIS (2006)
United States	T floodwall raising
	5.6 - 13.6 of Engineers (2009)
	4.3 - 5.2 Bos (2008)
	Floodwall/dike raising and strengthening
	5.1 - 10.5 of Engineers (2009)
Maintenance flood defences	0.1 (Me/km/y) Vellinga et al. (2006)

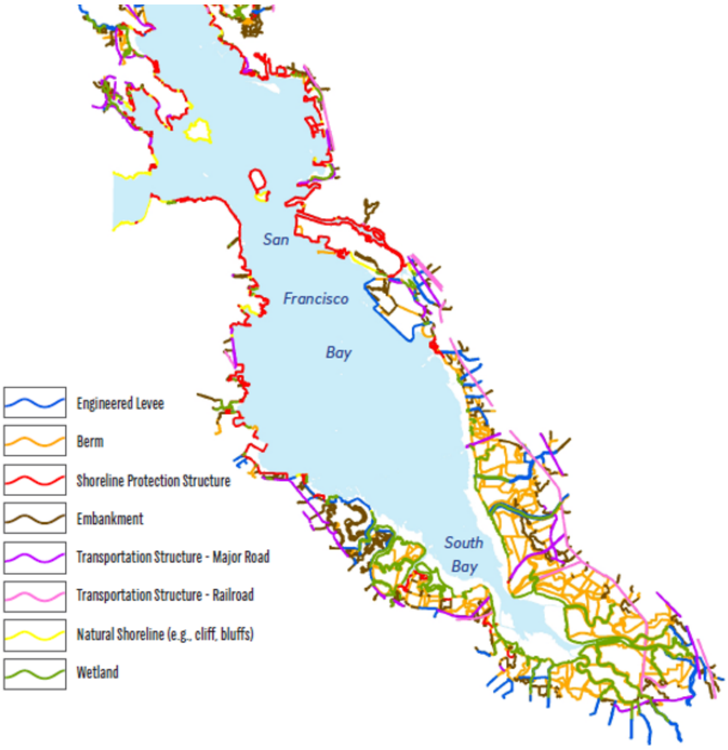


Figure 8.4: Mapped coastline San Francisco Bay area. From San Francisco Estuary Institute-Aquatic Science Centre (2016)

Lastly the amount of heightening/enforcing that a coastal defense needs to undergo has to be linked to sea level rise. This will vary per location and is dependent on the local safety standards and whether wave breaking is depth limited or not. In case wave breaking is depth limited, wave heights will grow as sea level rises. This causes the defence structures to increase more than the increase in sea level rise. Jonkman et al. (2013) provides a relationship for the relative height increase needed for a typical Dutch sea dike, for multiple levels of sea level rise. This is given in table 8.4. Safety standards in the Netherlands are relatively high, and an average Dutch dike is about 12 meters high. Since there is no data on the height of the coastal defences in the other locations, a standard value of 10 meters will be chosen. Furthermore the same relationship for sea level rise and increase in dike height, will be used for the flood walls.

Table 8.4: Relative height increase for typical Dutch sea dike for sea level rise Jonkman et al. (2013)

Sea level rise [m]	**Required dike heightening Δ H [m] (%)**
0	basis = 10
0.5	1 (10)
1	2 (20)
2	4 (40)
5	10 (100)

This leads to the following cost relationship:

$$C_{\text{cd,t}} = P\Delta H(0.7C_d + 0.7C_w) \tag{8.4}$$

With $C_{\text{cd,t}}$ the total costs of the coastal defences in million euros
With P the perimeter in of the basin in meters
With Δ H the required height increase in meters
With C_d the cost of heightening the dikes per kilometer per meter in million euros
With C_w the cost of heightening the floodwalls per kilometer per meter in million euros

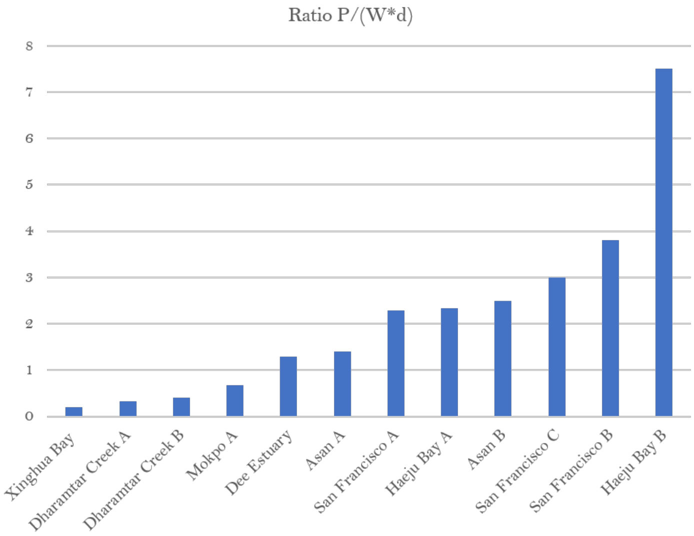


Figure 8.5: Perimeter to closure gap ratio

(a) Haeju Bay B, P = 45 km, W = 1 km, depth = 6 m

(b) Xinghua Bay, P = 130 km, W = 14 km, depth = 22 m

Figure 8.6: Locations with extreme perimeter to closure gap ratio

8.2. Comparison business cases alternatives

In this section the business cases of the three coastal defence alternatives are compared. This is done both for the design lifetime of 40 and 100 years. The results will also be presented for regional sea level rise. To understand why for some locations one solution is preferred over the other, a new parameter is introduced: the perimeter to closure gap ratio, $R_{\mathrm{c/d}}$. This parameter relates the ratio between the perimeter of the basin and the width and depth of the gap.

$$R_{\mathrm{c/d}} = \frac{P}{Wd} \tag{8.5}$$

With this ratio the basin morphology of the different basins can be compared on suitability for a dam. A large ratio indicates that a dam is favourable over updating coastal defences, a small ratio indicates the opposite. It should be noted that Asan C, Brouwersdam and Mokpo B have been excluded from this graph as there, a dam is already present.

From figure 8.5 it can seen that Haeju Bay B has the best configuration to build a dam, where Xinghua Bay has the least suitable basin configuration. These locations with their specific features are shown in figure 8.6.

The importance of this ratio changes over time as sea level rises. As was discussed in the previous section, the costs of each alternative are increased as sea level rises. For Haeju Bay B and Xinghua Bay the

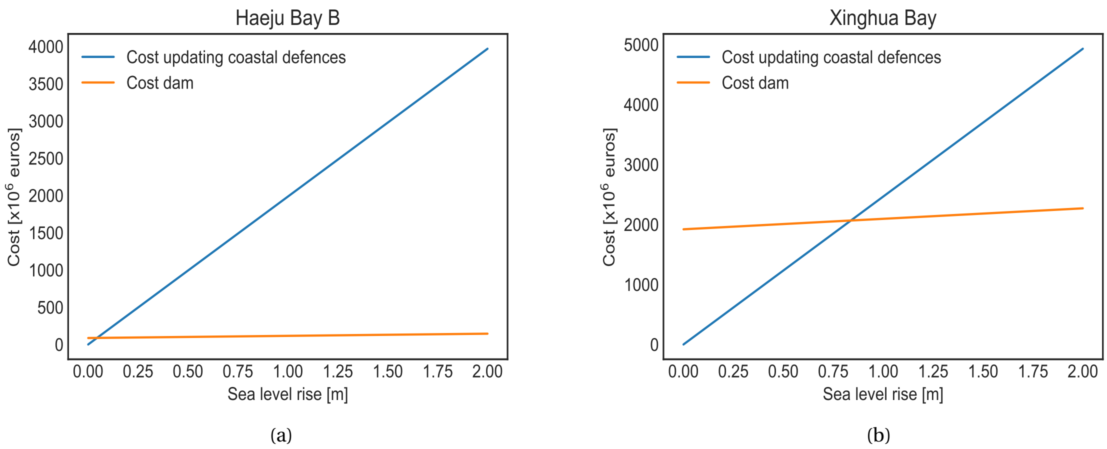


Figure 8.7: Costs dam and updating coastal defences for increasing sea level rise

cost development for a dam and updating the coastal defences is illustrated in figure 8.7. It can be seen that the costs increase for the coastal defences is more than for the dam per unit of sea level rise. The costs of a culvert or tidal power station are not included in the costs of the dam, this will cause a shift upwards of the orange line in both cases.

The best alternative will thus depend on the perimeter to closure gap ratio, the amount of sealevel rise, and the costs of the powerstation or culvert.

8.2.1. Design lifetime 40 years

For all locations only limited sea level rise is expected over the duration of 40 years. The expected sea level rise for the respective climate scenarios is still relatively close together after this period of time. It was shown in the previous chapter that the designs for design lifetime 40 years per location do not change significantly for the different climate scenarios. The results of all locations have been compared, and for each location and climate scenario the most economically favourable solution was inventoried. The result is shown in figure 8.8.

It can be seen that the most favourable solution per location is the same for each climate scenario. This was to be expected because of the narrow spread of sea level rise for the lifetime of 40 years. The results have been subdivided into the nature and economic strategy. From the image it becomes clear that the culvert is only an option if the economic strategy is applied. These locations include Asan C, Brouwersdam and Mokpo B, the locations where a dam was already present. Updating the coastal defences is the best solution for the four locations with the smallest perimeter to closure gap ratio ($R_{c/d}$). For locations with a relatively high return on investment the cost of a dam with culvert or dam with tidal power plant are very similar. The results for these locations are given in table 8.5. For Asan B the values are to close to give one preference over the other. The ratio between construction costs and cost of machinery is slightly in favour of machinery for Dee Estuary, which is why the for Dee Estuary the TPP is the preferable alternative. For Xinghua Bay the ratio of between construction costs and machinery is in favour of construction costs, which is why the culvert is cheaper than the TPP. However at this location because of the configuration of the basin it is cheapest to update the coastal defences (small $R_{c/d}$)

Table 8.5: Results business cases design lifetime 40 years locations ROI > 50%

	NPV culvert (x10^6)	NPV TPP (x10^6)	NPV coastal (x10^6)
Asan A economic	-1302	-1307	-2094
Asan B economic	-428	-426	-1212
Asan C economic	-33	-35	n.a.
Dee economic	-799	-788	-1131
Xinghua economic	-4867	-4992	-1232

When looking at the nature variants it becomes clear that the culvert is no longer a good option. Either

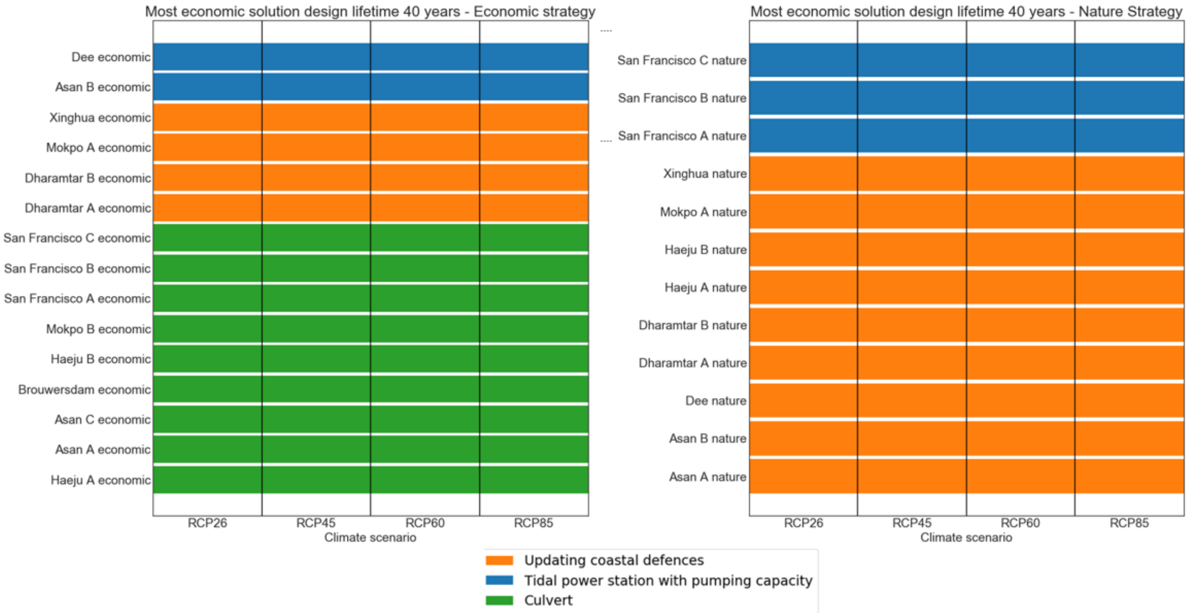


Figure 8.8: Most economic solution design lifetime 100 years, economic and nature strategy

an extremely large culvert is needed to maintain the stroke, or the tidal stroke at the lake can't be maintained over the duration of the lifetime without pumps. The tidal power stations needed to maintain the stroke is much more expensive than the economic counterpart. For limited sea level rise in most cases it is cheaper to update the coastal defences. Only for San Francisco Bay variants the dam with tidal power station is the preferred option. All variants have a relatively large perimeter to closure gap ratio and the costs of machinery relative to construction costs is in favour of the machinery. In this case the large investment of the tidal power station with pumping capacity is justified as the construction costs are lower.

8.2.2. Design lifetime 100 years

Figure 8.9 shows the result for the 100 years design lifetime. It can be seen that for none of the locations the culvert is now an appropriate solution. In most cases the tidal stroke cannot be maintained for a 100 years without pumps, and where it is possible, extremely large culverts are required.

When looking at the economic strategy it can be seen that for only three locations updating the coastal defences is the preferable solution. Xinghua Bay, Dharamtar A and Dharamtar B, the three locations with the lowest perimeter to closure gap ratio. These three locations also have relatively low construction costs compared to the cost to install machinery such as the pump/turbines. For RCP26 Mokpo A the coastal defences also deserve preference, after which the dam with TPP becomes more economical.

When looking at the results for the nature strategy, it can be seen that for the majority of the locations updating the coastal defences is more economic. With respect to design lifetime 40 years two locations have shifted. The dam with tidal power station and pumping is the most economical solution for the most severe climate scenarios at Haeju Bay B and Asan Bay B. These locations both have large perimeter to closure gap ratio, and Asan Bay has a relatively high return on investment. For all other locations the tidal powerstation with pumps is too expensive.

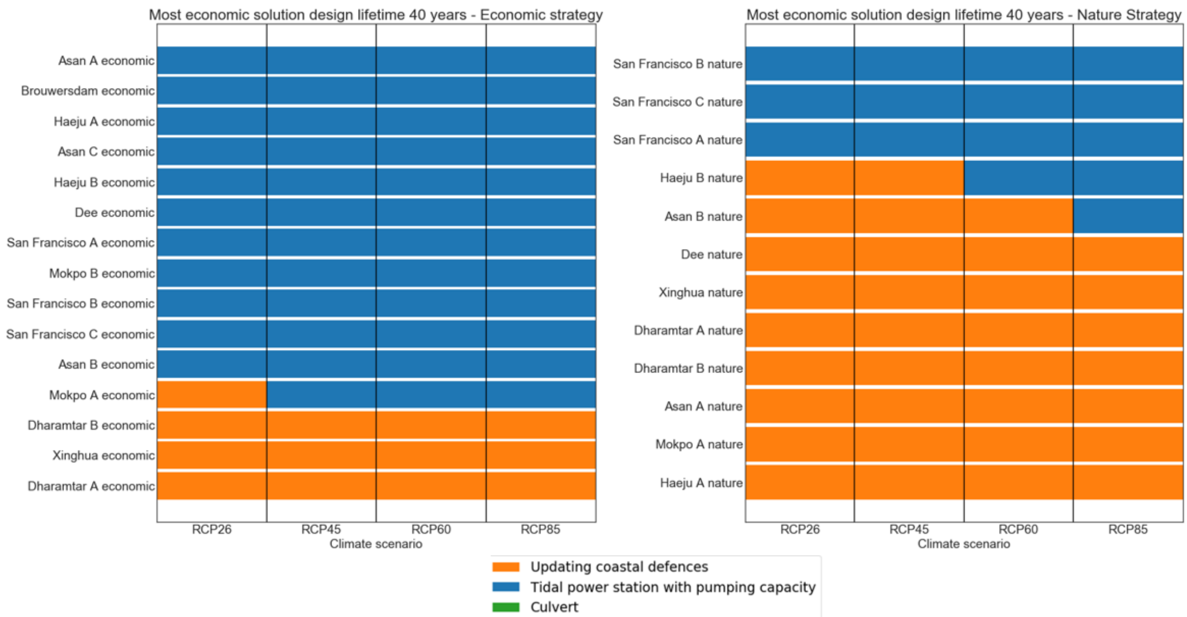


Figure 8.9: Most economic solution design lifetime 100 years, economic and nature strategy

8.2.3. Regional SLR

So far the analysis has been presented from the point of view of climate scenarios. In this section the results will be presented in a different manner: regional sea level rise. Figure 8.10 shows the amount of sea level rise corresponding to a certain climate scenario and lifetime. This figure uses global sea level rise (index = 1), regional sea level rise will deviate from this and can be scaled with per location to obtain regional sea level rise. This was discussed in section 2.3. This is thus a different manner to present the same results, as all optimised designs were also based on the expected regional sea level rise.

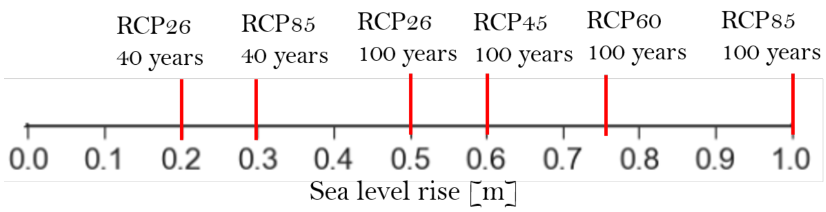


Figure 8.10: Link between Sea level rise and climate scenarios and lifetime

The results for all locations and strategies are shown in figure 8.11. This figure is a summary of the analysis of the 40 year and 100 year design lifetime business cases.

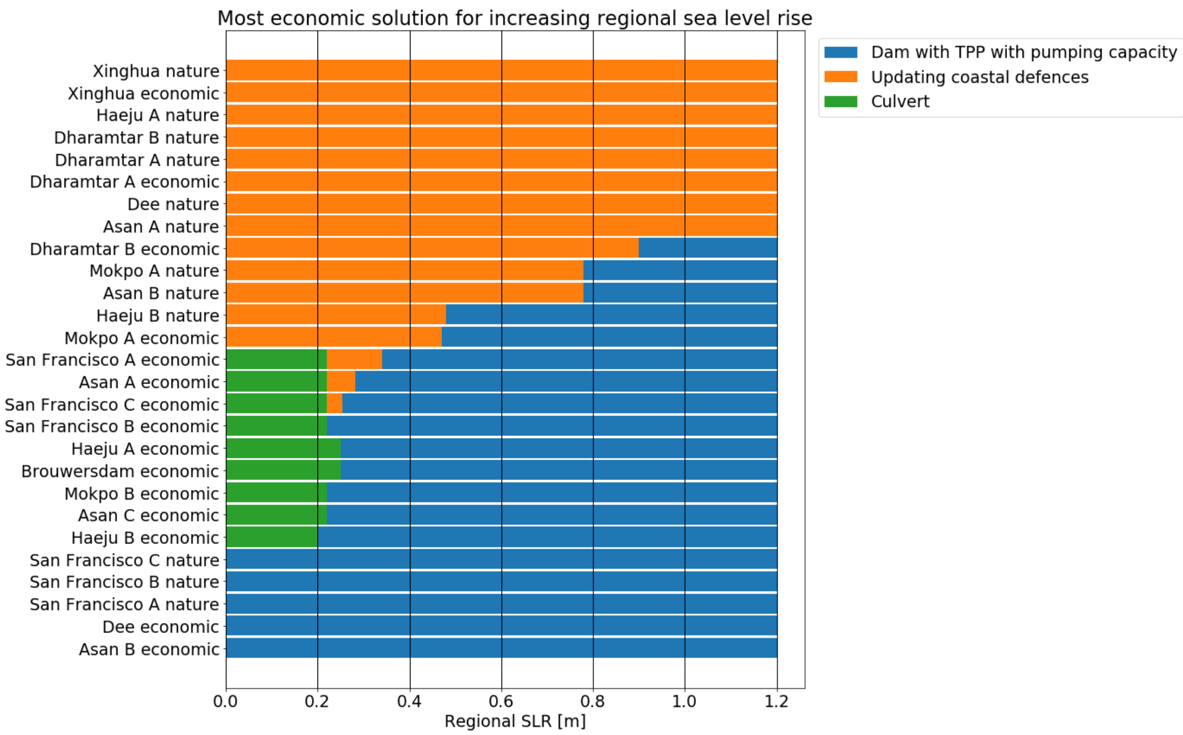


Figure 8.11: Results business cases for increasing regional sea level rise

In general it can be said that for limited amounts of sea level rise it is better to update coastal defences when the nature strategy is applied at a location (up to 0.2 meters sea level rise). When the economic strategy is applied and the perimeter to closure gap ratio is large, a dam with culvert is most favourable for this range of sea level rise. Beyond this amount of sea level rise a dam with culvert is no longer desirable. For these locations beyond this range it becomes favourable to implement a dam with tidal power station and pumps. Beyond 0.5 meters sea level rise locations with a nature strategy and a medium to large perimeter to closure gap ratio will also need a dam with tidal power station and pumping capacity. For locations with a small perimeter to closure gap ratio using the nature strategy, updating the coastal defences is the most favourable option up to a minimum of 1.2 meters sea level rise.

8.3. Export potential tidal power station with pumping capacity

With these insights a new assessment can be made about the export potential of a dam integrated with a tidal power station and pumping capacity. Section 3.1 describes the original study into this export potential. This global research based purely on spatial parameters concluded that there are 461 locations worldwide where the technology could be applicable. The detailed analysis in this study shows that the true number of locations is much smaller. In section 3.2 it was described how many of the locations were excluded. After visual inspection some of the locations were disqualified due to the presence of a large port or an unsuitable geometry. Some locations were defined multiple times, further reducing the number of potential locations. Only locations with high flood risk and high expected damages will require flood protection. With the threshold values applied in this study this left 81 potential locations. Of these locations eight were assessed in more detail. When it is assumed that this ratio for the analysed group is the same in the group of 81 potential locations a new estimation of the export potential can be made.

The results from this study show that depending on the strategy and lifetime of the structure that is desired the economic feasibility of the dam with tidal power station and thus the export potential of the Brouwersdam solution varies. On the short term (40 years), if a large reduction of the tidal stroke is allowed, the solution is feasible at two of the analysed locations. If only a small reduction of the tidal stroke is allowed only 1 analysed suitable location is left. , the export potential ranges from 10 to 20 locations as a short term solution.

When considering a long term solution (100 years) the export potential is different. If a large reduction of the tidal stroke is allowed the tidal power station with pumps is an economically the most advantageous long term solution for 6 of the 8 considered locations. A large reduction of the tidal stroke however will mean that still the basin ecology is significantly altered, but the water quality is guaranteed. In case the valuation of the nature is such that this is unacceptable, it is plausible that a more conventional flood protection measure will be taken.

