Sustainable Development of Long Island, Singapore

Optimizing the land reclamation design for climate resilience

MSc Thesis

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Haskoning
Enhancing Society Together

Sustainable Development of Long Island, Singapore

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by

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Preface

With this thesis, I conclude the Master of Science in Civil Engineering at Delft University of Technology. Over the past months, I have had the opportunity to work on a challenging and rewarding topic in collaboration with Haskoning. I am grateful that I got the opportunity to contribute to such a unique project and to visit Singapore, where I was able to present my preliminary findings. This project has been a great learning experience for me, on an academic as well as professional level.

I would like to thank some people who have supported me during this project. First of all, I would like to thank my supervisors. Matthijs, thank you for your trust and support throughout the project, which helped me stay confident and focused. And thank you for your efforts in making the Singapore visit possible. The trip was very inspiring for me. Maarten, thank you for your enthusiasm and inspiration, which helped me push through to the finish line. Bas and Olivier, thank you for your creative and honest perspectives, which helped me get to the fundamentals of my research.

A big thank you to my parents for their trust and for giving me the freedom to follow my own path. And to my loyal brother, thank you for always cheering me on and inspiring me to be a better person.

To all the friends I have met during my student years: thank you for your kindness and humor. The memories we have made together will always make me look back on these years with a smile.

Finally, a thank you to everyone at Haskoning for welcoming me and creating such a fun and supportive environment, which made this challenging process still very enjoyable.

Cilia Boot
Rotterdam, June 2025

Abstract

Singapore's Long Island project aims to protect the East Coast, meet freshwater demands, and support urban development. It involves constructing a freshwater reservoir by closing off part of the sea using three islands and two barrages. The islands, totaling 850 ha, will be used for urban development. The project is currently in its conceptual design phase.

Long Island presents several challenges. Singapore's flood risk policy focuses on raising the platform level, increasing demand for scarce construction materials. The multifunctional nature of Long Island, providing flood protection, freshwater supply, and urban space, complicates design. Uncertainty in future sea level rise (SLR) further challenges sea defense planning and adaptation.

This thesis develops a resilient conceptual design for Long Island's land reclamation, focusing on platform level optimization and sea defense adaptability. Six reclamation variants are proposed, ranging from polder systems to conventional landfills, combined with a caisson or a dike as sea defenses. Sea defenses are designed to accommodate up to 5 m SLR and are integrated into adaptation pathways. Each variant considers reservoir dike design, effective land area, settlements, and polder pumping requirements.

Designs are evaluated through capital cost analysis, lifetime cost assessments using Present Value, Multi-Criteria Analysis (MCA), and sensitivity analyses on design parameters, Social Discount Rates (SDRs) and SLR projections.

The most cost-effective design combines a platform level of -4 m SHD with either a dike or caisson. This polder approach is technically feasible and reduces reclamation volumes by 80 million m^3 and saves 3 billion SGD compared to a 5.1 m SHD design. Sensitivity analyses confirm its robustness under varying assumptions. Both sea defense types are adaptable and have comparable costs, though further research is needed to determine the optimal choice, including geotechnical design and nature-based integration. The MCA did not yield a clear preference due to close value-cost ratios and a lack of stakeholder validation.

While technically and economically promising, the polder system's societal acceptance and integration into Singapore's urban context require further assessment. Future design phases should address public perception of flood risk, desirability of polder developments, and nature-inclusive coastal environments, supported by stakeholder engagement. Additional research into flood risk, the polder pumping system, and SDRs is recommended to improve the design and inform decision-makers on platform level selection.

This thesis provides a technical foundation for Long Island's next design stages and supports platform level decision-making. It also offers insights for other regions pursuing land reclamation developments, especially where unit rates are high and/or materials are scarce, demonstrating an integral optimization approach focused on multifunctionality and climate resilience.

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Nomenclature

CAPEX	Capital Expenditures
CD	Chart Datum
GWT	Ground Water Table
HDB	Housing and Development Board
MCA	Multi Criteria Analysis
MSL	Mean Sea Level
PV	Present Value
OPEX	Operating Expenditures
PUB	Public Utilities Board
SDR	Social Discount Rate
SEA	South East Asia
SGD	Singaporean Dollars
SHD	Singapore Height Datum
SLR	Sea Level Rise
SSP	Shared Socioeconomic Pathway