If only a minimum reduction of the tidal stroke is allowed, the dam with tidal power station is a suitable solution for 1 of the 8 locations when considering mild climate scenarios RCP26 and RCP45. For climate scenario RCP85 the solution is applicable in 3 out of 8 locations. Since for the larger tidal stroke the disruptive effect on nature is minimal, this alternative is less likely to be rejected for ecological reasons.

Table 8.6: Export potential Brouwersdam solution per climate scenario

	RCP26 - RCP45	**RCP60**	**RCP85**
Long term solution (nr of locations)	10-60	20-60	30-60
Short term solution (nr of locations)		10-20	

CHAPTER
NINE

CASE STUDY: SAN FRANCISCO SOUTH BAY

9.1. Introduction

In this chapter one of the variants is analysed in more detail: San Francisco Nature variant B. This case study illustrates the multitude of factors that can influence the feasibility of the tidal power station with pumping capacity, which extend beyond the scope of the factors taken into account by the model in this thesis.

The San Francisco Bay area includes a number of coastal cities, with a cumulative population of approximately 7.8 million people. Flooding is a major concern in the area due to the economic damages, the rising sea level maakes the area more susceptible to flooding in the future. Herberger et al. (2012) estimates that " a 1.0 meter (m) sea level rise will put 220,000 people at risk of a 100-year flood event, given today's population. With a 1.4 m increase in sea levels, the number of people at risk of a 100-year flood event would rise to 270,000. Among those affected are large numbers of low-income people and communities of color, which are especially vulnerable. Critical infrastructure, such as roads, hospitals, schools, emergency facilities, wastewater treatment plants, power plants, and more will be at increased risk of inundation, as will vast areas of wetlands and other natural ecosystems." Figure 9.1 shows the areas at risk in the South Bay. The damages to property associated with the flooding events linked to the rise of the sea level are $49 billion US dollars for 1 meter sea level rise and $62 billion US dollars for 1.4 meters sea level rise. It is therefore pertinent that an adequate flood protection strategy is adapted, that takes into account future conditions at the regional scale.

The Bay has many features which are relevant to the decision of which strategy to adapt. A few will be highlighted in this section: technical challenges of constructing in the Bay, important infrastructure and the presence of wetlands. Two alternatives are evaluated: The design of the tidal power station with pumping capacity and updating current coastal protection around the perimeter of the South Bay. To be able to carry out a sound comparison, both alternatives should be evaluated for the same amount of sea level rise. As a point of reference 1 meter sea level rise is chosen (choice based on data availability). This corresponds to the most severe scenario according to the IPCC in 100 years: RCP85.

Finally, the alternative with the integrated dam tidal/pumping station is put into historical perspective. In the past there have been proposals to drastically change the outlook of the Bay, most importantly the 'Reber Plan'. As a reaction to the Reber Plan a movement started that is focused on conservation of the San Francisco Bay and recognizes its ecological value. The Reber Plan serves as a pilot study of sorts, and an estimation can be made whether the objections to the Reber Plan are overcome by the proposed solution.

9.1.1. Bay features

Construction challenges

The bay is located between two active faults: San Andreas and Hayward fault, and is therefore situated in a seismically active region. This has consequences for the design of civil works in the area. Structures need to be earthquake resistant, which will generally mean a more expensive design. For the caissons which function as the housing of the pump turbines this also applies. Besides soil structure interaction caused by seismic

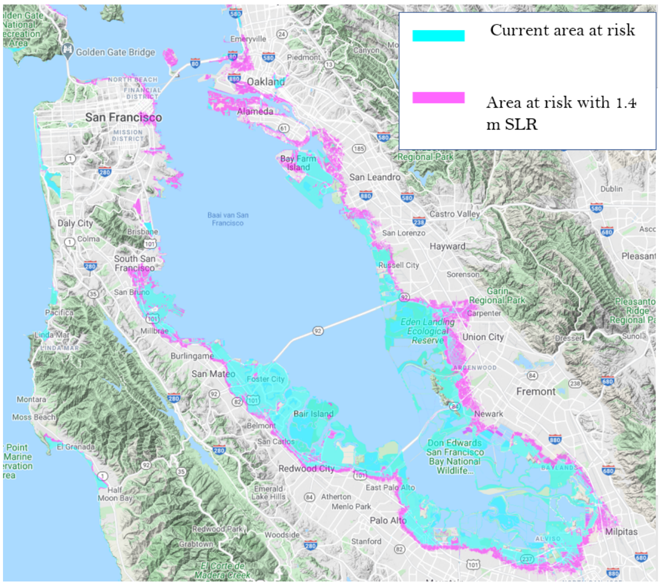


Figure 9.1: Areas at risk from a 100 year coastal flood event (Herberger et al. (2009))

waves, fluid structure interaction can cause additional vibrations. The location correction factor used for construction costs in San Francisco partially captures this aspect. This is however a generalised factor based mostly on land based construction. (Arcadis, 2019) In the case of a flood barrier the consequences of failure are very large. Most likely only very small deformation is allowed. This can lead to even higher project costs than initially anticipated.

The San Francisco Bay subsoil is made up of soft clay. Construction of a dam on previously unloaded soil will lead to a settlement of the structure, as the water drains from the clay layers. This settlement process can continue for many years and it is likely that in the first few decades regular maintenance and or heightening of the dam will have to be applied. Since the clay will not be homogeneous along the cross section, differential settlements can also be expected. The dimensioning of the dam will have to be such that this does not prove problematic. For an expensive structure such as the powerhouse of the tidal power/pumping station no differential settlements can be accepted. At the tidal station therefore an elaborate foundation will have to be installed. This will have a significant impact on the costs of the structure. If an alternative is chosen where the current flood protections are adjusted to meet future needs this will have less of an impact because the subsoil has been previously loaded. However when a dike is heightened still some settlements can be expected.

Infrastructure

There is a lot of infrastructure in and around the Bay Area. Figure 9.2 shows some of the most important infrastructural points in the Bay.

The port of Oakland is a large port, and needs to remain fully operational as a result of any intervention. When a dam is placed it is therefore assumed that the orientation is such that it does not interfere with the port activities. Other ports in the South Bay are relatively small in comparison. These ports still need to be accessible but it is estimated that a lock in a dam would have enough capacity for the expected traffic to these ports.

Both San Francisco and Oakland airport are at risk from flooding and are taking measures to withstand sea level rise. San Francisco Airport is to construct a 16 kilometer long sea wall around its perimeter, at a cost of 587 million US dollars. Oakland Airport is investing 46 million US dollars to raise the dikes along the runway by 0.6 meters. Rogers (2019)

Finally there is a submerged rail tunnel connecting San Francisco to Oakland. There are concerns that the tunnel is not resistant to a very large (1 in a 1000 years event) earthquake. For this reason a lot of investments has been done to update the tunnel, most recently in 2019 a 313 million US dollar project has started for seismic retrofitting of the tunnel. In case a dam would be built this would provide a opportunity to connect Oakland and San Francisco in an alternative way. This is not taken into account in the cost benefit analysis, but could have a significant impact.

Figure 9.2: Important infrastructural points in the South Bay

Wetlands

Wetlands are an important feature of the Bay, providing many ecosystem services. These include providing a breeding ground for many species, flood protection and carbon sequestration. Engle (2011). Over the past 150 years approximately 90% of the approximately 800 km^2 of wetlands in the Bay has disappeared or has been severely degraded. A rising sea level puts the current wetlands in the Bay under pressure as parts will be permanently submerged. Since the foundation of the 'Save the Bay' initiative in 1961 there have been some projects to restore some of the lost wetlands. In the South Bay around 60 km^2 of industrial salt pond are being restored to wetlands. N.N. (2013). The valuation of the wetlands in monetary terms is very difficult. For the South Bay it was estimated that the restoration costs around 16 million dollars per square kilometer. This reflects the willingness of the people to conserve and restore the wetlands. Herberger et al. (2009).

9.2. Solution for the South Bay

9.2.1. Alternative I: Dam with tidal power station with pumping capacity

The first alternative is the dam with tidal power station with pumping capacity. The nature nature based strategy is able to sustain a minimum tidal stroke of 1.2 meter (reduction of 0.5 meters) for a 100 years. To achieve this 218 turbines of 10 meter diameter are needed. With an estimated spacing of approximately 5 meters between the turbines the span of the tidal power station with pumps is around 3.3 km. Dredging works will be needed to create a suitable subsurface for the tidal power station with pumping capacity. Dredged sediment can be used in the south of the Bay towards wetland restoration, or as raw material for the remainder of the dam. With the total span of the dam estimated at 7 km this means 3.7 km of will be covered by a regular dam.

Table 9.1 gives an overview of the costs associated with this alternative. The costs for the station and dam is given in the net present value. The maintenance costs, profits for energy yield and the costs for pumping energy are thus incorporated. Sediment supplements to the wetlands that might be needed once the dam is built was not included in the calculations. As can be seen in the table the Net Present Value of the tidal power station with pumping capacity is lower than the investment costs, this is attributed to maintenance costs and costs of pumping energy. The average costs of pumping energy annually is higher than the profits made by generating energy. It can be seen that the costs for the tidal station make up the largest part of the overall project costs: 93%. Some benefits have not been taken into account: no monetary value has been given to the intertidal flats that can be maintained by the solution. Furthermore the value of the flood risk reduction has also not been taken into account.

Table 9.1

NPV Station [M€]	NPV Dam [M€]	Average annual yield energy [M€]	Average annual costs pumping energy [M€]	Compensation costs nature [M€]	total [M€]
-7913	-413	675	1078	110	-8216

9.2.2. Alternative II: Updating coastal defense

One option to increase flood safety when sea level rises is to update the flood protection around the coastline to be able to withstand flooding events with sea level rise. In the bay area there is already a system of dikes and other flood protection structures in place, but these will need to be updated as sea level rises. Figure 9.3 shows the an overview of protection types around the perimeter of the South Bay as well as infrastructure. In the area of interest approximately 35 kilometers of dike is present, 65 kilometers of shoreline protection structure and 80 kilometers of berm.

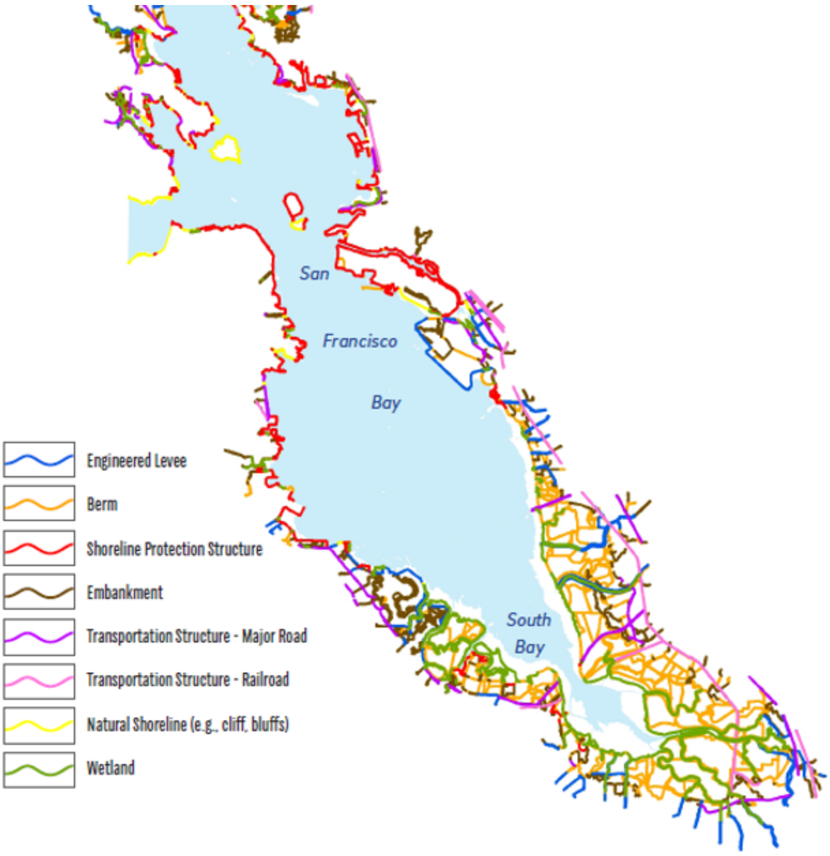


Figure 9.3: South Bay perimeter inventory San Francisco Estuary Institute-Aquatic Science Centre (2016)

Hirschfel and Hill (2017) makes an estimation of the costs of adapting the coast of the Bay for 1 meter sea level rise. This study uses the same inventory as figure 9.3, and makes a distinction in costs for updating the different structure types, and also includes parcel costs when more surface area is needed to heighten the dikes. Berms/wetlands have been omitted from the analysis. The study estimates that the total costs to adapt to 1 m SLR will be range between 36 tot 83 billion US dollar (2020 price level) for the entire Bay. The area of interest for this analysis is only the South Bay and part of the central Bay, this amounts to about 40% of the Bay. San Francisco Estuary Institute-Aquatic Science Centre (2016). The costs thus become 14.4 to 33.2 billion US dollars for the area of interest. These costs are the investment costs required and do not include operational and maintenance costs.

Again the benefit of increased flood protection is not taken into account in terms of money. The effects on ecology have also not been included in the analysis, unlike the alternative with Dam, this solution allows for sea level rise in the basin. This means that the wetlands at their current level will partially drown and disappear. However, as there is still a tide, the wetlands will partially move to further inland where this is acceptable.

9.2.3. Comparison alternatives

The two alternatives are compared through a very general multi criteria analysis. The scoring parameters are flood protection, costs and nature conservation. Both solutions are effective in serving as flood protection and can guarantee the same level of safety. Both costs estimations contain a lot of uncertainty, it can however be said that the dam with tidal power station will be the cheaper alternative. When the most optimistic cost estimation for the updating of the coastal defences is used (14.4 billion US dollars) this is still 6 billion dollars less than the estimated costs of the tidal power station. The two alternatives both have a similar score for the nature. In both alternatives the water quality in the Bay is guaranteed. In the alternative with dam some of the wetlands will be partially lost due to the reduction in tidal stroke, while in the alternative with the updated defences some wetlands will partially be lost because of the rising sea level.

Based on these considerations the alternative with dam and tidal power station with pumping capacity is the most beneficial due to the lower expected costs.

Table 9.2: Multi criteria analysis flood defences San Francisco Bay variant B

	Flood protection	**Cost**	**Nature**
Dam	++	+	+
Updating flood defences	++	-	+

9.3. Historical perspective: The Reber Plan

9.3.1. Reber Plan

The main elements of the Reber Plan were to close off the North and South Bay, while part of the East Bay is reclaimed. Several infrastructural elements were incorporated which would improve transportation options in the Bay. The Reber plan is shown in figure 9.4. The goal of the plan was to ensure a large freshwater supply for residents of the Bay area as well as for agriculture, and furthermore facilitate growth of both the population and the economy. The Reber Plan was proposed in 1929 and finally discarded in 1963. During it's lifetime it has had many supporters and opponents.

The city of San Francisco was one of the fervent supporters of the plan, as it would increase the accessibility to the city and would theoretically supply unlimited fresh water. Their commercial port would remain in open connection with the salt water, this was an advantage over other ports behind the barriers. Amongst the opponents were some of the other ports that would be behind the barrier or landfill, such as the port of Oakland. Vessels wanting to leave or access the port would have to go through a series of locks, causing significant delays. California farmers were strong supporters of the plan, as it would supply them with cheap access to irrigation water. Some of the delta farmers worried however that the winter storms would cause a rising of the water level behind the dam and put pressure on the dikes. Another concern was that the closed of lakes would become polluted.

Notably, environmental agencies were not one of the key opponents. Established conservation agencies at the time were mostly concerned with conserving untouched nature instead of the Bay where human activity had already touched and altered the nature. The plan however did set in motion the 'Save the Bay' movement, which intends to conserve and restore bay ecology. This has gotten a lot more attention since the plan and policies and regulations are now in place to preserve the nature in the Bay. This means that future plans which harm or significantly alter the Bay ecology can expect a lot of push back from environmental agencies today.

The plan gained so much traction that a series of studies were performed to assess its feasibility. Most importantly: the Congress awarded a budget of 2.5 million US dollars to the US Army Corps of Engineers to perform a study on the Reber Plan. These studies all came to the same findings: the costs of the project were severely underestimated, furthermore during the summer months water levels in the lakes could become critically low while in the winter months they could rise and cause flooding. Pollution of the lake would be another serious issue. Furthermore the plan would destroy several industries that were important to the Bay, such as the salt mining and fishing industry.

Ultimately the plan was no longer considered and the California state went in a different direction to ensure access to fresh water for the bay area. The 'Central Valley Project' fulfilled this purpose with a series of dams in the Sacramento and San Joaquin Rivers and pumped water from the delta's. This project was more feasible than the Reber Plan, and had less of a disruptive impact on Bay industries. (Wollenberg, 2014)

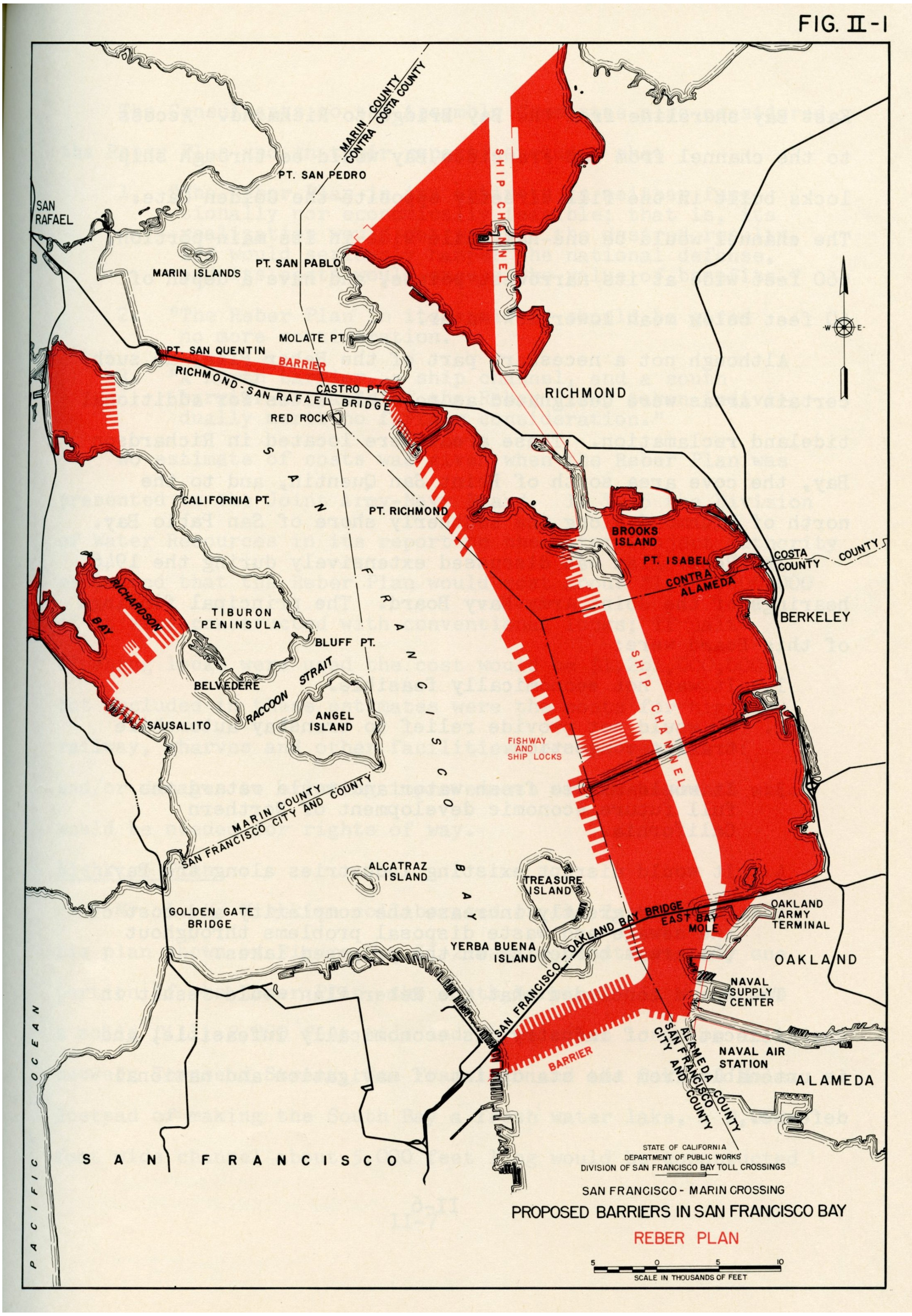


Figure 9.4: Reber plan from Wollenberg (2014)

9.3.2. Comparison Dam with tidal power station and pumping capacity

The Reber Plan did not pass for several reasons, the most important being: lack of technical feasibility, disruptive effect on Bay industries and the presence of a more favourable alternative. When looking at the Dam with tidal powerstation and pumping capacity, many of these aspects are overcome. The aim now is not a freshwater supply but flood protection. The created lake is still a salt water lake, and by including the tidal power station with pumping capacity the identified problems of the Reber plan: low water in summer, high water in winter and pollution of the lake are no longer an issue.

Parties that were in favour of the Reber Plan such as the city of San Francisco can be expected to also be in favour of this proposal. The dam will provide improved access to the city. Farmers can also be expected to be in favour of this proposal, but for a different reason. The dam with tidal power station and pumping capacity is able to maintain the mean water level at a fixed position, where it would otherwise rise with the sea level. A higher mean water level means more salt intrusion in the soil and groundwater, which have a negative effect on agriculture. This is prevented by the dam.

In this plan the dam does not interfere with the activities of the port of Oakland, so it is not expected that the port of Oakland will be opposed to the solution. Furthermore the dam brings the city of Oakland in direct contact with the city of San Francisco, improving its accessibility. The San Francisco and Oakland airport can both be expected to profit from the project as it provides them with flood protection. The salt mining industries in the South Bay have already been scaled back and have partially been reclaimed to restore wetlands, it is not expected that the Dam will have any impact on this industry.

Because the dam will include a large lock, fishing boats can still reach open seas from the South Bay. The pump turbines do provide a hazard for fish who would swim through them. For this reason it can be investigated to install 'fish friendly' pump/turbines. It could also be considered to install a barrier for the fish (e.g. nets) in front of the pump/turbines, and including a fish passage at another point in the dam. The plan can expect opposition from this party.

Standards for ecology in the Bay are much higher now than they were during the period of the Reber Plan. The dam with tidal power station and pumping capacity however is in large part focused on preserving Bay ecology where possible. The water quality will be of sufficient during the lifetime. The solution keeps the mean water level at the current level, with a reduced tide, this will cause part of the wetlands in tact, given that sediment nourishments will be supplied. The alternative, of updating coastal defense structures can not prevent part of the wetlands from disappearing, as they will drown under as sea level rises.

Which leads to the final point: where there was a more attractive alternative for the Reber plan, this is not the case for the dam with tidal power station and pumping capacity. The alternative with a dam can assure flood protection, performs well on preservation of the Bay ecology, and is the more financially attractive alternative.

CHAPTER
TEN

CONCLUSION

A dam with tidal power station and pumping capacity presents a new alternative to protect coastal regions from sea level rise induced flooding. This solution is able to keep the mean water level in the closed basin at a desired level, while maintaining a reduced tidal stroke and generating energy. This is an improvement on a traditional closure dam, where disruptive effects on ecology have caused it to be excluded from most flood protection considerations. As sea level rises the pumps are needed to artificially maintain the tidal stroke in the basin, which will have a lower mean water level than the mean water level at sea. Through the analysis performed in this study more insight is gained under which circumstances and sea level rise scenarios this new solution becomes an attractive alternative in comparison to conventional flood protections such as dikes and flood walls around the perimeter of the basin.

Eight locations with high flood exposure and a large population were chosen as locations with large potential for the solution. These locations were further analysed through computer models and a series of business cases.

Two models were developed in this study: a hydro-energetic cost model that can calculate the energy yield, pumping requirements, tidal stroke and costs for a tidal power station with pumps. The second model generates an optimal design of a tidal power station with pumps for a given location based on some key features (most importantly the number and size of the pump turbines) and a tidal signal. Both models were validated against the current design of the Brouwersdam. The results of the hydro energetic cost model were in good agreement with previously found results, and the optimization model was successful in finding a more favourable design of the Brouwersdam than currently considered.

It was found that the minimum tidal stroke at the Grevelingenlake could be maintained for 42 centimeters of sea level rise with the original design (12 pump turbines with 8 meters diameter). This is more than was calculated in previous studies where pumps were only used in flushing mode. Pumps in the new model are able to pump both in flushing mode and against a head difference. The optimization model found a different optimal design from the original, where the tidal stroke can be maintained up to 0.5 meters sea level rise and with a 9% less negative net present value. The most economically favourable solution for the range of sea level rise considered in the original study, 0.28 meters, was found to be a set of 18 culverts of 8 x 8 meters, at 70% of the investment costs of the original design.

It was found that the number and diameter of the pump/turbines were most influential on the energy yield, tidal stroke and costs of the structure. The number and diameter of the pump/turbines result in a certain flow area through which the water is able to pass the barrier. A minimum flow area is needed to facilitate the desired tidal stroke at the basin at zero meters sea level rise. Without using pumps the energy yield and tidal stroke at the lake reduces as sea level rises. Until 0.5 meters sea level rise the pumps corresponding to this baseline flow area are effective in maintaining the tidal stroke. Additional pumps are needed above this minimum in order to maintain the tidal stroke for a longer period and more sea level rise.

While this new solution has the potential to 'pay for itself' as energy can be generated by the tidal power station, this study has shown that this is not the case for practical applications. At the beginning of the lifetime the net energy yield is positive and an income is generated. As sea level rises, more energy is needed for the pumps to maintain the artificial tidal stroke. The generated income is reduced or even becomes negative, i.e. pumping costs more energy than is gained by turbining. Furthermore, the annual maintenance and operational costs of the structure were found to be in the same order of magnitude as the net annual energy yield, resulting in at best a marginal profit. It was found that primarily the uniformity of the tide and secondarily the amplitude of the tides were indicators of the return on investment. A large daily inequality in the tidal amplitude negatively affects the energy yield in two ways: Firstly, the turbine operates best at the design head, for a non uniform signal the design head will be a compromise between the two amplitudes and during neither tidal cycle during the day will the turbine operate at maximum efficiency. Secondly, pumping will already be needed for a small amount of sea level rise during the low amplitude tides. The resulting net energy yield is low or negative. A uniform tide with a large tidal amplitude will yield a larger profit than a uniform tide with a smaller tidal amplitude. In similar fashion a non-uniform tide with a large tidal amplitude will have a smaller net loss than a non-uniform tide with a smaller net amplitude. The highest return on investment (55 %) was found for Asan Bay in South Korea, a location with a uniform tide with and a mean tidal range of 6 meters, and the lowest return on investment (-30%) was found for San Francisco Bay, a location with a non uniform tide with and a mean tidal range of 1.7 meters. The investment is thus only partially returned at best.

While the solution is not a positive business case in itself, the study has shown that the solution can be the most economically attractive alternative to protect a coastal zone against flooding in an ecologically friendly manner, compared to updating coastal defences or a dam with simple culvert. The two most important considerations are the expected sea level rise and the desired tidal stroke in the basin. Two scenarios were considered for the required tidal stroke on the basin after closure: an economic scenario in which the tidal stroke may be reduced to the point where it is just sufficient to refresh water in the deeper parts of the basin (5 - 25 % of the original stroke depending on basin bathymetry), and a nature preservation scenario in which at least 80% of the currently present tidal stroke is maintained. Table 10.1 summarizes the best alternative for the cases assessed in the study.

Table 10.1: Best coastal defence alternative for sea level rise and tidal stroke

	Strongly reduced stroke (5 - 25% original)	**Mildly reduced stroke (80% original)**
Limited SLR (0 -25 cm)	Mixed: Dam with culvert or upgrading dikes and floodwalls	Upgrading dikes and floodwalls
Mild SLR (25 - 60 cm)	Dam with tidal power plant and pumps	Upgrading dikes and floodwalls
Severe SLR (60 - 100 cm)	Dam with tidal power plant and pumps	Dam with tidal power plant and pumps

For **limited sea level rise and a strong reduction in the tide** both the dam and updating dikes and floodwalls are suitable alternatives. In this situation pumps are not needed to maintain the tidal stroke, making a dam with culvert the cheaper dam option. When the tidal power station has a relatively large return on the investment (50% or more) the business case of the dam with culvert and dam with tidal power station are very close together. Whether a dam or dikes and flood walls are preferable is dependent on parameters such as the perimeter of the basin versus the width and depth of the closure gap.

For **limited sea level rise and the original tidal stroke mildly reduced**, the most cost efficient alternative is to update the coastal defences. When a dam is placed, very large culverts are needed or a tidal power station, these elements make the dam very expensive. On average the coastal defences are heightened by 0.5 meters for this range of sea level rise, in most of the cases this is the cheaper alternative. This changes as more sea level rise is expected and the coastal defences will need to be heightened more.

The results thus show that for limited sea level rise (0- 25 cm) the solution of a dam with tidal powerstation with pumping capacity is not the optimal solution. This is the range of sea level rise that is expected in the next 40 years. This means that the dam with tidal power station is not suitable as a short term solution.

For **mild sea level rise and and a strong reduction in the tide** the dam with tidal power plant and pumps becomes the most attractive alternative. Culverts are no longer sufficient to maintain the tidal stroke over the lifetime of the structure, so some pumps are needed, but a relatively small station suffices. The costs of upgrading the coastal defences are doubled for this range of sea level rise compared to the situation of limited sea level rise. This also holds for **severe sea level rise and a strongly reduced stroke**, where the difference becomes even more pronounced.

For **mild sea level rise and the original tidal stroke mildly reduced** a large station is needed, this is a very cost intensive element of the dam. Even for locations where a partial return on investment is made this does not compensate for the absolute increase in cost of a large tidal power station. In this case it is more cost efficient to adapt the existing coastal defenses eventhough the costs of this solution have doubled with respect to limited sea level rise.

Only for **severe sea level rise and a mildly reduced stroke** does the investment in a large tidal power station with pumps become favourable compared to the option to update coastal defences.

These conclusions are valid for most basins included in the analysis, however there are some basin characteristics that may lead to a different most suitable solution for a certain tidal stroke and sea level rise. For locations where labour and construction costs are relatively low updating the coastal defences will be the favoured alternative even for severe sea level rise. This is because the investment in machinery is relatively expensive compared to choosing a more labour intensive alternative. This also works in the opposite direction for locations with high labour and construction costs, a dam with tidal power station and pumps will sooner be the favoured solution.

The ratio between the perimeter of the basin and the closure gap width and depth is also of influence. If the perimeter is very large compared to the closure gap, the dam with TPP and pumps will be the favoured alternative already for small amounts of sea level rise. A large closure gap compared to basin perimeter results in the opposite: updating the coastal defences will be the preferred solution also for large amounts of sea level rise.

A more detailed case study of the San Francisco Bay supported the findings of the business cases. The case study identified added benefits of the dam such as transportation that contributed to the desirability of the alternative. A dam with tidal power station is a feasible solution for the San Francisco South Bay.

The study has shown that the export potential of the solution is significantly lower than the 461 locations initially estimated by Deltares (2019). Through analysis of the dataset it became clear that some of the data was inaccurate, basins were defined multiple times and the scoring system was skewed. This study concludes based on extrapolation of findings of a selected number of case studies that this number will be around 60 locations at most. Table 10.2 summarizes the number of potential locations depending on the climate scenario used and timescale are used.

Table 10.2: Number of estimated cases for which a dam including a tidal power station with pumps could be an interesting solution per climate scenario

	RCP26-45	**RCP60**	**RCP85**
Nummer of locations Deltares		461	
Number of locations short term(40 y) this study		10-20	
Number of locations long term (100 y) this study	10-60	20-60	30-60

RCP: Representative Concentration Pathway. Climate scenarios as defined by the Intergovernmental Panel on Climate Change (IPCC). RCP26 is a low greenhouse gas emission scenario, RCP45 and RCP60 are intermediate scenario and RCP85 is a high emission scenario

CHAPTER
ELEVEN

RECOMMENDATIONS

This study was aimed to research the applicability and economical feasibility of a tidal power station with pumping capacity. The results give a good outlook about under which circumstances this is the best solution. To further improve these findings the following topics can be investigated:

11.1. Improvement of the model

- Fixed parameters are used in the hydro energetic cost model to indicate the moment of turbining, and pumping. The parameters used in the model are optimized to yield the best results for as many tidal cycles as possible. However as the tidal signal is not constant over time, the chosen parameters will lead to poor timing in some cycles. It would be a valuable addition to the model when it can predict the best moments for pumping and turbining, anticipating the next tide. This would lead to more energy yield, less pumping costs and a longer lifetime. Since the tide at a location is predictable this should be feasible and could be achieved by incorporating machine learning into the model.
- If the optimization model were to be used for a very large number of locations, it could be worthwhile to investigate the use of computationally more efficient non-linear optimization methods. However, the applicability of such methods with or without derivatives is a whole field of its own.
- The influence that a river draining on the basin will have on the water level in a basin was not considered. If the river discharge is very large this can have a significant impact on the results, especially concerning the energy generation and required pumping capacity.

11.2. Next steps in feasibility research

- More insight in the export potential of the solution can be gained when a new inventory is made of potential locations. This new inventory should be a corrected version of the previous study, where unreliable data is removed. The selected locations should have a high flood risk, and large damages, both population and GDP are a good indicator of the damages. Areas with large regional sea level rise are particularly interesting, as well as areas that have high construction and labour costs. Valuable features of the basin are the perimeter and the length and depth of the closure gap. The basins of interest have a relatively large perimeter compared to the width and depth of the closure gap. Valuable features of the tidal signal are the mean tidal range, the relative daily inequality and the spring to neaptide ratio. A uniform tidal signal will lead to a larger return on the investment of the tidal pumping station.
- More research should be done on the effects of the reduction of the tidal stroke in a basin. Sedimentation patterns within the basin will change and this can have undesired consequences on the basin morphology and ecology. The needed dredging works and nourishments that are needed in order to maintain the nature within the basin can have a significant impact on the costs and feasibility of the solution.

- There are several aspects of the pump turbine that can be further investigated to improve insight on feasibility of the solution.
  - The relationship between the minimum head difference over the turbines and the turbine diameter
  - The expected lifetime of the pump/turbine. The solution is only interesting if it is applied for the long term, which would also require a long life expectancy of the pump/turbines.
  - The safety for both fish, marine mammals and other creatures to pass through the structure, both while it is in turbining and pumping mode. This will most likely be the largest hurdle to overcome from an ecological point of view.

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APPENDIX
A

REQUIREMENTS WATERLEVEL

A.1. IUCN protected areas

The local policy on nature preservation is important for the decision making on a water level management strategy. There are many nature preservation regulations on local scale. For this project use is made of the global database of The International Union for Conservation of Nature (IUCN). The IUCN has a large database of protected areas, subdivided into different categories. The categories differ on level of protection and function of the area. The categories are explained in table A.1. Ia being the most strictly protected category. Locations that are protected will have stricter requirements on nature value and water quality compared to areas which are not protected. Locations qualifying for the solution in this project are found to either not be protected or of category IV to VI. The Grevelingenlake is an example of a category IV protected area.

Table A.1: IUCN protected area classification

Category IUCN	Description
Ia	Strict nature reserve
Ib	Wilderness area
II	Ecosystem conservation and protection
III	Conservation of natural features
IV	Conservation through active management
V	Landscape/seascape conservation and recreation
VI	Sustainable use of natural resources

Categories IV to VI are defined by the IUCN as: Dudley (2008)

- "**Category IV** protected areas aim to protect particular species or habitats and management reflects this priority. Many category IV protected areas will need regular, active interventions to address the requirements of particular species or to maintain habitats, but this is not a requirement of the category."
- "**Category V** A protected area where the interaction of people and nature over time has produced an area of distinct character with significant ecological, biological, cultural and scenic value: and where safeguarding the integrity of this interaction is vital to protecting and sustaining the area and its associated nature conservation and other values. "
- "**Category VI** protected areas conserve ecosystems and habitats, together with associated cultural values and traditional natural resource management systems. They are generally large, with most of the area in a natural condition, where a proportion is under sustainable natural resource management and where low-level non-industrial use of natural resources compatible with nature conservation is seen as one of the main aims of the area."

A.2. Water quality and a reduced tide

The water quality in the lake is dependent on the flow of water within the lake. Important for good water quality are the absence of hypoxic and hypertrophic zones. The properties of the water should be such that the desired species can thrive and it is safe for recreational purposes if desired.

As a metric of the water quality in the Brouwersdam the oxygen levels are used. For the Brouwersdam a 40 cm tidal stroke is the minimum stroke where the water quality is improved in the deeper parts, meaning the oxygen level stays above a critical value. The outer limit is 35 cm, while 50 cm would be optimal. (Grevelingen, 2019) These limits were established through models. The minimum tidal stroke required to maintain the standard of water quality is much smaller than the original tidal stroke before the Brouwersdam was placed, namely 2.3 meters.

The results of this research for the Brouwersdam will be used to derive an estimation of the required minimum tidal stroke at any location. The tidal stroke serves to increase oxygen levels in deeper parts of the basin. The 40 cm stroke according to the study by Deltares is sufficient to reach the deeper parts of the Grevelingenlake, which are 22 meters deep on average. The minimum tidal stroke to guarantee sufficient water quality in each location then becomes:

$$R_{\text{min}} = \frac{0.4}{22} d_{d_{\text{avg}}} \tag{A.1}$$

With R_{min} the minimum tidal stroke in meters
With $d_{d_{\text{avg}}}$ the average depth of the deepest parts in meters

This relationship is used as an assumption of the required minimum tidal stroke. However models should be used to simulate the influence of the tidal stroke on the water quality at a considered location. The detailed modelling of the water quality in the basins is beyond the scope of this research, but should be done for a detailed design.

A.3. Intertidal character basin

When a dam with an inlet is placed, the equilibrium situation is disturbed. This will have an effect on morphology through e.g. altered sediment transport patterns. The adjustments to a new equilibrium situation happen on a very large timescale and the effects are sometimes difficult to foresee. Extensive modelling of each basin is needed to make predictions about this aspect, which is beyond the scope of this research. It can be said however that the tidal stroke at the lake should resemble the current situation as much as possible (around same tidal amplitude). This preserves the intertidal character of the intertidal flats and nature around the waterline, which is inundated during high tide and exposed during low tide. Furthermore by observing the developments in the basin additional measures can be taken such as dredging operations or sediment supplements.

The extent in which losses in the intertidal character are accepted will depend on local policies and nature preservation regulations. It can be considered to compensate for the habitat loss by creating new habitats elsewhere. A balance needs to be made between project costs, lifetime of the structure, value of the ecosystem and possible additional mitigation measures. The valuation of each ecosystem in monetary terms is beyond the scope of this project. An assumption for nature mitigation costs is therefore made.

The costs of the compensation measures are based on the ones estimated in the Brouwersdam project. For the Brouwersdam the compensation is for the loss of above water nature, but it is assumed that these costs are in the same order of magnitude as the loss of intertidal character. The introduced tide at the Brouwersdam is 40 cm in a 110 km^2 basin, and the corresponding compensation for nature value is 23 million euros. This leads to the following relation: a 440 million m^3 reduction of the tidal prism leads to 23 million euros in mitigation costs. Translating this to cost per cubic meter reduction leads to:

$$C_{\text{nature}} = |R_1 - R_2| S 50000 \tag{A.2}$$

With C_{nature} the compensation cost for nature in euros
With R_1 the initial tidal stroke at the lake in meters
With R_2 the altered tidal stroke at the lake in meters
With S the surface area of the lake in m^2

This will be used as a first estimation of the nature mitigation costs at a location. It is important to note that the Grevelingenlake is a IUCN category IV protected area, which is reflected in the mitigation cost. For a category I protected area the cost could be much higher, or for a unprotected area the cost will be lower. As mentioned in section A.1 the selected locations fall under category IV to VI, which are assumed to have mitigation costs of the same order of magnitude. Unprotected areas will have less mitigation costs, since there is no information on policies for these areas an assumption is made that the compensation costs will be 50% of the compensation costs for a protected area.

A.4. Infrastructure

Changing the tidal signal at the lake will have consequences for infrastructure along the shore of the created lake. E.g mooring facilities will have to be altered, if a port is present at the lake this will also need to be modified to facilitate mooring with the new tide. The cost related to this are the infrastructure mitigation cost. The perimeter of the lake and the vertical distance over which the tide is altered are the most important factors influencing this. The compensation cost for infrastructure estimated for the Grevelingenlake are used to derive a general relationship for the compensation cost for infrastructure at any location. The perimeter around the Grevelingenlake is about 60 km, and the change in tidal stroke is 0.4 meter which resulted in 14 million euros compensation costs. Translating this into cost per square meter equation A.3 is found.

$$C_{\text{infra}} = |R_1 - R_2| p600 \tag{A.3}$$

With C_{infra} the compensation cost for infrastructure in euros
With p the perimeter of the lake in meters

A.5. Agriculture

The water levels have an effect on agriculture around the lake. Changing the mean water level can influence the salinity of the soil and groundwater. A raised mean water level means more salinity in the groundwater and soil which can have negative consequences for agriculture. No compensation cost are linked to this. This mean, maximum and minimum water level will be discussed in relation to the minimum tidal stroke.

APPENDIX
B

LOCATION ANALYSIS SATELLITE IMAGERY

The locations that will be explored are as follows:

- Locations with extreme values
  - 4 largest basin areas
  - 4 smallest basin areas
  - 4 largest population sizes
  - 4 smallest population sizes
- Top 10 locations identified as most promising by Deltares

It is expected that with this selection will give a good overview of the most important features of the dataset.

B.1. Largest basin areas

In figures B.1a, B.1b, B.1c and B.1d the four largest basin areas in the dataset are shown with their respective surface areas in the captions. In the figures a line is included for scale. It becomes clear that the points identified in the database do not match the indicated surface area. It should be noted that a surface area of 2.56 million km^2 is extremely large, the entire red sea is approximately 0.44 km^2. Furthermore the surface area of the whole of Egypt is approximately 1 million km^2.

Figure B.1a shows the river mouth of the Nile. While the total area of the Nile might be substantial, it will not come near 2.56 million km^2. It is not realistic that a barrage can be implemented, and considering the tidal window that the total surface area is utilised. The extremely large surface area seems to be wrong. Figure B.1b shows the second largest basin in Iraq, here again the basin area seems to be incorrect. When the river mouth is meant the same arguments can be made as for the one in Egypt, and also if a neighbouring tidal basin is meant this is not nearly the correct surface area. If the entire Persian Gulf is considered then the surface area is still smaller (0.25 million km^2 instead of 0.87 million km^2).

Figure B.1c also shows a river mouth, the surface area of any potential basin here also does not match the 0.5 km^2. If entire Gulf of Mexico is meant as the tidal basin, this would be too large (1.6 km^2). Figure B.1d also does not match the surface area in the database.

B.2. Smallest basin areas

In figures B.2a, B.2b, B.2c and B.2d the four smallest basin areas in the database are shown, in the caption their area according to the database is shown. In all figures a line is included to get a feel for the size of the body of water. In figure B.2d it can be seen that the area that is in the database is incorrect, the area of the basin is approximately 300 km^2 instead of 51 km^2. The other small basins seem to be correct.

(a) Location Egypt (2.56 million km^2)

(b) Location Iraq (0.87 million km^2)

(c) Location Mexico (0.50 million km^2)

(d) Location Brazil (0.34 million km^2)

Figure B.1: Four largest basin areas in dataset

(a) Smallest basin in database: Madagascar (50 km^2)

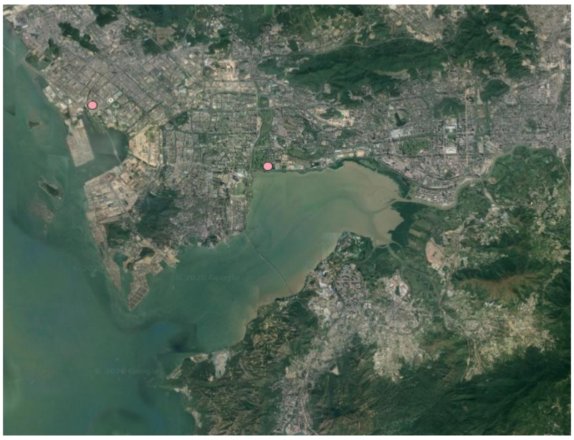
(b) Second smallest basin area in database: Indonesia (50.6 km^2)

(c) Third smallest basin area in database: Russia (50.9 km^2)

(d) Fourth smallest basin area in database: Mexico (50.99 km^2)

Figure B.2: Four smallest basin areas in dataset

(a) Largest population location in China



(b) Two neighbouring sites with very large population sizes in China

(c) Location with very large population in India

(d) Zoomed in picture location China with port

Figure B.3: Four largest basin areas in dataset

B.3. locations with the largest populations

In figures B.3a, B.3b and B.3c the locations with the largest population sizes are shown. Upon closer inspection of figure B.3b shows that one of the locations might not be suitable because of the presence of a large port. This is shown in figure B.3d. In all images a large urban area can be seen, this is an indication that the population data is accurate. The large city in India that has two is in fact Mumbai, one of the most densely populated cities on earth (approximately 28 thousand people per square kilometer). With the 50 km radius of interest this encompasses the whole of Mumbai. There are approximately 18.4 million people living in Mumbai, while the population count according to the dataset is only 12.2 million people. The population count thus seems to have an underestimation of the true number. The populations are however in the same order of magnitude.

B.4. Locations with smallest population sizes

Four locations are looked at that all have a population of zero, they are shown in figure B.4. All locations are very northern, and ice could become problematic here. Figures B.5, B.6, B.7 and B.8 show satellite images of these locations. It can be seen that indeed there is no sign of urban areas which makes it plausible that the zero population is correct.

Figure B.9 shows a zoomed in image of the location in Nova Scotia. Here it can be seen that ice is present, which could make this location not suitable for tidal energy generation. Furthermore it is observed that there doesn't seem to be any large infrastructure present. This could make it difficult to construct a tidal barrage at this location.