1 Introduction

1.1. Background

The city-state of Singapore is one of the world's most densely populated countries, with approximately 7,600 residents per square kilometer (The World Bank, 2023). Given its limited land area and growing population, Singapore is one of the most water-stressed nations globally (Ministry of Sustainability and the Environment, Singapore, 2024). To meet its freshwater needs, Singapore partially relies on imports from Malaysia (Ministry of Foreign Affairs, Singapore, 2024). Climate change intensifies this challenge, as it brings more intense rainfall and longer dry spells (National Climate Change Secretariat, Singapore, 2024b), leading to higher demands for effective water management. In response, Singapore has undertaken projects to increase its freshwater capacity, including the construction of several reservoirs (PUB, 2024). But as the water demand is expected to almost double in 2065, additional steps are required to provide future needs (PUB, 2025). Next to that, Singapore faces an increase in flood risk due to future sea level rise (SLR). Singapore is a low-lying country with 30% of the land area being less than 5 above Mean Sea Level (MSL), therefore, the country is facing large impacts due to SLR (Koh, 2023). In Figure 1.1, these low-lying areas are indicated. One of these vulnerable areas is East Coast Park and its hinterland, located in the south of Singapore along the Singapore Strait.

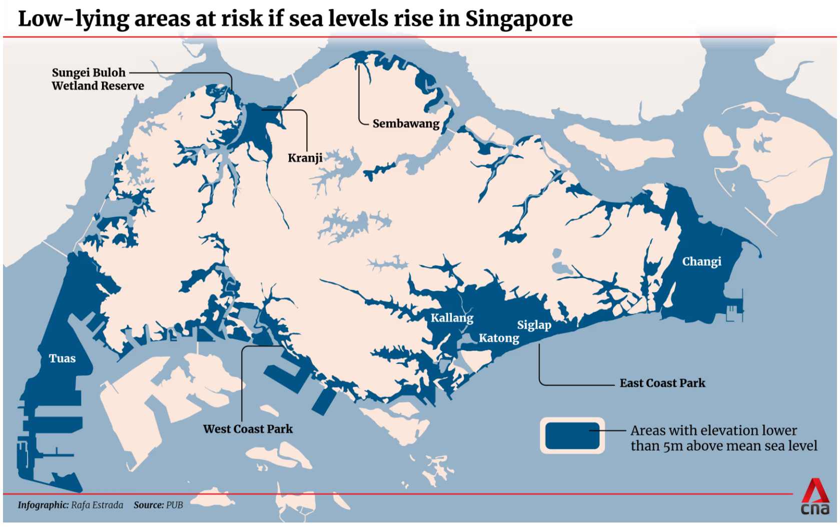


Figure 1.1: Overview of low-lying areas in Singapore (Koh, 2023). Blue shaded areas indicate elevation lower than 5 meters above MSL. East Coast Park is found in the southeast.

To protect Singapore's East Coast, meet freshwater demands, and support urban development, the Long Island project was initiated. Long Island will consist of the construction of a freshwater reservoir by closing off a part of the sea. This closure is achieved by the construction of three islands and two barrages. These islands will be used for urban development. The project's lay-out is illustrated in Figure 1.2, with preliminary estimates for land dimensions and the reservoir area already developed. At the end of 2024, further technical studies of Long Island commenced (Ng & Begum, 2023).

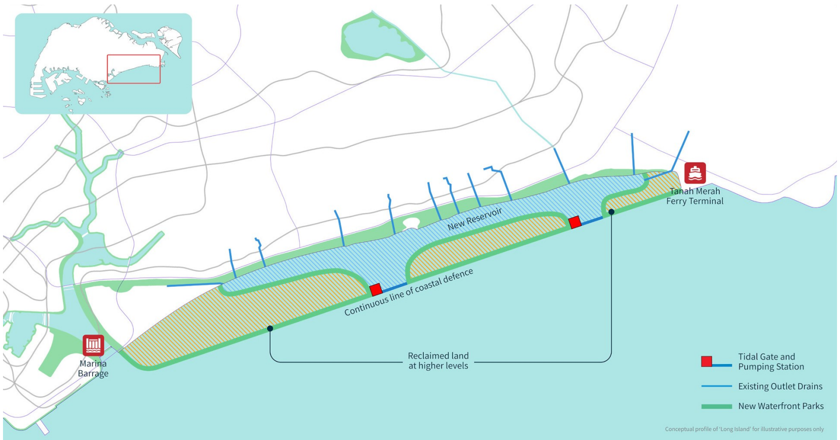


Figure 1.2: Lay-out of Long Island (Urban Redevelopment Authority, 2024), showing the three islands, the reservoir, and the barrages.

1.2. Problem analysis

The proposed Long Island project will reshape Singapore's coastline, bringing along physical, environmental and societal challenges. This section presents the challenges considered in this thesis.