Figure B.4: Locations with 0 population

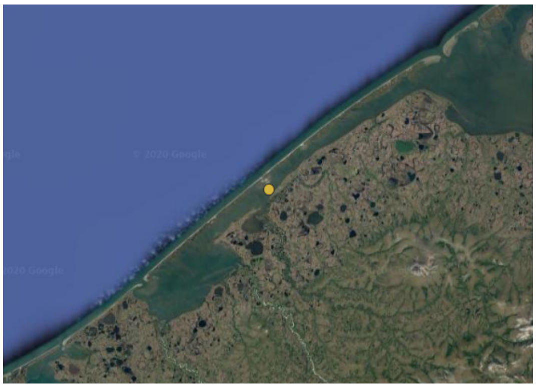


Figure B.5: Population zero: Alaska 1 (US)

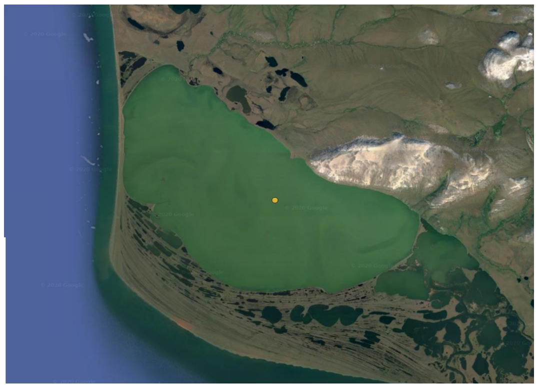


Figure B.6: Population zero: Alaska 2 (US)

Figure B.7: Population zero: Nova Scotia (US)

Figure B.8: Population zero: Canada

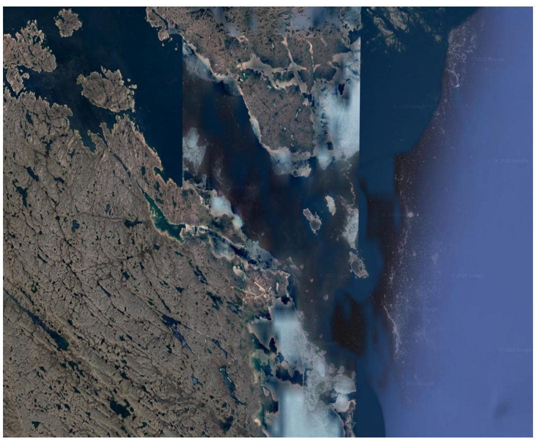


Figure B.9: Zoomed in image river mouth Nova Scotia

Figure B.10: Overview top 10 locations as identified by Deltares

B.5. 10 most promising sites Deltares

Using the filtering criteria and weighing factors as indicated by the Deltares the top 10 scoring locations have been selected. Their locations can be seen in figure B.10. In India and China there are two locations that are relatively close by which is why they show up as the same dot in this zoomed out overview. Five of these overlap with sites that already have been explored: the four sites with the largest population sizes (3 in China, 1 in India) and the location in Egypt with the large basin area. The other five sites will be discussed in this section.

Figures B.11, B.12, B.13, B.14 and B.15 show the other locations in the top 10 as identified by Deltares. In South Korea (figure B.15) there is a bridge already present. The location in the Netherlands (figure B.14) already has 2 dams present: The ' Afsluitdijk' and the ' Houtribdijk' . It is interesting to note that the most promising location in the Netherlands according to this analysis is not the Brouwersdam.

Figure B.11: Top 10 Location as identified by Deltares in China

Figure B.12: Top 10 Location as identified by Deltares in India

Figure B.13: Top 10 Location as identified by Deltares in France

Figure B.14: Top 10 Location as identified by Deltares in Netherlands

Figure B.15: Top 10 Location as identified by Deltares in South Korea

B.6. Preliminary selections

B.6.1. Highest energy potential

Area and Mean Tidal Range

When focusing solely on energy generation then the locations with high tidal amplitude and large basin area best suited. With the thresholds set at a basin area larger than 2000 km^2 and tidal range higher than 4.5 meters this leaves 9 locations with a high energy potential, they are shown in figure B.16 and their properties given.

Table B.1: Top 9 locations with highest energy potential data from dataset

Country	Basin Area [km^2]	Flood Exposure [m]	Mean Tidal Range [m]	Population
United States of America	2792		7.6	275768
United Kingdom	8081		5.7	191241
Mozambique	28580		5.4	294726
Italy	14741		4.6	246674
Italy	4859		4.6	513403
Italy	3347		4.6	449565
Brazil	16166		4.5	668232
Brazil	3468		4.5	1824092
Brazil	3313		4.5	646790

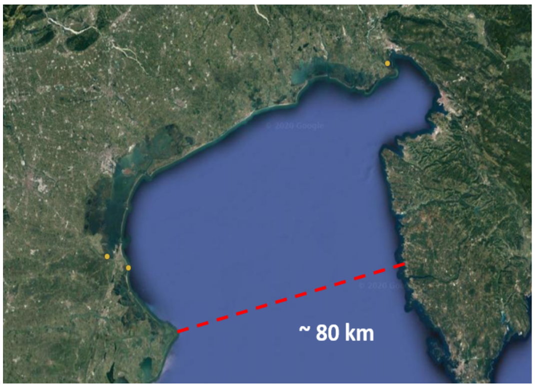
Figure B.16: 9 Locations with highest energy potential

All locations have been investigated through satellite imagery. The locations in the UK, Brazil and Mozambique all have a surface area that is much smaller than indicated in the dataset (approximately factor 10). Furthermore one of the locations in Brazil is a port (figure B.17) and is therefore not suitable. In Italy the only way the surface area in the dataset can be matched (3000 - 4000 km^2) is if a very large dam would be built as indicated in figure B.18. The span would have to be approximately 80 km. The average water depth at the cross section is approximately 35 m. It is not realistic that such a structure would be built. Furthermore it is shown that the three potential locations then amount to one tidal power station. The last location is in Anchorage Alaska, the basin area is approximately right, a dam would have to be approximately 30 km long to close the basin. (figure B.19) There is a relatively deep section in the middle of the basin (approx 40 m) while the rest of the span is approximately 20 m. Because of the climate at this location there could be a problems with ice.

All in all this analysis of the most promising sites based on energy generation shows that this is not the best metric for the selection of potential locations. Upon closer investigations none of the sites live up to the energy potential that was calculated. Locations where the huge basin area could be realised this would mean that an enormous dam would have to be constructed also over a large depth, which is unrealistic. Some of the basin areas prove to be incorrect and thus while the tidal range is still promising the energy yield of the location will be lower in reality. Two sites were furthermore eliminated because of the presence of ice and because of the presence of a port.

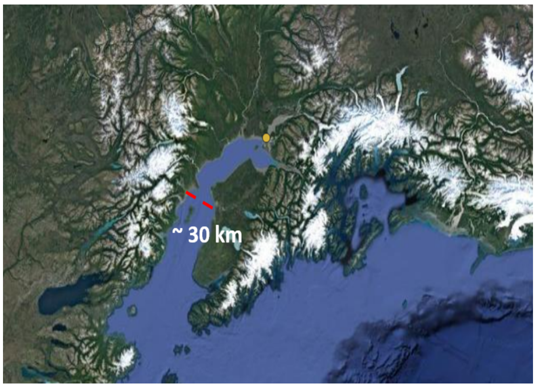
(a)

Figure B.17: Location Brazil with port



(a)

Figure B.18: Dam needed Italy for basin area 4000 km^2



(a)

Figure B.19: Dam needed in Alaska (US) for basin area 2800 km^2

Mean tidal range

It can be considered to use the energy potential metric but only based on mean tidal range, in this case the top 10 locations are shown in figure B.20 and their properties are given in table B.2

The location in the United States (Anchorage) is the same as in the previous selection and is not suitable because of its cold climate. In Argentina there are two suitable locations. The most northern one has a true basin area of approximately 160 km^2, a tidal barrage would have a length of approximately 900 m, the largest depth in that span in around 10 meters. (Figure B.21) This is a feasible location with high energy potential. The more southern location in Argentina has a true surface area of around 100 km^2, a barrage of about 5 km would be needed and the largest depth over this span is approximately 16 m. (Figure B.22). In the UK 3 suitable locations are found. Two are close together in the south east. If one dam is placed then this would have to be 17 km long. The tidal lake that is created by this dam has a surface area of approximately 460 km^2. (figure B.23) The largest depth over the span of the barrage is 30 m. This would be a very large project, it is however unlikely that such a large dam would be placed over such a large water depth. The other location in the UK in the north west has a surface area of 130 km^2, a barrage would be approximately 10 km. (Figure B.24 There is no data on the local water depth but judging by the surrounding area a maximum depth of approximately 12 m can be expected. (figureB.25) The location in France is close to La Rance tidal power station. There is no clear basin to be closed. The average depth in this area is about 7 m. (Figure B.13) Location in Thailand has a basin area of approximately 30 km^2, a barrage would span 5 km and the average depth is 1 m. (Figure B.27). The location in South Korea is the same as in the top 10 identified by Deltares (figure B.15), it closed of a area of 30 km^2. The span of the dam is approximately 4 km. The dam would have to be transformed to be suitable for a tidal power station. The last location is in Vietnam (figure B.28), the basin area is approximately 200 km^2. The span of a barrage would be 7 km, the deepest point in the cross section is 19 m.

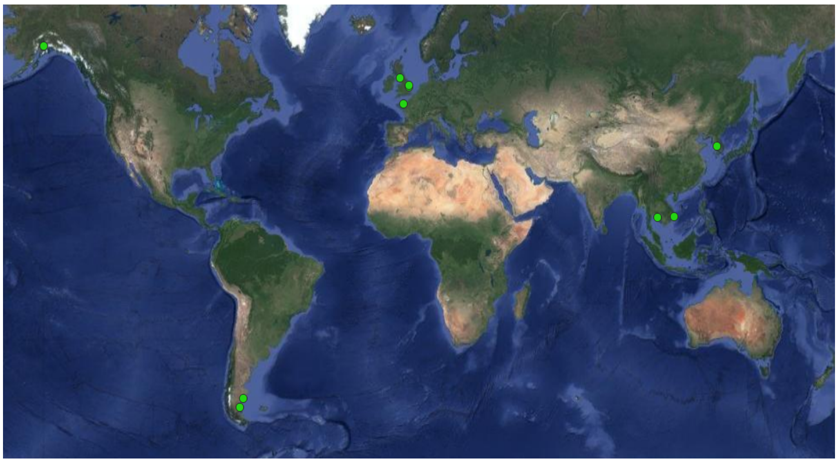


Figure B.20: Top 10 locations with highest mean tidal range

Table B.2: Top 10 locations based on mean tidal range data from dataset

country	Basin area [km^2]	Flood exposure [m]	MTR [m]	Population
France	205	4.9	9.3	202639
Argentina	67.49	6.3	8.7	8536
United States of America	2792	2.3	7.6	275768
South Korea	102	5.1	6.9	1605528
Argentina	73.17	5.3	6.9	882
United Kingdom	118	0.9	6.1	162621
United Kingdom	8081	2.1	5.7	191241
Vietnam	139	1.3	5.7	548288
Thailand	257	1.6	5.7	143798
United Kingdom	330	3.6	5.6	116173

Table B.3 gives an overview of the collected data after inspection of the locations. Sites that were not suitable have been excluded.

Table B.3: Collected data suitable locations based on inspection

Country	Basin Area [km2]	Length Barrage [km]	Deepest point in cross section [m]
Argentina North	160	0.9	10
Argentina South	100	5	16
UK North West	130	10	12
Thailand	30	5	1
South Korea	30	4	N.A
Vietnam	200	7	19

Figure B.21: Northern location Argentina

Figure B.22: Southern location Argentina

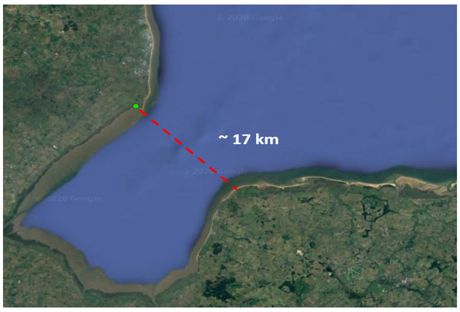


Figure B.23: Location South East UK

Figure B.24: Location North West UK

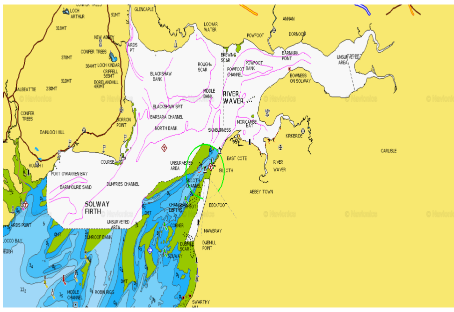


Figure B.25: Depth Chart location North West UK (Garmin (2018))

Figure B.26: Location in France

Figure B.27: Location in Thailand

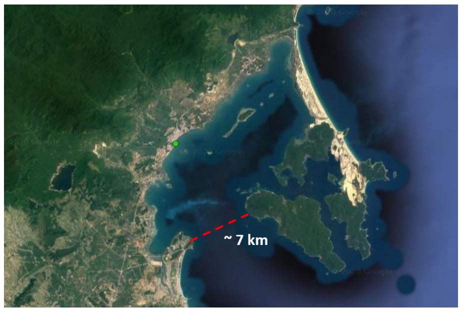


Figure B.28: Location in Vietnam

B.6.2. Selected locations based on flood risk

When the solution is solely focused on flood prevention then the locations with the highest flood risk are chosen. These are shown in figure B.29 and their properties are given in table B.4.

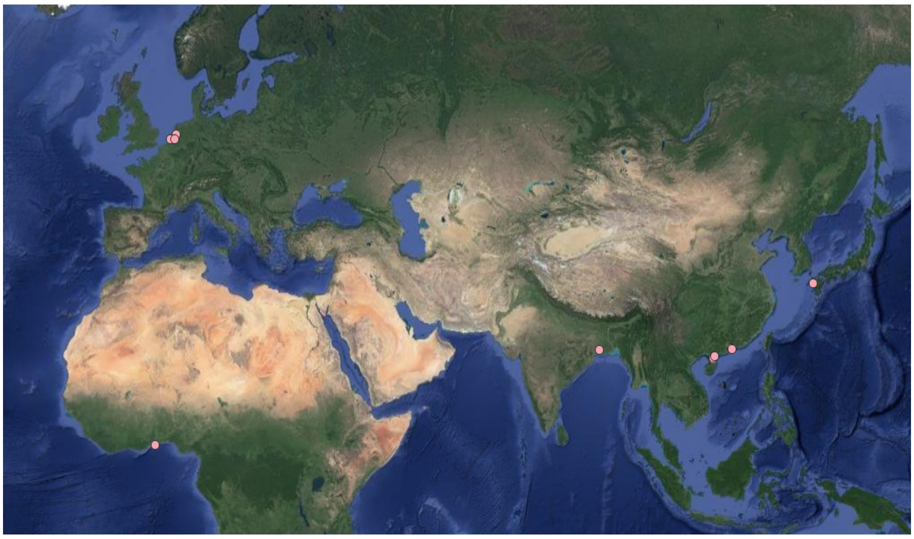


Figure B.29: Top 12 locations with highest flood risk data from dataset

Table B.4: Top 12 locations with highest flood risk

Country	Basin Area [km^2]	Flood Exposure [m]	Mean Tidal Range [m]	Population
Netherlands	110.36	6.4	3.9	592462
Japan	52	6.6	3.8	687101
China	87	6.2	3.6	887856
China	88	5.6	3.6	12975102
China	99	5.6	3.6	12877444
China	173	6.2	3.6	2271870
China	295	6.2	3.6	1954967
China	400	5.6	3.6	2464749
Netherlands	1664	6.0	3.4	2108428
Netherlands	72.38	7.2	2.7	1530444
Togo	248.02	5.8	1.3	1395095
India	76.4	7.3	1.0	8048481

It can be seen that almost all locations are in either the Netherlands or China. The location in India has a relatively low tidal range.

The locations found in the Netherlands include the Brouwersdam. All three basins are already closed by a dam and are thus upon first sight well suited for this technology. (Figure B.30). The location in Togo is actually already closed off from the sea by a natural barrier and only seasonally there is an inflow of sea water at high tide. Since this is a natural lake introducing a tide at this location will significantly alter the ecosystem. (Figure B.31). There are six locations in China which are numbered in figures B.32 and B.33. Location 1 needs a barrage of around 3 km and has a basin area of 60 km^2. The deepest point in the cross section of the barrage is approximately 15 m. Location 2 needs a barrage of approximately 3 km, the depth around the closure is on average 3 m. The basin area of the lake is around 20 km^2. Location 3 is the same as B.11, with a dam already present with a span of around 500 m and a basin area of 5 km^2. Location 4 needs a barrage of approximately 1 km, and would close of a basin with 10 km^2 area, average depth is 3 m. Location 5 and 6 are the same as B.3b. Location 5 is not suitable because this is a large port. Location 6 would need a barrage of around 5 km, which closes of an area of 60 km^2, average depth of average depth in the cross section is 15 m. The location in India has a basin area of around 60 km^2, the tidal barrage would span around 4 km, with the largest depth in the cross section being 15 m. The final location is in Japan, with a large basin area of approximately 460 km^2. A span of around 2 km needs to be closed with a barrage, but because there are some islands the construction would not be as long. No information is available on the average depth.

In table B.5 an overview is given of the data gathered with inspection. Locations that were not suitable have been excluded.

Table B.5: Collected data suitable locations based on inspection

Country	**Basin Area [km^2]**	**Length Barrage [km]**	**Deepest point in cross section [m]**
Netherlands 1	110	5	N.A
Netherlands 2	110	2.5	N.A
Netherlands 3	640	30	N.A
China 1	60	3	15
China 2	20	3	15
China 3	5	0.5	N.A
China 4	10	1	3
China 6	60	5	15
India	60	4	15
Japan	460	2	-

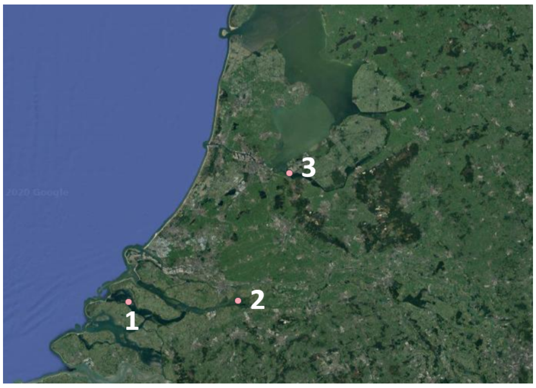


Figure B.30: Three locations in the Netherlands with high flood risk

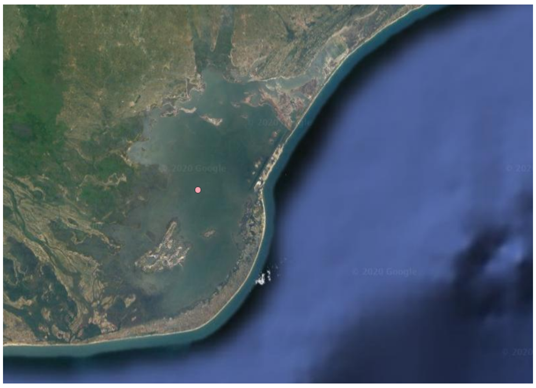


Figure B.31: Lake in Togo with high flood risk

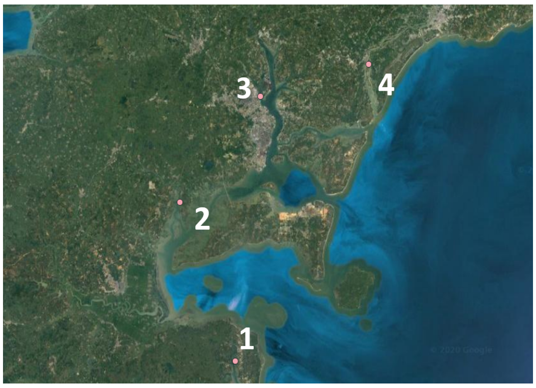


Figure B.32: Four locations in China with high flood risk

Figure B.33: Two more locations in China with high flood risk

Figure B.34: Location with high flood risk in Japan

Figure B.35: Location in India with high flood risk

APPENDIX
C

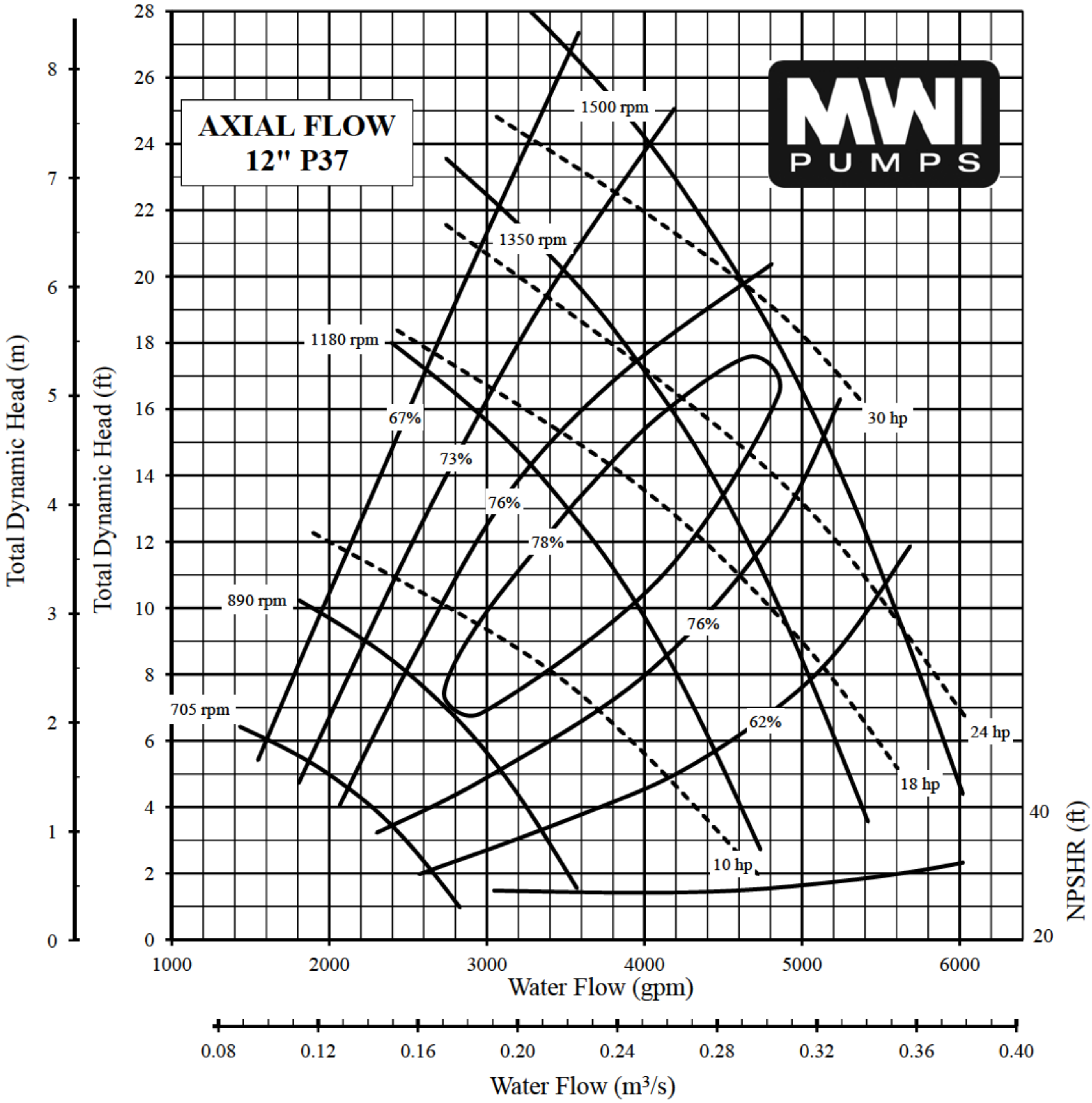
PUMP PROPERTIES

There are two curves that define the behaviour of a pump, these are the performance curve and the system curve. Both will be elaborated on in this section, and lastly it is explained how these can be combined to define the operation range of the pump.

C.1. Performance curve

A pump has specific properties that can be described by a performance curve. This curve describes the maximum discharge that can be achieved by the pump for a certain head difference. This curve limits the maximum amount of work that can be done at each moment by the pump. The performance curve is normally supplied by the manufacturer.

Since there are no reference projects of large pumps that would be suited for the application of a tidal power station with pump capacity an analysis is done of smaller pumps to see how these can be scaled up. First the possibility to do this with the affinity laws is researched. As this proves to be difficult to apply for the found performance curves, other methods are researched. The performance curves and how they change for different pump sizes is analysed. The observations made are used to find a performance curve based on maximum capacity. The curves of the analysed pumps are shown in the next pages. Four pumps are selected, with each pump being twice the size of the previous.



PUMP BOWL PERFORMANCE CURVE

TYPE: AXIAL FLOW	PROPELLER DIA: 12"
MODEL NO: NC312P37	SPEED: As Noted
INTAKE DIA: 18"	DISCHARGE COLUMN DIA: 12"
CURVE NO.: VS312P37A	Ns: 11300 CODE: 0.50

SINGLE STAGE PERFORMANCE
FOR TWO STAGES MULTIPLY HEAD AND HORSEPOWER BY 2 0 AND EFFICIENCY BY 1 0
PERFORMANCE IS BASED ON PUMPING CLEAR, NON-AERATED WATER, WITH A SPECIFIC GRAVITY OF 1 0, TEMPERATURE 85 °F OR LESS AND AT SEA LEVEL PUMP PERFORMANCE MAY BE AFFECTED BY HIGHER TEMPERATURES, SPECIFIC GRAVITY, ALTITUDES AND SUMP CONDITIONS

IT IS HEREBY CERTIFIED THAT THIS CURVE REPRESENTS THE TRUE PERFORMANCE CHARACTERISTICS OF THE MWI PUMP MODEL SHOWN AND WAS OBTAINED BY SCALE MODEL TEST AND CALCULATIONS IN ACCORDANCE WITH STANDARDS OF THE HYDRAULIC INSTITUTE.

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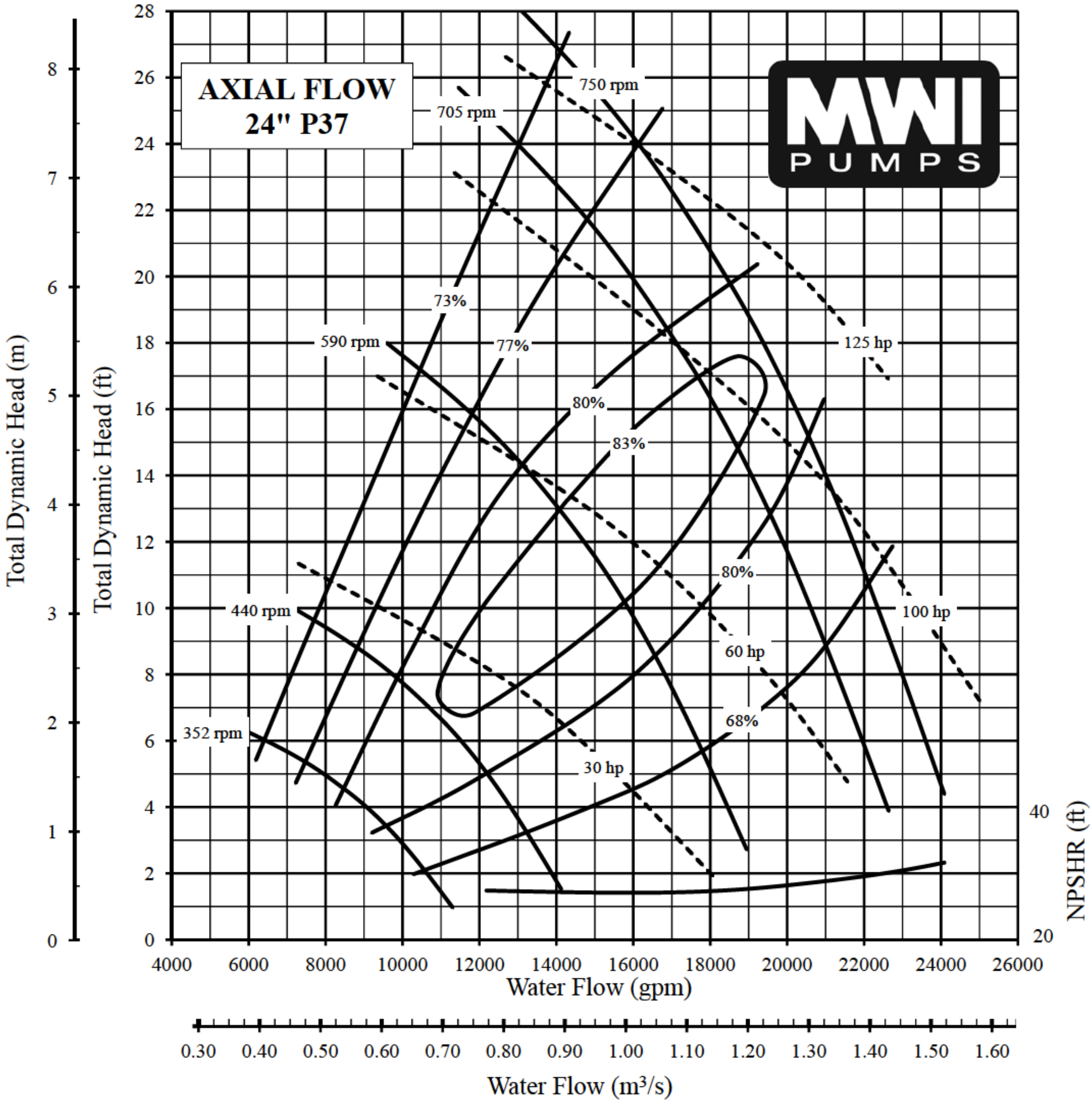

PUMP BOWL PERFORMANCE CURVE	
TYPE: AXIAL FLOW	**PROPELLER DIA: 24"**
MODEL NO: NC324P37	**SPEED: As Noted**
INTAKE DIA: 36"	**DISCHARGE COLUMN DIA: 24"**
CURVE NO.: VS324P37A	**Ns: 11300 CODE: 0.50**

SINGLE STAGE PERFORMANCE
FOR TWO STAGES MULTIPLY HEAD AND HORSEPOWER BY 2 0 AND EFFICIENCY BY 1 0
PERFORMANCE IS BASED ON PUMPING CLEAR, NON-AERATED WATER, WITH A SPECIFIC GRAVITY OF 1 0, TEMPERATURE 85 °F OR LESS AND AT SEA LEVEL PUMP PERFORMANCE MAY BE AFFECTED BY HIGHER TEMPERATURES, SPECIFIC GRAVITY, ALTITUDES AND SUMP CONDITIONS

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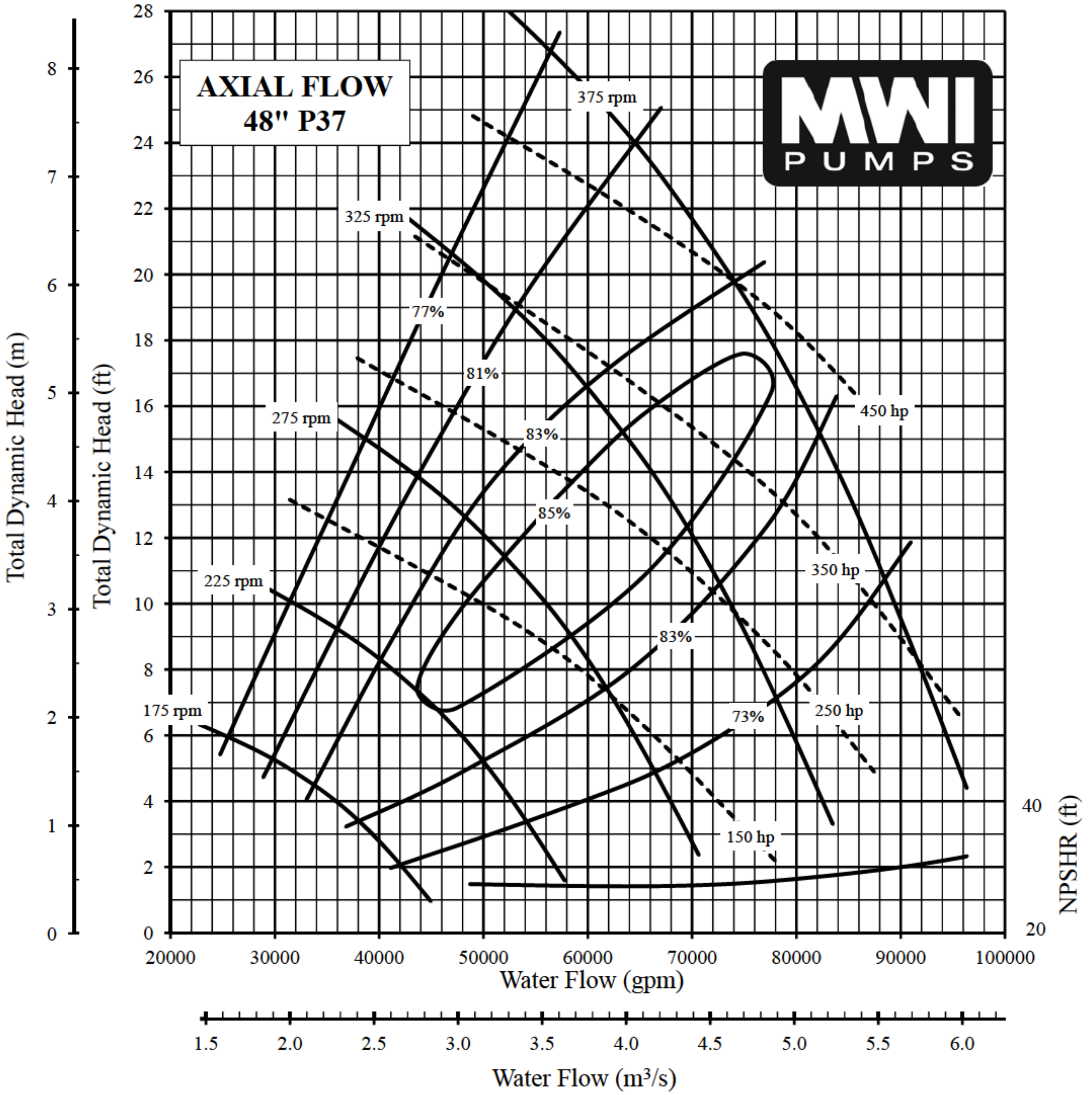

PUMP BOWL PERFORMANCE CURVE	
TYPE: AXIAL FLOW	**PROPELLER DIA: 48"**
MODEL NO: NC348P37	**SPEED: As Noted**
INTAKE DIA: 72"	**DISCHARGE COLUMN DIA: 48"**
CURVE NO.: VS348P37A	**Ns: 11300 CODE: 0.50**

SINGLE STAGE PERFORMANCE
FOR TWO STAGES MULTIPLY HEAD AND HORSEPOWER BY 2 0 AND EFFICIENCY BY 1 0
PERFORMANCE IS BASED ON PUMPING CLEAR, NON-AERATED WATER, WITH A SPECIFIC GRAVITY OF 1 0, TEMPERATURE 85 °F OR LESS AND AT SEA LEVEL PUMP PERFORMANCE MAY BE AFFECTED BY HIGHER TEMPERATURES, SPECIFIC GRAVITY, ALTITUDES AND SUMP CONDITIONS

IT IS HEREBY CERTIFIED THAT THIS CURVE REPRESENTS THE TRUE PERFORMANCE CHARACTERISTICS OF THE MWI PUMP MODEL SHOWN AND WAS OBTAINED BY SCALE MODEL TEST AND CALCULATIONS IN ACCORDANCE WITH STANDARDS OF THE HYDRAULIC INSTITUTE.

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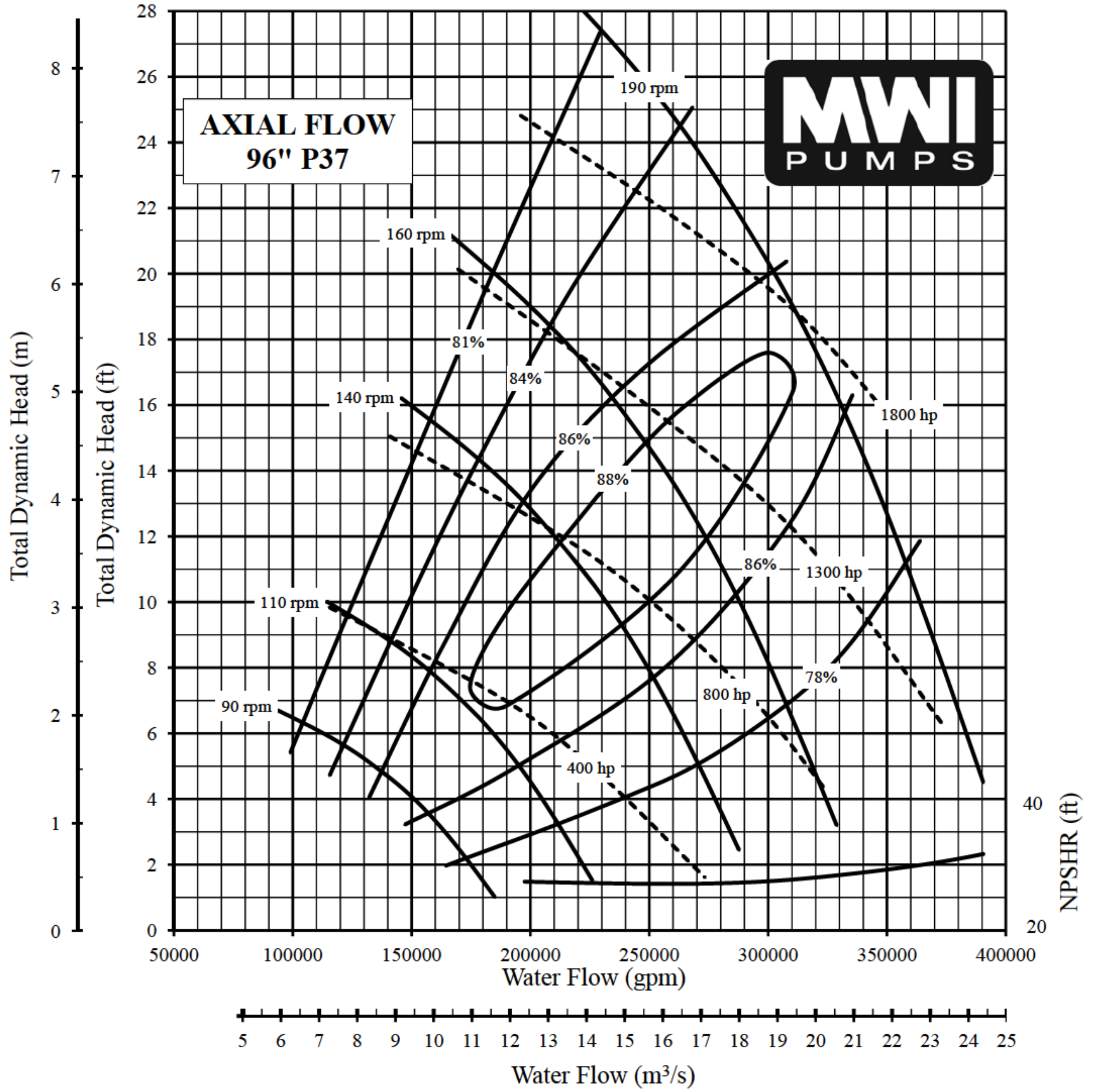

PUMP BOWL PERFORMANCE CURVE	
TYPE: AXIAL FLOW	**PROPELLER DIA: 96"**
MODEL NO: NC396P37	**SPEED: As Noted**
INTAKE DIA: 144"	**DISCHARGE COLUMN DIA: 96"**
CURVE NO.: VS396P37A	**Ns: 11300 CODE: 0.50**
SINGLE STAGE PERFORMANCE FOR TWO STAGES MULTIPLY HEAD AND HORSEPOWER BY 2 0 AND EFFICIENCY BY 1 0 PERFORMANCE IS BASED ON PUMPING CLEAR, NON-AERATED WATER, WITH A SPECIFIC GRAVITY OF 1 0, TEMPERATURE 85 °F OR LESS AND AT SEA LEVEL PUMP PERFORMANCE MAY BE AFFECTED BY HIGHER TEMPERATURES, SPECIFIC GRAVITY, ALTITUDES AND SUMP CONDITIONS	

IT IS HEREBY CERTIFIED THAT THIS CURVE REPRESENTS THE TRUE PERFORMANCE CHARACTERISTICS OF THE MWI PUMP MODEL SHOWN AND WAS OBTAINED BY SCALE MODEL TEST AND CALCULATIONS IN ACCORDANCE WITH STANDARDS OF THE HYDRAULIC INSTITUTE.

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C.1.1. Affinity laws

Constant Diameter

With the affinity laws the performance curve can be scaled to a different shaft speed for the same diameter pump. The discharge is proportional to the shaft speed (equation C.1) while the power is proportional to the cube of the shaft speed (equation C.2).

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \tag{C.1}$$

With N the shaft speed in rotations per minute

$$\frac{P_1}{P_2} = \frac{N_1}{N_2}^3 \tag{C.2}$$

The application of the affinity laws in the performance curves is checked. The results are given in table C.1. The discharge and power of the lower shaft speed are predicted by using the ratio of the shaft speeds and the value from the larger shaft speed. It can be seen that both the discharge and power predicted by the affinity laws are not the same as read from the graph. Direct application of the affinity laws on these graphs is therefore difficult.

Table C.1: Check affinity laws constant diameter

Diameter [inch]	**Shaft speed [rpm]**	**H [m]**	**Q [m^3/s] graph**	**Q [m^3/s] predicted**	**eta [%]**	**P [MW] graph**	**P [MW] predicted**
96 (2.44 m)	190	6.3	19	-	0.86	1.37	-
96 (2.44 m)	160	6.3	11.5	16	0.81	0.88	0.82

C.1.2. Constant shaft speed

According to the affinity laws, when the shaft speed (rpm) is held constant the discharge is directly proportional to the diameter (equation C.3). The power of the pump is proportional to the cube of the diameter (equation C.4).

$$\frac{Q_1}{Q_2} = \frac{D_1}{D_2} \tag{C.3}$$

$$\frac{P_1}{P_2} = \frac{D_1}{D_2}^3 \tag{C.4}$$

The pumps 96 inch and 72 inch both have a curve with 190 rpm, hence shaft speed held constant. At around 5 meters head difference the discharge for both pump diameters are read, the same is done for two smaller pumps of 30 and 24 inch with a shaft speed of 352 rpm. This can be seen in table C.1 .When the discharge of the largest pump is used to predict the discharge of the smaller pump the predicted discharge is larger than the actual discharge read in the graph. This shows that the affinity laws can't be used to predict the discharge of the curves by this manufacturer, or that the these curves can't be used if the affinity laws are used.

C.1.3. Analysis performance curves

First the properties of a performance curve are analysed for a single sized turbine, for this the 96 inch (2.44 m) pump will be used. Next it is analysed how these properties change for different pump sizes.

Shape curve and maximum capacity

The shape of the performance curve is similar for different shaft speeds and different pump sizes. The curve is a relatively steep concave line. In table C.2 some of the points of the upper curve of the largest pump are shown. For each point the capacity was calculated using equation C.5. It can be seen that the capacity of the pump is not constant over the performance curve. For high head difference and lower discharge the capacity is larger than for low head difference and high discharge. The efficiency is also not constant over the curve, this is in part related to the power not being constant. The efficiency increases as the discharge increases and the head difference decreases, up to a maximum around a discharge of 20 m3/s, after which the efficiency decreases again.

$$P_{\text{pump}} = \frac{\rho g Q H}{\eta_{\text{pump}}} \tag{C.5}$$

Table C.2: Points on performance curve diameter 96 inches (2.44 m) 190 rpm

H [m]	Q [m3/s]	P [MW]	eta [%]
8.2	14.5	1.44	0.81
7.5	16.5	1.45	0.84
6.1	19	1.32	0.86
4.8	20.5	1.12	0.86
3	22.5	0.85	0.78

Pump efficiency with increasing diameter

In every pump performance curve also efficiency lines are visible. For a certain head difference and discharge the pump efficiency varies. What can be observed is that from small to large the maximum efficiencies of the pumps are 78%, 83%, 85%, 88% respectively. Larger pumps have a higher efficiency. There is an upper limit to this as a pump with a 100% efficiency is not realistic.

Rotations per minute

There are multiple lines representing different rotational speed of the pumps. For a simple pump there is a fixed number of rotations per minute, the pump is either on or off. It can be seen that the maximum number of rotations per minute decreases as the pump size increases.

C.1.4. Performance curve based on capacity

Since the affinity laws cannot be used to produce a performance curve of a larger pump, a different method is chosen. A simplified relation will be used but as many of the elements resulting from the analysis are included. The amount of water that the pump can move is limited by its capacity. The capacity of the pump is related to the turbine capacity, as it uses the same mechanical driving mechanisms. Using the turbine capacity as the pump capacity will be the starting point. The discharge at each head difference can then be calculated with the expression:

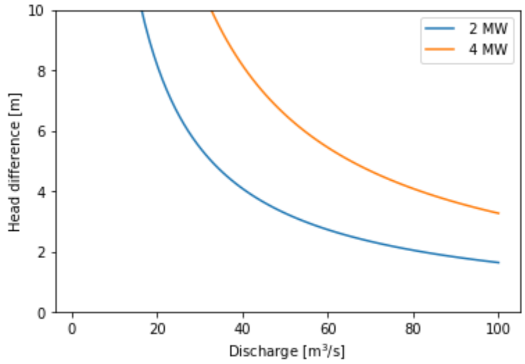
$$Q = \frac{MW\eta}{\rho g H} \tag{C.6}$$

The relation to the size of the pump is now implicit through the capacity. The performance curves for pumps with 2 MW and 4 MW capacity are shown in figure C.1a.

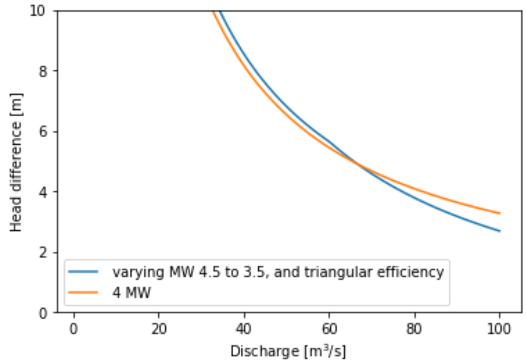
A few features are missing from this curve. This curve is for a constant capacity and a constant efficiency, the shaft speed is undefined. It is also clear that the shape is different from the performance curves used as examples. From the analysis it became clear that the capacity decreases at higher discharges, and the efficiency varies for the discharge with a maximum at an intermediate discharge. Larger pumps furthermore have a larger efficiency. The curve is therefore altered with the following two relationships: A triangular distribution of the efficiency is assumed (0.75 at 0, 0.85 at 0.6 Q and 0.75 at Q), and furthermore the power linearly decreases with increasing discharge. The result is shown in figure C.1b, for comparison also the curve with constant capacity 4 MW and constant efficiency 80% is included. It can be seen that the difference between the lines is minimal. For this reason it is chosen to include only average values in the model. The accuracy of the model is not increased by including these relationships, based on assumptions. The shape of the curve is concave and provides a reliable lower bound of the pumping capacity.

C.2. System curve

Water is pumped from a location with a lower head to a location with a higher head. The head difference that needs to be overcome to do so equals the height difference (static head) plus compensation for energy losses along the path due to friction and viscous effects (dynamic head).



(a) Performance curve based on capacity



(b) Performance curve based on capacity with effect for power and efficiency included

The static head is the difference in water level at sea and at the lake. (equation C.7). Because the tide is constantly changing the static head is not constant but will also fluctuate. It is assumed that the rate of change of the water level is low enough that at each point in the calculation the water levels can be considered stationary.

$$H = h_{\text{sea}} - z_{\text{lake}} \tag{C.7}$$

The dynamic head is caused by losses over the length of the pipes due to friction and losses at transitions along the flowpath. The dynamic head is added to the static head. The dynamic head can be approximated with Darcy Weisbach (equation C.8).

$$H_d = \frac{Kv^2}{2g} \tag{C.8}$$

With H_d the dynamic head [m]
With K the loss coefficient [-]
With v the velocity in the pipe [m/s]
With g the gravitational constant [m/s^2]

Substituting the velocity with discharge, Q, results in the following equation

$$H_d = \frac{K\frac{4Q}{\pi D^2}^2}{2g} \tag{C.9}$$

With D the pipe diameter [m]

The K factor is the loss factor and consists of two components, friction loss and transitions loss (called minor losses). (see equation C.10)

$$K = K_{\text{friction}} + K_{\text{minor}} \tag{C.10}$$

For friction loss equation C.11 can be used.

$$K = \frac{fL}{D} \tag{C.11}$$

With f the friction coefficient [-]
With L the length of the pipe [m]

With f the friction coefficient, L the length of the pipe in meters and D the diameter in meters. The friction coefficient can be calculated using a modified version of the Colebrook-White equation:

$$f = \frac{0.25}{[log(\frac{k}{3.7D} + \frac{5.74}{Re^{0.9}}]^2} \tag{C.12}$$

With k the roughness factor [m]
With Re the Reynolds number

Values for k for different materials are given in table C.3.

Table C.3: Standard values for roughness factor k

Piping material	**Roughness k [mm]**
Cast iron	0.26
Commercial steel and wrought iron	0.045
Concrete	0.3 - 3.0
Drawn tubing	0.0015
Galvanized iron	0.15
Plasic and glass	0 (smooth)
Riveted steel	0.9 - 9.0

The Reynolds number can be calculated with the following equation:

$$Re = \frac{vD}{\nu} \tag{C.13}$$

With ν the kinematic viscosity [m^2/s]

The kinematic viscosity for salt water at 10 degrees is 1.36 x 10 $^{-6}$ m^2/s.
For transition losses standard tables exist with standard values for transitions. Minor losses are due to viscous effect when fluid flows through a transitional component. Table C.4 shows the K factors for the transitions relevant in this situation.

Table C.4: Minor losses from Elger et al. (2014)

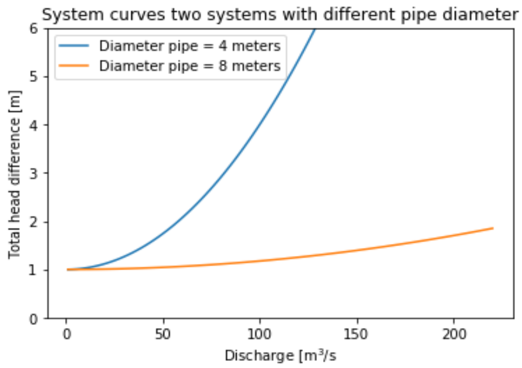
Description	**K**
Pipe Entrance	0.12
Expansion (outlet)	0.87

By adding the two K factors the dynamic head in equation C.8 can be calculated.

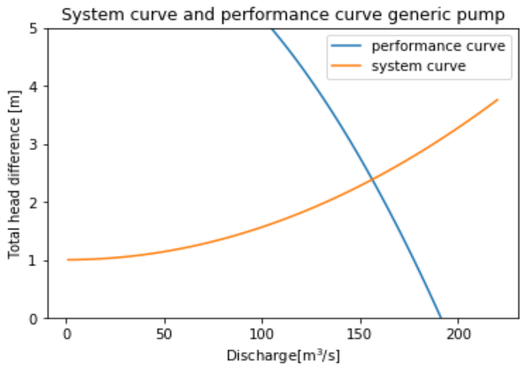
By repeating this calculation for a range of discharges a system curve can be developed. This curve is unique to the properties of the system under evaluation and shows the dynamic head at a discharge rate. In figure C.1c it can be seen that the diameter of the pipe is of large influence in the systemcurve. When the diameter is large, less friction is experienced and the flow becomes more like open channel flow. In this case the dynamic head to overcome is much less than for a pipe with a smaller diameter.

C.2.1. Operational range

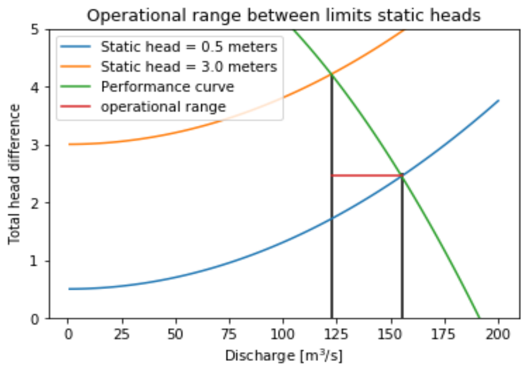
The intersection of the system curve with the performance curve of the pump gives the discharge of the pump. This is illustrated in figure C.1e
At a tidal power plant the tide is continuously changing and with it the head difference also changes. This means that the static head changes, but the system curve remains the same. This gives a range of discharges under which the pump can operate. This is illustrated in figure C.1e. When 4 meters is the maximum head difference and 0.5 meters is the minimum head difference, the discharge of the pumps will be in between the red lines. This intervals signifies the operational range for this system.



(c) Systemcurves for system with different pipe diameters, static head = 1 meter



(d) System and performance curve for generic system and pump



(e) Operational range genereric system

Figure C.1: System and performance curves generic systems

RESULTS OPTIMIZATION MODEL

D.1. Design lifetime 40 years

Table D.1: Results design lifetime 40 years

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Asan A economic	RCP26	250	0.1	11	5	216	3.8	0.55	0.45	-27	102	-1	74	75
	RCP45	250	0.1	11	5	216	3.8	0.55	0.45	-27	102	-1	74	76
	RCP60	250	0.1	11	5	216	3.8	0.55	0.45	-26	102	-1	74	76
	RCP85	250	0.1	11	5	216	3.8	0.55	0.45	-26	102	-1	74	76
Asan A nature	RCP26	250	3	75	11	7124	3.8	0.55	0.45	-1190	3252	-61	63	2062
	RCP45	250	3	75	11	7124	3.8	0.55	0.45	-1247	3252	-66	62	2005
	RCP60	250	3	50	13.5	7153	3.8	0.55	0.45	-1245	3252	-66	62	2007
	RCP85	250	3	50	13.5	7153	3.8	0.55	0.45	-1311	3252	-71	60	1941
Asan B Econ	RCP26	100	0.1	29	2	91	4	0.3	0.1	-10	145	-1	93	135

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Table D.1 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
	RCP45	100	0.1	29	2	91	4	0.3	0.1	-10	145	-1	93	135
	RCP60	100	0.1	29	2	91	4	0.3	0.1	-10	145	-1	93	135
	RCP85	100	0.1	29	2	91	4	0.3	0.1	-11	145	-1	93	134
Asan B nature	RCP26	100	3	51	8.5	2893	3.4	0.9	0.1	-438	1315	-22	67	878
	RCP45	100	3	51	8.5	2893	3.4	0.9	0.1	-452	1315	-23	66	863
	RCP60	100	3	51	8.5	2893	3.4	0.9	0.1	-481	1315	-25	63	834
	RCP85	100	3	51	8.5	2893	3.4	0.9	0.1	-508	1315	-28	61	807
Asan C economic	RCP26	15	0.1	16	1	13	3.8	0.6	0.5	-6	10	-0.3	36	4
	RCP45	15	0.1	16	1	13	3.8	0.6	0.5	-6	10	-0.3	36	4
	RCP60	15	0.1	16	1	13	3.8	0.6	0.5	-6	10	-0.3	38	4
	RCP85	15	0.1	16	1	13	3.8	0.6	0.5	-6	10	-0.3	39	4
Brouwersdam economic	RCP26	110	0.35	47	4	590	1.7	0.4	0.1	-164	181	-8	9	17
	RCP45	110	0.35	47	4	590	1.7	0.4	0.1	-164	181	-8	9	17
	RCP60	110	0.35	47	4	590	1.7	0.4	0.1	-165	181	-8	9	16
	RCP85	110	0.35	47	4	590	1.7	0.4	0.1	-165	181	-8	9	16
Dee economic	RCP26	90	0.3	5	8	251	4	0.3	0.1	-86	176	-4	51	90
	RCP45	90	0.3	5	8	251	4	0.3	0.1	-87	176	-4	51	89
	RCP60	90	0.3	5	8	251	4	0.3	0.1	-90	176	-4	49	86
	RCP85	90	0.3	5	8	251	4	0.3	0.1	-91	176	-5	49	86
Dee nature	RCP26	90	5	65	10.5	5626	5.6	1.15	0.1	-3375	3907	-171	14	532
	RCP45	90	5	65	10.5	5626	5.6	1.15	0.1	-3388	3907	-176	13	519
	RCP60	90	5	65	10.5	5626	5.6	1.15	0.1	-3405	3907	-182	13	502
	RCP85	90	5	100	8.5	5672	5.6	1.15	0.1	-3418	3939	-176	13	521
Dharamtar A economic	RCP26	280	0.2	61	3.5	587	2.4	0.5	0.4	-155	131	-7	-18	-24
	RCP45	280	0.2	61	3.5	587	2.4	0.5	0.4	-146	131	-7	-11	-15

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Table D.1 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
	RCP60	280	0.2	61	3.5	587	2.4	0.5	0.4	-145	131	-7	-11	-14
	RCP85	280	0.2	61	3.5	587	2.4	0.5	0.4	-145	131	-7	-11	-14
Dharamtar A nature	RCP26	280	1.9	169	6	4776	3.2	0.7	0.5	-1832	1068	-95	-72	-764
	RCP45	280	1.9	169	6	4776	3.2	0.7	0.5	-1804	1068	-98	-69	-736
	RCP60	280	1.9	110	7.5	4857	3.2	0.7	0.5	-1901	1086	-95	-75	-815
	RCP85	280	1.9	110	7.5	4857	3.2	0.7	0.5	-1876	1086	-97	-73	-790
Dharamtar B economic	RCP26	20	0.1	27	1	21	2	0.5	0.4	-9	8	-0.4	-7	-1
	RCP45	20	0.1	27	1	21	2	0.5	0.4	-9	8	-0.4	-6	-1
	RCP60	20	0.1	27	1	21	2	0.5	0.4	-9	8	-0.4	-5	0
	RCP85	20	0.1	27	1	21	2	0.5	0.4	-9	8	-0.4	-5	0
Dharamtar B nature	RCP26	20	1.9	37	3.5	356	3.2	0.7	0.1	-128	83	-7	-54	-45
	RCP45	20	1.9	37	3.5	356	3.2	0.7	0.1	-127	83	-7	-52	-43
	RCP60	20	1.9	38	3.5	365	3.2	0.7	0.1	-130	83	-7	-56	-47
	RCP85	20	1.9	38	3.5	365	3.2	0.7	0.1	-129	83	-7	-55	-46
Haeju A economic	RCP26	110	0.2	40	2.5	196	4.2	0.6	0.5	-107	122	-5	13	16
	RCP45	110	0.2	40	2.5	196	4.2	0.6	0.5	-98	122	-5	19	24
	RCP60	110	0.2	40	2.5	196	4.2	0.6	0.5	-99	122	-5	19	23
	RCP85	110	0.2	40	2.5	196	4.2	0.6	0.5	-99	122	-5	19	23
Haeju A nature	RCP26	110	4.35	110	8	5526	5.8	1.31	0.1	-3389	3382	-154	0	-7
	RCP45	110	4.35	110	8	5526	5.8	1.31	0.1	-3392	3382	-155	0	-10
	RCP60	110	4.35	110	8	5526	5.8	1.31	0.1	-3398	3382	-155	0	-16
	RCP85	110	4.35	110	8	5526	5.8	1.31	0.1	-3405	3382	-155	-1	-23
Haeju B economic	RCP26	45	0.2	25	2	79	5.6	0.75	0.65	-53	51	-2	-4	-2
	RCP45	45	0.2	25	2	79	5.6	0.75	0.65	-50	51	-2	2	1
	RCP60	45	0.2	25	2	79	5.6	0.75	0.65	-50	51	-2	2	1

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Table D.1 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
	RCP85	45	0.2	25	2	79	5.6	0.75	0.65	-50	51	-2	2	1
Haeju B nature	RCP26	45	4.35	80	6	2261	5.6	1.31	0.1	-1365	1383	-62	1	18
	RCP45	45	4.35	80	6	2261	5.6	1.31	0.1	-1366	1383	-62	1	17
	RCP60	45	4.35	80	6	2261	5.6	1.31	0.1	-1368	1383	-62	1	15
	RCP85	45	4.35	80	6	2261	5.6	1.31	0.1	-1370	1383	-62	1	13
Mokpo A economic	RCP26	50	0.5	5	8.5	284	2	0.55	0.35	-127	106	-6	-20	-21
	RCP45	50	0.5	5	8.5	284	2	0.55	0.35	-115	106	-5	-8	-9
	RCP60	50	0.5	5	8.5	284	2	0.55	0.35	-115	106	-5	-8	-9
	RCP85	50	0.5	5	8.5	284	2	0.55	0.35	-115	106	-6	-8	-9
Mokpo A nature	RCP26	50	2.25	54	5.5	1282	2.8	0.65	0.1	-596	476	-27	-25	-120
	RCP45	50	2.25	54	5.5	1282	2.8	0.65	0.1	-598	476	-27	-26	-122
	RCP60	50	2.25	54	5.5	1282	2.8	0.65	0.1	-601	476	-27	-26	-125
	RCP85	50	2.875	54	5.5	1282	2.8	0.65	0.1	-603	476	-28	-27	-127
Mokpo B economic	RCP26	15	0.35	18	2	57	2	0.45	0.35	-27	25	-1	-8	-2
	RCP45	15	0.35	18	2	57	2	0.45	0.35	-27	25	-1	-8	-2
	RCP60	15	0.35	18	2	57	2	0.45	0.35	-27	25	-1	-8	-2
	RCP85	15	0.35	18	2	57	2	0.45	0.35	-27	25	-1	-8	-2
San Francisco A economic	RCP26	950	1.3	198	9	12590	1.7	0.3	0.1	-5879	4227	-268	-39	-1652
	RCP45	950	1.3	198	9	12590	1.7	0.3	0.1	-5899	4227	-269	-40	-1672
	RCP60	950	1.3	198	9	12590	1.7	0.3	0.1	-5902	4227	-272	-40	-1675
	RCP85	950	1.3	198	9	12590	1.7	0.3	0.1	-5864	4227	-273	-39	-1637
San Francisco A nature	RCP26	950	0.9	245	10	19233	1.7	0.4	0.1	-9070	6456	-413	-40	-2614
	RCP45	950	0.9	245	10	19233	1.7	0.4	0.1	-9097	6456	-414	-41	-2641
	RCP60	950	0.9	245	10	19233	1.7	0.4	0.1	-9074	6456	-417	-41	-2618

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Table D.1 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
	RCP85	950	0.9	245	10	19233	1.7	0.4	0.1	-9014	6456	-421	-40	-2558
San Francisco B economic	RCP26	500	0.35	63	7.5	2782	1.7	0.3	0.2	-1268	937	-58	-35	-331
	RCP45	500	0.35	63	7.5	2782	1.7	0.3	0.2	-1271	937	-58	-36	-334
	RCP60	500	0.35	63	7.5	2782	1.7	0.3	0.2	-1268	937	-59	-35	-331
	RCP85	500	0.35	63	7.5	2782	1.7	0.3	0.2	-1261	937	-59	-35	-324
San Francisco B nature	RCP26	500	1.2	178	9	11318	1.7	0.3	0.1	-5009	3801	-228	-32	-1208
	RCP45	500	1.2	178	9	11318	1.7	0.3	0.1	-5028	3801	-229	-32	-1227
	RCP60	500	1.2	178	9	11318	1.7	0.3	0.1	-5032	3801	-230	-32	-1231
	RCP85	500	1.2	178	9	11318	1.7	0.3	0.1	-5003	3801	-233	-32	-1202
San Francisco C economic	RCP26	300	0.45	73	6	2063	1.7	0.3	0.15	-945	695	-43	-36	-250
	RCP45	300	0.45	73	6	2063	1.7	0.3	0.15	-947	695	-43	-36	-252
	RCP60	300	0.45	73	6	2063	1.7	0.3	0.15	-946	695	-44	-36	-251
	RCP85	300	0.45	73	6	2063	1.7	0.3	0.15	-940	695	-44	-35	-245
San Francisco C nature	RCP26	300	1.2	125	6.5	4146	1.7	0.3	0.15	-1966	1394	-90	-41	-572
	RCP45	300	1.2	125	6.5	4146	1.7	0.3	0.15	-1968	1394	-90	-41	-574
	RCP60	300	1.2	125	6.5	4146	1.7	0.3	0.15	-1965	1394	-90	-41	-571
	RCP85	300	1.2	125	6.5	4146	1.7	0.3	0.15	-1955	1394	-91	-40	-561
Xinghua economic	RCP26	480	0.45	65	7	2500	2.6	0.4	0.1	-328	685	-18	52	357
	RCP45	480	0.45	57	7.5	2517	2.6	0.4	0.1	-333	685	-18	51	352
	RCP60	480	0.45	36	9.5	2550	2.6	0.4	0.1	-330	685	-18	52	355
	RCP85	480	0.45	37	9.5	2621	2.6	0.4	0.1	-323	685	-17	53	362
Xinghua nature	RCP26	480	3.15	222	10	17427	4.2	0.8	0.1	-3793	4918	-173	23	1125
	RCP45	480	3.15	222	10	17427	4.2	0.8	0.1	-3774	4918	-172	23	1144
	RCP60	480	3.15	222	10	17427	4.2	0.8	0.1	-3777	4918	-174	23	1141
	RCP85	480	3.15	222	10	17427	4.2	0.8	0.1	-3805	4918	-178	23	1113

D.2. Design lifetime 100 years

Table D.2: Results design lifetime 100 years

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Asan A economic	RCP26	250	0.1	22	4	276.32	3.4	0.4	0.1	-75	133	-1.15	80	106
	RCP45	250	0.1	22	4	276.32	3.4	0.4	0.1	-75	133	-1.15	80	106
	RCP60	250	0.1	22	4	276.32	3.4	0.4	0.1	-75	133	-1.15	80	106
	RCP85	250	0.1	22	4	276.32	3.4	0.4	0.1	-75	133	-1.15	80	106
Asan A nature	RCP26	250	3	105	9.5	7438.856	3.4	0.85	0.1	-922	3344	-42	72	2411
	RCP45	250	3	105	9.5	7439	3.4	0.85	0.1	-922	3344	-42	72	2422
	RCP60	250	3	75	11.5	7786	3.4	0.85	0.1	-1087	3533	-50	69	2446
	RCP85	250	3	82	11	7789	3.4	0.85	0.1	-1207	3555	-55	66	2348
Asan B Econ	RCP26	100	0.1	29	2	91	3.6	0.2	0.1	-27	45	-0.5	78	35
	RCP45	100	0.1	29	2	91	3.6	0.2	0.1	-27	45	-0.5	78	35
	RCP60	100	0.1	29	2	91	3.6	0.4	0.1	-27	45	-0.5	78	35
	RCP85	100	0.1	29	2	91	3.6	0.4	0.1	-27	45	-0.5	78	35
Asan B nature	RCP26	100	3	52	8.5	2949	3.6	0.85	0.1	-379	344	-17	-10	-35
	RCP45	100	3	52	8.5	2949	3.6	0.85	0.1	-377	344	-17	-10	-33
	RCP60	100	3	68	7.5	3003	3.6	0.85	0.4	-403	1369	-18	71	966
	RCP85	100	3	68	7.5	3003	3.6	0.85	0.4	-408	1369	-19	70	961
Asan C economic	RCP26	15	0.1	4	2	13	3.6	0.6	0.5	-7	10	-0.3	30	3
	RCP45	15	0.1	4	2	13	3.6	0.6	0.5	-7	10	-0.3	30	3
	RCP60	15	0.1	4	2	13	3.6	0.6	0.5	-7	10	-0.3	30	3
	RCP85	15	0.1	4	2	13	3.6	0.6	0.5	-7	10	-0.3	30	3

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Table D.2 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Brouwersdam economic	RCP26	110	0.35	47	4	590	1.7	0.4	0.1	-164	181	-7.5	9	17
	RCP45	110	0.35	63	3.5	606	1.7	0.4	0.1	-170	186	-8	9	16
	RCP60	110	0.35	14	8	703	1.7	0.4	0.1	-207	215	-9	4	8
	RCP85	110	0.35	14	8.5	794	1.7	0.4	0.1	-246	242	-11	-2	-4
Dee economic	RCP26	90	0.3	38	3	268	4	0.6	0.5	-93	187	-4	50	94
	RCP45	90	0.3	38	3	268	4	0.6	0.5	-89	187	-4	52	98
	RCP60	90	0.3	39	3	276	4	0.6	0.5	-90	193	-4	53	103
	RCP85	90	0.3	41	3	290	4	0.6	0.5	-95	203	-4	53	108
Dee nature	RCP26	90	5	67	10.5	5799	5.4	1	0.1	-3460	4022	-158	14	562
	RCP45	90	5	74	10	5809	5.4	1	0.1	-3461	4029	-158	14	568
	RCP60	90	5	83	9.5	5880	4.8	0.7	0.1	-3582	4061	-164	12	479
	RCP85	90	5	135	8	6782	5	0.8	0.2	-4472	4691	-203	5	219
Dharamtar A economic	RCP26	280	0.2	61	3.5	587	2.4	0.45	0.35	-178	131	-7	-36	-47
	RCP45	280	0.2	61	3.5	587	2.4	0.45	0.35	-153	131	-7	-17	-22
	RCP60	280	0.2	61	3.5	587	2.4	0.45	0.35	-146	131	-7	-11	-15
	RCP85	280	0.2	64	3.5	615	2.4	0.45	0.35	-152	137	-7	-11	-15
Dharamtar A nature	RCP26	280	1.9	138	7.5	6093.563	3.4	0.2	0.1	-1765	1370	-80	-29	-395
	RCP45	280	1.9	138	7.5	6093.563	3.4	0.2	0.1	-1765	1370	-80	-29	-395
	RCP60	280	1.9	138	7.5	6093.563	3.4	0.3	0.1	-1802	1370	-82	-32	-432
	RCP85	280	1.9	138	7.5	6093.563	3.4	0.3	0.1	-1834	1370	-82	-34	-464
Dharamtar B economic	RCP26	20	0.1	27	1	21.195	2	0.5	0.4	-9	8.4	-0.4	-7	-0.6
	RCP45	20	0.1	27	1	21.195	2	0.5	0.4	-9	8.4	-0.4	-7	-0.6
	RCP60	20	0.1	27	1	21.195	2	0.5	0.4	-9	8.4	-0.4	-7	-0.6
	RCP85	20	0.1	28	1	21.98	2	0.5	0.4	-9	8.4	-0.4	-7	-0.6

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Table D.2 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Dharamtar B nature	RCP26	20	1.9	42	3.5	403.8825	3	0.7	0.1	-138	93	-6	-48	-45
	RCP45	20	1.9	42	3.5	403.8825	3	0.7	0.1	-130	93	-6	-40	-37
	RCP60	20	1.9	42	3.5	403.8825	3	0.7	0.1	-128	93	-6	-38	-35
	RCP85	20	1.9	42	3.5	403.8825	3	0.7	0.1	-128	93	-6	-38	-35
Haeju A economic	RCP26	110	0.2	40	2.5	196.25	3.8	0.6	0.5	-106	122	-4	13	16
	RCP45	110	0.2	40	2.5	196.25	3.8	0.6	0.5	-92	122	-4	25	30
	RCP60	110	0.2	40	2.5	196.25	3.8	0.6	0.5	-93	122	-4	24	29
	RCP85	110	0.2	40	2.5	196.25	3.8	0.6	0.5	-93	122	-4	24	29
Haeju A nature	RCP26	110	4.35	110	8	5526.4	5.8	1.3	0.1	-3389	3382	-154	0	-7
	RCP45	110	4.35	110	8	5526.4	5.8	1.3	0.1	-3392	3382	-154	0	-10
	RCP60	110	4.35	110	8	5526.4	5.8	1.3	0.1	-3398	3382	-154	0	-16
	RCP85	110	4.35	110	8	5526.4	5.8	1.3	0.1	-3405	3382	-154	-1	-23
Haeju B economic	RCP26	45	0.2	25	2	78.5	5.4	0.75	0.65	-53	51.7	-2	-3	-1.3
	RCP45	45	0.2	25	2	78.5	5.4	0.75	0.65	-49	51.7	-2	5	2.7
	RCP60	45	0.2	25	2	78.5	5.4	0.75	0.65	-49	51.7	-2	5	2.7
	RCP85	45	0.2	25	2	78.5	5.4	0.75	0.65	-49	51.7	-2	5	2.7
Haeju B nature	RCP26	45	4.35	80	6	2260.8	5.6	1.3	0.1	-1365	1383	-62	1	18
	RCP45	45	4.35	80	6	2260.8	5.6	1.3	0.1	-1366	1383	-62	1	17
	RCP60	45	4.35	80	6	2260.8	5.6	1.3	0.1	-1368	1383	-62	1	15
	RCP85	45	4.35	80	6	2260.8	5.6	1.3	0.1	-1370	1383	-62	1	13
Mokpo A economic	RCP26	50	0.5	5	8.5	283.5813	2	0.55	0.35	-122	106	-5	-15	-16
	RCP45	50	0.5	5	8.5	283.5813	2	0.55	0.35	-117	106	-5	-10	-11
	RCP60	50	0.5	5	9	317.925	1.8	0.55	0.35	-126	117	-6	-8	-9
	RCP85	50	0.5	5	9.5	354.2313	1.8	0.45	0.35	-135	131	-6	-3	-4

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Table D.2 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Mokpo A nature	RCP26	50	2.25	54	5.5	1282.298	2.8	0.65	0.1	-595	476	-27	-25	-119
	RCP45	50	2.25	54	5.5	1282.298	2.8	0.65	0.1	-597	476	-27	-25	-121
	RCP60	50	2.25	54	5.5	1282.298	2.8	0.65	0.2	-601	476	-27	-26	-125
	RCP85	50	2.875	54	5.5	1282.298	2.8	0.65	0.2	-605	476	-28	-27	-129
Mokpo B economic	RCP26	15	0.35	18	2	56.52	2.4	0.45	0.35	-27	24	-1.2	-13	-3
	RCP45	15	0.35	18	3	127.17	2.4	0.45	0.35	-27	24	-1.2	-13	-3
	RCP60	15	0.35	18	4	226.08	2.4	0.45	0.35	-27	24	-1.2	-13	-3
	RCP85	15	0.35	18	5	353.25	2.4	0.45	0.35	-27	24	-1.2	-13	-3
San Francisco A economic	RCP26	950	1.3	198	9	12589.83	1.7	0.3	0.1	-5879	4227	-267	-39	-1652
	RCP45	950	1.3	198	9	12589.83	1.7	0.3	0.1	-5899	4227	-269	-40	-1672
	RCP60	950	1.3	180	10	14130	1.7	0.2	0.1	-6586	4744	-300	-39	-1842
	RCP85	950	1.3	236	13	31308.94	1.7	0.2	0.1	-14628	10564	-668	-38	-4064
San Francisco A nature	RCP26	950	0.9	245	10	19232.5	1.7	0.4	0.1	-9069	6456	-413	-40	-2613
	RCP45	950	0.9	245	10	19232.5	1.7	0.4	0.1	-9097	6456	-414	-41	-2641
	RCP60	950	0.9	288	10	22608	1.7	0.2	0.1	-10706	7629	-488	-40	-3077
	RCP85	950	0.9	290	12	32781.6	1.7	0.2	0.1	-15505	11060	-709	-40	-4445
San Francisco B economic	RCP26	500	0.35	63	7.5	2781.844	1.7	0.3	0.2	-1268	937	-58	-35	-331
	RCP45	500	0.35	63	7.5	2781.844	1.7	0.3	0.2	-1271	937	-58	-36	-334
	RCP60	500	0.35	87	7.5	3841.594	1.9	0.4	0.3	-1818	1299	-83	-40	-519
	RCP85	500	0.35	87	10	6829.5	1.9	0.4	0.2	-3189	2307	-147	-38	-882
San Francisco B nature	RCP26	500	1.2	178	9	11318.13	1.7	0.3	0.1	-5009	3801	-228	-32	-1208
	RCP45	500	1.2	178	9	11318.13	1.7	0.3	0.1	-5028	3801	-229	-32	-1227
	RCP60	500	1.2	154	10	12089	1.9	0.2	0.1	5372	4081	-244	232	9453
	RCP85	500	1.2	218	10	17113	1.9	0.2	0.1	-7644	5775	-349	-32	-1869

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Table D.2 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
San Francisco C economic	RCP26	300	0.45	73	6	2062.98	1.7	0.3	0.15	-945	695	-43	-36	-250
	RCP45	300	0.45	73	6	2062.98	1.7	0.3	0.15	-947	695	-43	-36	-252
	RCP60	300	0.45	112	5.5	2659.58	1.9	0.25	0.15	-1266	900	-58	-41	-366
	RCP85	300	0.45	64	8.5	3629.84	1.9	0.25	0.15	-1731	1228	-79	-41	-503
San Francisco C nature	RCP26	300	1.2	124	6.5	4112.615	1.7	0.4	0.1	-1945	1383	-89	-41	-562
	RCP45	300	1.2	124	6.5	4112.615	1.7	0.4	0.1	-1946	1383	-89	-41	-563
	RCP60	300	1.2	129	6.5	4278.446	1.7	0.2	0.1	-2019	1439	-92	-40	-580
	RCP85	300	1.2	172	6	4860.72	1.9	0.2	0.1	-2363	1643	-108	-44	-720
Xinghua economic	RCP26	480	0.45	96	6	2712.96	2.6	0.5	0.4	-268	738	-12	64	470
	RCP45	480	0.45	98	6	2769.48	2.8	0.5	0.4	-275	758	-13	64	483
	RCP60	480	0.45	24	13	3183.96	4.2	0.5	0.4	-427	901	-19	53	474
	RCP85	480	0.45	88	7.5	3885.75	2.4	0.5	0.4	-587	1048	-27	44	461
Xinghua nature	RCP26	480	3.15	222	10	17427	4.2	0.8	0.1	-3792	4918	-172	23	1126
	RCP45	480	3.15	222	10	17427	4.2	0.8	0.1	-3774	4918	-172	23	1144
	RCP60	480	3.15	219	10.5	18953.63	3.2	0.8	0.25	-3784	5225	-172	28	1441
	RCP85	480	3.15	245	10.5	21203.83	3.2	0.85	0.4	-4626	5845	-211	21	1219

D.3. Design lifetime 100 years, adaptive approach

Table D.3: Results adaptive design approach, lifetime 100 years

	Climate scenario	**Basin area (km2)**	**min tidal stroke (m)**	**n**	**D (m)**	**A (m2)**	**Hrated (m)**	**Hstart (m)**	**Hstop (m)**	**NPV (M€)**	**total (M€)**	**EEA (M€)**	**ROI (%)**	**profit (M€)**
Asan A economic	RCP26	250	0.1	23	4	289	3.6	0.55	0.1	-27	135	-1.2	0.80	108
	RCP45	250	0.1	23	4	289	3.6	0.55	0.1	-27	135	-1.2	0.80	108
	RCP60	250	0.1	23	4	289	3.6	0.55	0.1	-27	135	-1.2	0.80	108
	RCP85	250	0.1	23	4	289	3.6	0.55	0.1	-27	135	-1.2	0.80	108
Asan A nature	RCP26	250	3	97	10.5	8395	3	0.75	0.1	-1338	3796	-61	0.65	2458
	RCP45	250	3	97	10.5	8395	3	0.75	0.1	-1348	3796	-61	0.64	2448
	RCP60	250	3	97	10.5	8395	3	0.75	0.1	-1338	3796	-61	0.65	2458
	RCP85	250	3	97	10.5	8395	3	0.75	0.1	-1414	3796	-61	0.63	2382
Asan B Econ	RCP26	100	0.1	35	2	110	3.8	0.55	0.1	-12	54	-0.5	0.78	42
	RCP45	100	0.1	35	2	110	3.8	0.55	0.1	-12	54	-0.5	0.78	42
	RCP60	100	0.1	35	2	110	3.8	0.55	0.1	-12	54	-0.5	0.78	42
	RCP85	100	0.1	35	2	110	3.8	0.55	0.1	-12	54	-0.5	0.78	42
Asan B nature	RCP26	100	3	57	9	3624	2.4	0.6	0.1	-709	1617	-32	0.56	908
	RCP45	100	3	57	9	3624	2.4	0.6	0.1	-726	1617	-32	0.55	891
	RCP60	100	3	57	9	3624	2.4	0.6	0.1	-725	1617	-32	0.55	892
	RCP85	100	3	57	9	3624	2.4	0.6	0.1	-752	1617	-33	0.53	865
Asan C economic	RCP26	15	0.1	3	3	21	3.2	0.75	0.5	-9	13	-0.4	0.31	4
	RCP45	15	0.1	3	3	21	3.2	0.75	0.5	-9	13	-0.4	0.31	4
	RCP60	15	0.1	3	3	21	3.2	0.75	0.5	-9	13	-0.4	0.31	4
	RCP85	15	0.1	3	3	21	3.2	0.75	0.5	-9	13	-0.4	0.31	4
Brouwersdam economic	RCP26	110	0.35	40	4.5	636	1.7	0.5	0.1	-194	195	-9	0.01	1
	RCP45	110	0.35	40	4.5	636	1.7	0.5	0.1	-213	195	-9	-0.09	-18
	RCP60	110	0.35	25	8	1256	0.7	0.3	0.1	-391	360	-18	-0.09	-31
	RCP85	110	0.35	42	6.5	1393	0.7	0.3	0.1	-445	399	-20	-0.12	-46

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Table D.3 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Dee economic	RCP26	90	0.3	52	2.5	255	4.2	0.4	0.3	-93	181	-4	0.49	88
	RCP45	90	0.3	52	2.5	255	4.2	0.4	0.3	-84	179	-4	0.53	95
	RCP60	90	0.3	15	5	294	4.2	0.4	0.1	-96	206	-4	0.53	110
	RCP85	90	0.3	24	4	301	4.2	0.6	0.1	-99	210	-5	0.53	111
Dee nature	RCP26	90	5	124	8	6230	4.2	1.15	0.1	-3707	4282	-168.9	0.13	575
	RCP45	90	5	124	8	6230	4.2	1.15	0.1	-3700	4282	-168.5	0.14	582
	RCP60	90	5	110	9	6994	4.2	1.15	0.1	-4415	4807	-201	0.08	392
	RCP85	90	5	155	8	7787	4.2	1.15	0.1	-5177	5351	-235.8	0.03	174
Dharamtar A economic	RCP26	280	0.2	83	4	1042	1.8	0.3	0.1	-228	223	-10	-0.02	-5
	RCP45	280	0.2	83	4	1042	1.8	0.3	0.1	-243	223	-10	-0.09	-20
	RCP60	280	0.2	83	4	1042	1.8	0.3	0.1	-244	223	-10	-0.09	-21
	RCP85	280	0.2	83	4	1042	1.8	0.3	0.1	-244	223	-10	-0.09	-21
Dharamtar A nature	RCP26	280	1.9	191	8	9596	1.6	0.3	0.1	-2124	1996	-92.9	-0.06	-128
	RCP45	280	1.9	191	8	9596	1.6	0.3	0.1	-2146.8	1996	-92.6	-0.08	-151
	RCP60	280	1.9	191	8	9596	1.6	0.3	0.1	-2022.9	1996	-92.6	-0.01	-27
	RCP85	280	1.9	192	8	9646	1.6	0.3	0.1	-2063.3	2007	-92	-0.03	-56
Dharamtar B economic	RCP26	20	0.1	4	3.5	38	2	0.3	0.1	-15	14	-0.7	-0.07	-1
	RCP45	20	0.1	4	3.5	38	2	0.3	0.1	-15	14	-0.7	-0.07	-1
	RCP60	20	0.1	4	3.5	38	2	0.3	0.1	-18	14	-0.7	-0.29	-4
	RCP85	20	0.1	4	3.5	38	2	0.3	0.1	-18	14	-0.7	-0.29	-4
Dharamtar B nature	RCP26	20	1.9	48	4.5	763	1.4	0.3	0.1	-161.7	192	-7	0.16	30
	RCP45	20	1.9	48	4.5	763	1.4	0.3	0.1	-163.7	192	-7	0.15	28
	RCP60	20	1.9	48	4.5	763	1.4	0.3	0.1	-163.7	192	-7	0.15	28
	RCP85	20	1.9	48	4.5	763	1.4	0.3	0.1	-166.8	192	-7	0.13	25

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Table D.3 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
Haeju A economic	RCP26	110	0.2	40	3	283	4	0.3	0.1	-136	174	-6	0.22	38
	RCP45	110	0.2	40	3	283	4	0.3	0.1	-120	174	-6	0.31	54
	RCP60	110	0.2	40	3	283	4	0.3	0.1	-122	174	-6	0.30	52
	RCP85	110	0.2	40	3	283	4	0.3	0.1	-123	174	-6	0.29	51
Haeju A nature	RCP26	110	4.35	139	9.5	9848	4	1.21	0.1	-6652.6	5931	-303	-0.12	-722
	RCP45	110	4.35	139	9.5	9848	4	1.21	0.1	-6658	5931	-303	-0.12	-727
	RCP60	110	4.35	139	9.5	9848	4	1.21	0.1	-6659	5931	-303	-0.12	-728
	RCP85	110	4.35	139	9.5	9848	4	1.21	0.1	-6657.7	5931	-303	-0.12	-727
Haeju B economic	RCP26	45	0.2	17	3	120	4.2	0.3	0.1	-66	76	-3	0.13	10
	RCP45	45	0.2	17	3	120	4.2	0.3	0.1	-57	76	-3	0.25	19
	RCP60	45	0.2	17	3	120	4.2	0.3	0.1	-58	76	-3	0.24	18
	RCP85	45	0.2	17	3	120	4.2	0.3	0.1	-58	76	-3	0.24	18
Haeju B nature	RCP26	45	4.35	38	11.5	3945	4	1	0.1	-2698	2378	-123	-0.13	-320
	RCP45	45	4.35	38	11.5	3945	4	1	0.1	-2701	2378	-123	-0.14	-323
	RCP60	45	4.35	38	11.5	3945	4	1	0.1	-2702	2378	-123	-0.14	-324
	RCP85	45	4.35	38	11.5	3945	4	1	0.1	-2703	2378	-123	-0.14	-325
Mokpo A economic	RCP26	50	0.5	48	4	603	1.4	0.3	0.1	-231	216	-10	-0.07	-15
	RCP45	50	0.5	48	4	603	1.4	0.3	0.1	-231	216	-10	-0.07	-15
	RCP60	50	0.5	48	4	603	1.4	0.3	0.1	-226	216	-10	-0.05	-10
	RCP85	50	0.5	48	4	603	1.4	0.3	0.1	-227	216	-10	-0.05	-11
Mokpo B economic	RCP26	15	0.35	3	8	151	1.2	0.35	0.1	-63	57	-3	-0.11	-6
	RCP45	15	0.35	3	8	151	1.2	0.35	0.1	-63	57	-3	-0.11	-6
	RCP60	15	0.35	3	8	151	1.2	0.35	0.1	-63	57	-3	-0.11	-6
	RCP85	15	0.35	3	8	151	1.2	0.35	0.1	-63	57	-3	-0.11	-6

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Table D.3 – continued from previous page

	Climate scenario	Basin area (km2)	min tidal stroke (m)	n	D (m)	A (m2)	Hrated (m)	Hstart (m)	Hstop (m)	NPV (M€)	total (M€)	EEA (M€)	ROI (%)	profit (M€)
San Francisco A economic	RCP26	950	1.3	261	11.5	27096	1.9	0.3	0.1	-12601	9969	-563.8	-0.26	-2632
	RCP45	950	1.3	261	11.5	27096	1.9	0.3	0.1	-12597.8	9143	-563.8	-0.38	-3455
	RCP60	950	1.3	261	11.5	27096	1.9	0.3	0.1	-12406	9143	-565	-0.36	-3263
	RCP85	950	1.3	262	12.5	32136	1.9	0.3	0.1	-12382.9	9178	-568	-0.35	-3205
San Francisco B economic	RCP26	500	0.35	152	6.5	5041	1	0.3	0.1	-2003	1647	-91	-0.22	-356
	RCP45	500	0.35	152	6.5	5041	1	0.3	0.1	-2016	1647	-91	-0.22	-369
	RCP60	500	0.35	350	10	27475	0.2	0.3	0.1	-9793	8185	-450	-0.20	-1608
	RCP85	500	0.35	400	10	31400	0.2	0.3	0.1	-11193.7	9353	-520	-0.20	-1841
San Francisco C economic	RCP26	300	0.45	120	6.5	3980	0.8	0.3	0.1	-1527.7	1279	-69.6	-0.19	-249
	RCP45	300	0.45	142	6	4013	0.8	0.3	0.1	-1542.1	1290	-70.2	-0.20	-252
	RCP60	300	0.45	251	7	9655	0.8	0.3	0.1	-3524.9	2961	-162.1	-0.19	-564
	RCP85	300	0.45	283	7	10886	0.8	0.3	0.1	-3982.8	3338	-185.4	-0.19	-645
San Francisco C Nature	RCP26	300	1.2	116	9	7376	1.9	0.3	0.1	-3575.8	2491	-136	-0.44	-1085
	RCP45	300	1.2	116	9	7376	1.9	0.3	0.1	-3577	2491	-136	-0.44	-1086
	RCP60	300	1.2	116	9	7376	1.9	0.3	0.1	-3031	2491	-138	-0.22	-540
	RCP85	300	1.2	116	9	7376	1.9	0.3	0.1	-3031	2491	-140	-0.22	-540
Xinghua economic	RCP26	480	0.45	36	10	2826	2.8	0.65	0.4	-287	773	-13	0.63	486
	RCP45	480	0.45	84	7	3231	2.4	0.5	0.1	-369	872	-17	0.58	503
	RCP60	480	0.45	94	7	3616	2.4	0.45	0.1	-476.6	976	-22	0.51	499
	RCP85	480	0.45	40	11.5	4153	2.4	0.45	0.35	-654	1120	-30	0.42	466
Xinghua nature	RCP26	480	3.15	288	10	22608	2.4	0.8	0.1	-4847.9	6083	-220	0.20	1235
	RCP45	480	3.15	288	10	22608	2.4	0.8	0.1	-4823	6083	-220	0.21	1260
	RCP60	480	3.15	288	10	22608	2.4	0.8	0.1	-4757	6083	-218	0.22	1326
	RCP85	480	3.15	306	10	24021	2.4	0.8	0.1	-5337	6463	-244	0.17	1126

APPENDIX
E

RESULTS BUSINESS CASES

E.1. Culvert design lifetime 40 years

Table E.1: Culvert design lifetime 40 years

Location	Strategy	RCP	Flow area m^2	NPV (x-10^6 €)
Asan A	economic	RCP26	292	-73
Asan A	economic	RCP45	292	-73
Asan A	economic	RCP60	292	-73
Asan A	economic	RCP85	292	-73
Asan A	nature	RCP26	8385	-2083
Asan A	nature	RCP45	9643	-2398
Asan A	nature	RCP60	9643	-2398
Asan A	nature	RCP85	9643	-2398
Asan B	economic	RCP26	119	-30
Asan B	economic	RCP45	119	-30
Asan B	economic	RCP60	119	-30
Asan B	economic	RCP85	119	-30
Asan B	nature	RCP26	3405	-847
Asan B	nature	RCP45	3405	-846
Asan B	nature	RCP60	3915	-974
Asan B	nature	RCP85	3915	-974
Asan C	economic	RCP26	17	-4
Asan C	economic	RCP45	17	-4
Asan C	economic	RCP60	17	-4
Asan C	economic	RCP85	17	-4
Brouwersdam	economic	RCP26	730	-104
Brouwersdam	economic	RCP45	869	-124
Brouwersdam	economic	RCP60	869	-124
Brouwersdam	economic	RCP85	869	-124
Dee	economic	RCP26	251	-106
Dee	economic	RCP45	251	-106
Dee	economic	RCP60	251	-106
Dee	economic	RCP85	251	-106
Dee	nature	RCP26	5628	-2364
Dee	nature	RCP45	5628	-2364
Dee	nature	RCP60	5628	-2364

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Table E.1 – continued from previous page

Location	Strategy	RCP	Flow area m^2	NPV (x-10^6 €)
Dee	nature	RCP85	5628	-2364
Dharamtar A	economic	RCP26	587	-49
Dharamtar A	economic	RCP45	587	-49
Dharamtar A	economic	RCP60	587	-49
Dharamtar A	economic	RCP85	587	-49
Dharamtar A	nature	RCP26	n.a.	n.a.
Dharamtar A	nature	RCP45	n.a.	n.a.
Dharamtar A	nature	RCP60	n.a.	n.a.
Dharamtar A	nature	RCP85	n.a.	n.a.
Dharamtar B	economic	RCP26	21	-2
Dharamtar B	economic	RCP45	21	-2
Dharamtar B	economic	RCP60	21	-2
Dharamtar B	economic	RCP85	21	-2
Dharamtar B	nature	RCP26	n.a.	n.a.
Dharamtar B	nature	RCP45	n.a.	n.a.
Dharamtar B	nature	RCP60	n.a.	n.a.
Dharamtar B	nature	RCP85	n.a.	n.a.
Haeju A	economic	RCP26	289	-104
Haeju A	economic	RCP45	289	-104
Haeju A	economic	RCP60	289	-104
Haeju A	economic	RCP85	289	-104
Haeju A	nature	RCP26	n.a.	n.a.
Haeju A	nature	RCP45	n.a.	n.a.
Haeju A	nature	RCP60	n.a.	n.a.
Haeju A	nature	RCP85	n.a.	n.a.
Haeju B	economic	RCP26	123	-44
Haeju B	economic	RCP45	123	-44
Haeju B	economic	RCP60	123	-44
Haeju B	economic	RCP85	123	-44
Haeju B	nature	RCP26	n.a.	n.a.
Haeju B	nature	RCP45	n.a.	n.a.
Haeju B	nature	RCP60	n.a.	n.a.
Haeju B	nature	RCP85	n.a.	n.a.
Mokpo A	economic	RCP26	618	-115
Mokpo A	economic	RCP45	684	-128
Mokpo A	economic	RCP60	684	-128
Mokpo A	economic	RCP85	751	-140
Mokpo A	nature	RCP26	n.a.	n.a.
Mokpo A	nature	RCP45	n.a.	n.a.
Mokpo A	nature	RCP60	n.a.	n.a.
Mokpo A	nature	RCP85	n.a.	n.a.
Mokpo B	economic	RCP26	110	-20
Mokpo B	economic	RCP45	110	-20
Mokpo B	economic	RCP60	123	-23
Mokpo B	economic	RCP85	123	-23
San Francisco A	economic	RCP26	27415	-4463
San Francisco A	economic	RCP45	27415	-4463
San Francisco A	economic	RCP60	27415	-4463
San Francisco A	economic	RCP85	31120	-5064
San Francisco A	nature	RCP26	n.a.	n.a.

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Table E.1 – continued from previous page

Location	Strategy	RCP	Flow area m^2	NPV (x-10^6 €)
San Francisco A	nature	RCP45	n.a.	n.a.
San Francisco A	nature	RCP60	n.a.	n.a.
San Francisco A	nature	RCP85	n.a.	n.a.
San Francisco B	economic	RCP26	4807	-783
San Francisco B	economic	RCP45	5820	-947
San Francisco B	economic	RCP60	5820	-947
San Francisco B	economic	RCP85	5820	-947
San Francisco B	nature	RCP26	n.a.	n.a.
San Francisco B	nature	RCP45	n.a.	n.a.
San Francisco B	nature	RCP60	n.a.	n.a.
San Francisco B	nature	RCP85	n.a.	n.a.
San Francisco C	economic	RCP26	3278	-534
San Francisco C	economic	RCP45	3278	-534
San Francisco C	economic	RCP60	3278	-534
San Francisco C	economic	RCP85	3278	-534
San Francisco C	nature	RCP26	n.a.	n.a.
San Francisco C	nature	RCP45	n.a.	n.a.
San Francisco C	nature	RCP60	n.a.	n.a.
San Francisco C	nature	RCP85	n.a.	n.a.
Xinghua	economic	RCP26	2501	-318
Xinghua	economic	RCP45	2501	-318
Xinghua	economic	RCP60	2501	-318
Xinghua	economic	RCP85	2501	-318
Xinghua	nature	RCP26	21538	-2737
Xinghua	nature	RCP45	21538	-2737
Xinghua	nature	RCP60	21538	-2737
Xinghua	nature	RCP85	21538	-2737

E.2. Culvert design lifetime 100 years

Table E.2: Designs culvert 100 years

Location	RCP	Flow area m^2	NPV (x-10^6 €)
	RCP	A_c	NPV_c
Asan A economic	RCP26	340	-86
	RCP45	340	-86
	RCP60	384	-97
	RCP85	517	-131
Asan A nature	RCP26	11383	-2885
	RCP45	12696	-3218
	RCP60	n.a	n.a
	RCP85	n.a	n.a
Asan B Econ	RCP26	135	-34
	RCP45	135	-34
	RCP60	166	-42
	RCP85	212	-54
Asan B nature	RCP26	4513	-1144
	RCP45	5034	-1276
	RCP60	n.a.	n.a.

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Table E.2 – continued from previous page

Location	RCP	Flow area m^2	NPV (x-10^6 €)
	RCP85	n.a.	n.a.
Asan C economic	RCP26	20	-5
	RCP45	23	-6
	RCP60	23	-6
	RCP85	30	-8
Brouwersdam economic	RCP26	1809	-264
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Dee economic	RCP26	269	-115
	RCP45	269	-115
	RCP60	269	-115
	RCP85	332	-142
Dee nature	RCP26	5883	-2513
	RCP45	8023	-3427
	RCP60	8023	-3426
	RCP85	n.a.	n.a.
Dharamtar A economic	RCP26	1554	-137
	RCP45	1830	-161
	RCP60	2796	-246
	RCP85	n.a.	n.a.
Dharamtar A nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Dharamtar B economic	RCP26	404	-36
	RCP45	404	-36
	RCP60	404	-36
	RCP85	404	-36
Dharamtar B nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Haeju A economic	RCP26	335	-123
	RCP45	335	-123
	RCP60	335	-123
	RCP85	427	-157
Haeju A nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Haeju B economic	RCP26	140	-52
	RCP45	140	-52
	RCP60	157	-58
	RCP85	191	-70
Haeju B nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Mokpo A economic	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
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Table E.2 – continued from previous page

Location	RCP	Flow area m^2	NPV ($x\text{-}10^6$ €)
	RCP85	n.a.	n.a.
Mokpo A nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Mokpo B economic	RCP26	172	-33
	RCP45	192	-37
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco A economic	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco A nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco B economic	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco B nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco C economic	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
San Francisco C nature	RCP26	n.a.	n.a.
	RCP45	n.a.	n.a.
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.
Xinghua economic	RCP26	2771	-366
	RCP45	3423	-452
	RCP60	3423	-452
	RCP85	4075	-538
Xinghua nature	RCP26	25641	-3387
	RCP45	25641	-3387
	RCP60	n.a.	n.a.
	RCP85	n.a.	n.a.

E.3. Business cases design lifetime 40 years

Table E.3: Business cases Dam with culvert, Updating coastal defences and dam with TPP, design lifetime 40 years

	Climate scenario	**SLR (m)**	**NPV Dam with culvert ($x\text{-}10^6$ €)**	**NPV Updating coastal defences* ($x\text{-}10^6$ €)**	**NPV Dam with TPP ($x\text{-}10^6$ €)**
Asan A economic	RCP26	0.17	1315	2095	1306
Asan A economic	RCP45	0.21	1315	2095	1306

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Table E.3 – continued from previous page

	Climate scenario	SLR (m)	NPV Dam with culvert (x-10^6 €)	NPV Updating* coastal defences (x-10^6 €)	NPV Dam with TPP (x-10^6 €)
Asan A economic	RCP60	0.23	1326	2095	1306
Asan A economic	RCP85	0.26	1358	2095	1306
Asan A nature	RCP26	0.17	4000	2095	5178
Asan A nature	RCP45	0.21	4319	2095	5178
Asan A nature	RCP60	0.23	4634	2095	5178
Asan A nature	RCP85	0.26	4627	2095	5178
Asan B economic	RCP26	0.17	431	1213	426
Asan B economic	RCP45	0.21	431	1213	426
Asan B economic	RCP60	0.23	439	1213	426
Asan B economic	RCP85	0.26	451	1213	426
Asan B nature	RCP26	0.17	1496	1213	1931
Asan B nature	RCP45	0.21	1623	1213	1931
Asan B nature	RCP60	0.23	1875	1213	1931
Asan B nature	RCP85	0.26	1744	1213	1931
Asan C economic	RCP26	0.17	34	220500	35
Asan C economic	RCP45	0.21	35	220500	35
Asan C economic	RCP60	0.23	35	220500	35
Asan C economic	RCP85	0.26	37	220500	35
Brouwersdam economic	RCP26	0.2	286	220500	193
Brouwersdam economic	RCP45	0.23	285	220500	193
Brouwersdam economic	RCP60	0.25	285	220500	194
Brouwersdam economic	RCP85	0.29	284	220500	194
Dee economic	RCP26	0.25	808	1131	782
Dee economic	RCP45	0.25	808	1131	783
Dee economic	RCP60	0.25	808	1131	786
Dee economic	RCP85	0.25	834	1131	786
Dee nature	RCP26	0.25	3136	1131	4071
Dee nature	RCP45	0.25	4023	1131	4084
Dee nature	RCP60	0.25	4022	1131	4101
Dee nature	RCP85	0.25	4017	1131	4114
Dharamtar A economic	RCP26	0.17	1856	794	1909
Dharamtar A economic	RCP45	0.21	1874	794	1900
Dharamtar A economic	RCP60	0.23	1937	794	1900
Dharamtar A economic	RCP85	0.26	1918	794	1900
Dharamtar A nature	RCP26	0.17	2691	794	3586
Dharamtar A nature	RCP45	0.21	2783	794	3558
Dharamtar A nature	RCP60	0.23	2773	794	3656
Dharamtar A nature	RCP85	0.26	n.a.	794	3630
Dharamtar B economic	RCP26	0.17	335	159	314
Dharamtar B economic	RCP45	0.21	335	159	313
Dharamtar B economic	RCP60	0.23	335	159	313
Dharamtar B economic	RCP85	0.26	335	159	313
Dharamtar B nature	RCP26	0.17	389	159	432
Dharamtar B nature	RCP45	0.21	388	159	431
Dharamtar B nature	RCP60	0.23	387	159	435
Dharamtar B nature	RCP85	0.26	386	159	434
Haeju A economic	RCP26	0.16	626	1543	614
Haeju A economic	RCP45	0.18	626	1543	606
Haeju A economic	RCP60	0.20	626	1543	606

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Table E.3 – continued from previous page

	Climate scenario	SLR (m)	NPV Dam with culvert ($x\text{-}10^6$ €)	NPV Updating* coastal defences ($x\text{-}10^6$ €)	NPV Dam with TPP ($x\text{-}10^6$ €)
Haeju A economic	RCP85	0.23	658	1543	607
Haeju A nature	RCP26	0.16	n.a.	1543	3896
Haeju A nature	RCP45	0.18	n.a.	1543	3900
Haeju A nature	RCP60	0.20	n.a.	1543	3906
Haeju A nature	RCP85	0.23	n.a.	1543	3912
Haeju B economic	RCP26	0.16	151	992	154
Haeju B economic	RCP45	0.18	151	992	152
Haeju B economic	RCP60	0.20	157	992	152
Haeju B economic	RCP85	0.23	169	992	152
Haeju B nature	RCP26	0.16	n.a.	992	1466
Haeju B nature	RCP45	0.18	n.a.	992	1468
Haeju B nature	RCP60	0.20	n.a.	992	1470
Haeju B nature	RCP85	0.23	n.a.	992	1472
Mokpo A economic	RCP26	0.17	1341	1102	1287
Mokpo A economic	RCP45	0.21	1341	1102	1275
Mokpo A economic	RCP60	0.23	1340	1102	1275
Mokpo A economic	RCP85	0.26	1339	1102	1275
Mokpo A nature	RCP26	0.17	n.a.	1102	1756
Mokpo A nature	RCP45	0.21	n.a.	1102	1758
Mokpo A nature	RCP60	0.23	n.a.	1102	1761
Mokpo A nature	RCP85	0.26	n.a.	22	290603
Mokpo B economic	RCP26	0.17	61	220500	56
Mokpo B economic	RCP45	0.21	65	220500	56
Mokpo B economic	RCP60	0.23	72	220500	56
Mokpo B economic	RCP85	0.26	72	220500	56
San Francisco A economic	RCP26	0.17	n.a.	14112	10040
San Francisco A economic	RCP45	0.21	n.a.	14112	10060
San Francisco A economic	RCP60	0.23	n.a.	14112	10064
San Francisco A economic	RCP85	0.26	n.a.	14112	10026
San Francisco A nature	RCP26	0.17	n.a.	14112	13232
San Francisco A nature	RCP45	0.21	n.a.	14112	13258
San Francisco A nature	RCP60	0.23	n.a.	14112	13236
San Francisco A nature	RCP85	0.26	n.a.	14112	13176
San Francisco B economic	RCP26	0.17	n.a.	8820	2892
San Francisco B economic	RCP45	0.21	n.a.	8820	2895
San Francisco B economic	RCP60	0.23	n.a.	8820	2892
San Francisco B economic	RCP85	0.26	n.a.	8820	2885
San Francisco B nature	RCP26	0.17	n.a.	8820	6633
San Francisco B nature	RCP45	0.21	n.a.	8820	6652
San Francisco B nature	RCP60	0.23	n.a.	8820	6656
San Francisco B nature	RCP85	0.26	n.a.	8820	6627
San Francisco C economic	RCP26	0.17	2013	4586	2062
San Francisco C economic	RCP45	0.21	2011	4586	2064
San Francisco C economic	RCP60	0.23	2008	4586	2062
San Francisco C economic	RCP85	0.26	n.a.	4586	2056
San Francisco C nature	RCP26	0.17	n.a.	4586	3082
San Francisco C nature	RCP45	0.21	n.a.	4586	3084
San Francisco C nature	RCP60	0.23	n.a.	4586	3082
San Francisco C nature	RCP85	0.26	n.a.	4586	3072

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Table E.3 – continued from previous page

	Climate scenario	SLR (m)	NPV Dam with culvert (x-10^6 €)	NPV Updating* coastal defences (x-10^6 €)	NPV Dam with TPP (x-10^6 €)
Xinghua economic	RCP26	0.2	4903	1233	4997
Xinghua economic	RCP45	0.23	4958	1233	5002
Xinghua economic	RCP60	0.25	4958	1233	4999
Xinghua economic	RCP85	0.29	5013	1233	4992
Xinghua nature	RCP26	0.2	6834	1233	8462
Xinghua nature	RCP45	0.23	6834	1233	8443
Xinghua nature	RCP60	0.25	7172	1233	8446
Xinghua nature	RCP85	0.29	7156	1233	8474

*SLR used for updating coastal defences is 0.25 m, given accuracy of heigtening

E.4. Business cases design lifetime 100 years

Table E.4: Business cases Dam with culvert, Updating coastal defences and dam with TPP, design lifetime 100 years

	Climate scenario	SLR (m)	NPV Dam with culvert (x-10^6 €)	NPV Updating coastal defences (x-10^6 €)	NPV Dam with TPP (x-10^6 €)
Asan A economic	RCP26	0.45	1315	2095	1306
Asan A economic	RCP45	0.54	1364	2514	1356
Asan A economic	RCP60	0.675	1448	3142	1430
Asan A economic	RCP85	0.9	1604	4189	1553
Asan A nature	RCP26	0.45	4000	2095	5590
Asan A nature	RCP45	0.54	4363	2514	5639
Asan A nature	RCP60	0.675	n.a.	3142	5713
Asan A nature	RCP85	0.9	n.a.	4189	5836
Asan B economic	RCP26	0.45	431	1213	426
Asan B economic	RCP45	0.54	447	1455	442
Asan B economic	RCP60	0.675	479	1819	466
Asan B economic	RCP85	0.9	530	2425	506
Asan B nature	RCP26	0.45	1496	1213	2131
Asan B nature	RCP45	0.54	1636	1455	2147
Asan B nature	RCP60	0.675	n.a.	1819	2171
Asan B nature	RCP85	0.9	n.a.	2425	2210
Asan C economic	RCP26	0.45	34	220500	36
Asan C economic	RCP45	0.54	38	264600	39
Asan C economic	RCP60	0.675	42	330750	43
Asan C economic	RCP85	0.9	51	441000	50
Brouwersdam economic	RCP26	0.5	286	220500	193
Brouwersdam economic	RCP45	0.6	n.a.	264600	202
Brouwersdam economic	RCP60	0.75	n.a.	330750	243
Brouwersdam economic	RCP85	1.0	n.a.	441000	290
Dee economic	RCP26	0.375	808	1131	789
Dee economic	RCP45	0.45	831	1357	808
Dee economic	RCP60	0.5625	865	1697	844
Dee economic	RCP85	0.75	949	2262	907
Dee nature	RCP26	0.375	3136	1131	3274
Dee nature	RCP45	0.45	4043	1357	3275
Dee nature	RCP60	0.5625	4072	1697	3371

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Table E.4 – continued from previous page

	Climate scenario	SLR (m)	NPV Dam with culvert (x-10^6 €)	NPV Updating coastal defences (x-10^6 €)	NPV Dam with TPP (x-10^6 €)
Dee nature	RCP85	0.75	n.a.	2262	3885
Dharamtar A economic	RCP26	0.45	1856	794	1964
Dharamtar A economic	RCP45	0.54	1905	953	1996
Dharamtar A economic	RCP60	0.675	2014	1191	2044
Dharamtar A economic	RCP85	0.9	n.a.	1588	2124
Dharamtar A nature	RCP26	0.45	n.a.	794	4020
Dharamtar A nature	RCP45	0.54	n.a.	953	4052
Dharamtar A nature	RCP60	0.675	n.a.	1191	4100
Dharamtar A nature	RCP85	0.9	n.a.	1588	4180
Dharamtar B economic	RCP26	0.45	335	159	318
Dharamtar B economic	RCP45	0.54	343	191	326
Dharamtar B economic	RCP60	0.675	356	238	339
Dharamtar B economic	RCP85	0.9	377	318	361
Dharamtar B nature	RCP26	0.45	n.a.	159	424
Dharamtar B nature	RCP45	0.54	n.a.	191	433
Dharamtar B nature	RCP60	0.675	n.a.	238	446
Dharamtar B nature	RCP85	0.9	n.a.	318	468
Haeju A economic	RCP26	0.4	626	1543	610
Haeju A economic	RCP45	0.48	640	1852	624
Haeju A economic	RCP60	0.6	662	2315	646
Haeju A economic	RCP85	0.8	730	3087	682
Haeju A nature	RCP26	0.4	n.a.	1543	6180
Haeju A nature	RCP45	0.48	n.a.	1852	6194
Haeju A nature	RCP60	0.6	n.a.	2315	6216
Haeju A nature	RCP85	0.8	n.a.	3087	6253
Haeju B economic	RCP26	0.4	151	992	146
Haeju B economic	RCP45	0.48	154	1191	148
Haeju B economic	RCP60	0.6	164	1488	154
Haeju B economic	RCP85	0.8	n.a.	1984	161
Haeju B nature	RCP26	0.4	n.a.	992	1226
Haeju B nature	RCP45	0.48	n.a.	1191	1228
Haeju B nature	RCP60	0.6	n.a.	1488	1233
Haeju B nature	RCP85	0.8	n.a.	1984	1239
Mokpo A economic	RCP26	0.45	n.a.	1102	1282
Mokpo A economic	RCP45	0.54	n.a.	1323	1292
Mokpo A economic	RCP60	0.675	n.a.	1654	1322
Mokpo A economic	RCP85	0.9	n.a.	2205	1368
Mokpo A nature	RCP26	0.45	n.a.	1102	1731
Mokpo A nature	RCP45	0.54	n.a.	1323	1746
Mokpo A nature	RCP60	0.675	n.a.	1654	1776
Mokpo A nature	RCP85	0.9	n.a.	44	435637
Mokpo B economic	RCP26	0.45	61	220500	60
Mokpo B economic	RCP45	0.54	68	264600	63
Mokpo B economic	RCP60	0.675	n.a.	330750	67
Mokpo B economic	RCP85	0.9	n.a.	441000	74
San Francisco A economic	RCP26	0.45	n.a.	14112	10040
San Francisco A economic	RCP45	0.54	n.a.	16934	10081
San Francisco A economic	RCP60	0.675	n.a.	21168	10798
San Francisco A economic	RCP85	0.9	n.a.	28224	18891

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Table E.4 – continued from previous page

	Climate scenario	**SLR (m)**	**NPV Dam with culvert (x-10^6 €)**	**NPV Updating coastal defences (x-10^6 €)**	**NPV Dam with TPP (x-10^6 €)**
San Francisco A nature	RCP26	0.45	n.a.	14112	13230
San Francisco A nature	RCP45	0.54	n.a.	16934	13279
San Francisco A nature	RCP60	0.675	n.a.	21168	14918
San Francisco A nature	RCP85	0.9	n.a.	28224	19768
San Francisco B economic	RCP26	0.45	n.a.	8820	2892
San Francisco B economic	RCP45	0.54	n.a.	10584	2915
San Francisco B economic	RCP60	0.675	n.a.	13230	3493
San Francisco B economic	RCP85	0.9	n.a.	17640	4914
San Francisco B nature	RCP26	0.45	n.a.	8820	6633
San Francisco B nature	RCP45	0.54	n.a.	10584	6672
San Francisco B nature	RCP60	0.675	n.a.	13230	7047
San Francisco B nature	RCP85	0.9	n.a.	17640	9370
San Francisco C economic	RCP26	0.45	n.a.	4586	2062
San Francisco C economic	RCP45	0.54	n.a.	5504	2084
San Francisco C economic	RCP60	0.675	n.a.	6880	2433
San Francisco C economic	RCP85	0.9	n.a.	9173	2949
San Francisco C nature	RCP26	0.45	n.a.	4586	3062
San Francisco C nature	RCP45	0.54	n.a.	5504	3083
San Francisco C nature	RCP60	0.675	n.a.	6880	3186
San Francisco C nature	RCP85	0.9	n.a.	9173	3581
Xinghua economic	RCP26	0.5	4903	1233	4937
Xinghua economic	RCP45	0.6	4997	1479	4985
Xinghua economic	RCP60	0.75	5056	1849	5198
Xinghua economic	RCP85	1.0	5207	2465	5459
Xinghua nature	RCP26	0.5	6834	1233	8461
Xinghua nature	RCP45	0.6	6864	1479	8484
Xinghua nature	RCP60	0.75	n.a.	1849	8554
Xinghua nature	RCP85	1.0	n.a.	2465	9498