1.2.1. Platform level requirements and material scarcity

Singapore's flood risk policy focuses on increasing platform level requirements, leading to a significant demand for building materials, which are limited in supply. The same protection level can be achieved in a more material efficient manner by reinforcing the flood defenses while lowering the platform level. Platform level optimization requires an integral approach due to its inflexible character and widespread impact on Long Island's functionality and operations.

In 2011, the Public Utilities Board (PUB) increased the minimum platform level requirement for land reclamation projects from +3 m to +4 m Singapore Height Datum (SHD) (National Climate Change Secretariat, Singapore, 2024a). For the Long Island project, this standard has been further elevated to +4.5 m SHD, with a probability of being raised to +5.1 m SHD. The platform level is the ground elevation of the land reclamation, and raising of platform levels results in substantial additional volumes of sand required for the construction of Long Island.

A more material efficient strategy relevant for Singapore is lowering Long Island's platform level and reinforcing the flood defenses, whilst maintaining the same safety level. However, lower platform levels could lead to a reduction of the spatial quality of Long Island, as the low platform levels can be challenging to integrate with the high sea defenses. The public of Singapore expects Long Island to be a high-quality living environment, where the coastline also serves a recreational purpose and is well-integrated with the urban environment (Begum, 2024). Moreover, the lower platform levels could lead to problems in the future due to SLR: seepage issues can occur or large pumps are needed to pump out the collected water. These aspects should all be taken into consideration when deciding upon a platform level.

Limiting these additional required reclamation volumes is relevant due to Singapore's tight sand market. There is a high demand in sand due to Singapore's large-scale civil infrastructure developments and land reclamation projects. This scale can be illustrated by the territory expansion of Singapore through land reclamation, which has been more than 23% over the past 45 years (United Nations Environment Programme, 2019). Next to Singapore, the global demand in sand is increasing due to climate change and global economic development (World Economic Forum, 2023). But the local supply of sand in Singapore is very limited due to its natural coastal system and its small area. To overcome this, Singapore imports the material from Southeast Asian (SEA) nations, resulting in Singapore being the world's largest importer of sand (United Nations Environment Programme, 2019). But these imports are limited: several countries such as Indonesia, Vietnam, Cambodia, and Malaysia have over time implemented several temporary bans on sand exports to Singapore due to environmental and political reasons (Torres et al., 2021). This further tightens the local sand market.

1.2.2. Fragmented design approach

The Long Island development results in a complex system that serves multiple functions and provides protection against various hazards. Optimizing the platform level adds further complexity to the design process. This can lead to a fragmented design approach, where each function or hazard is addressed in isolation due to differing engineering expertise, resulting in suboptimal overall performance as interconnections are not considered. To avoid this, an integral design approach is required that aligns Long Island's functions and hazards and enables platform level optimizations.

The main functions of Long Island, being flood protection, urban development, and water storage, are all interdependent on each other. For example, the flood defenses can have an impact on the spatial quality of the urban development. And the operations of the reservoir impact the flood defense requirements at the reservoir side. The same accounts for the flood risks: the coastal, pluvial, and reservoir flooding are all interrelated. They influence and intensify each other through shared infrastructure and operational constraints. For example, during heavy rainfall, the reservoir may reach capacity, preventing further drainage and causing pluvial flooding. Alternatively, continuing to drain into an already full reservoir could result in reservoir flooding. The design of Long Island should therefore consider these functions and risks not individually, but as one integral system. The same applies to platform-level optimization, where an integrated approach could enhance the performance and resilience of Long Island's overall system.

1.2.3. Climate change uncertainty

The uncertainty of future SLR complicates the decision-making process for the sea defense design and its potential adaptation strategy. An adaptive pathway approach could be promising in addressing this uncertainty in sea defense design, but it is not applicable for the whole land reclamation design optimization of Long Island due to lock-in design decisions or high-cost flexibility.

Future SLR projections exhibit a wide range of outcomes due to uncertainties in socio-economic developments, natural climate variability, and limitations in climate models. The variety of SLR projections is illustrated in Figure 1.3 with SLR ranging from 1.5 to 6 meters by 2150 (Meteorological Service Singapore, 2020). These projections, based on the IPCC's Sixth Assessment Report (AR6), are influenced by three key components:

- Shared Socioeconomic Pathways (SSPs): These describe potential future developments in technology, society, and policy that influence greenhouse gas emissions and, consequently, SLR (ClimateData.ca, n.d.). For example, SSP5-8.5 represents a high-emission scenario, while SSP2-4.5 is considered intermediate.
- Confidence levels: These reflect the degree of scientific agreement and evidence supporting each projection (Mastrandrea et al., 2010).
- Percentiles: These indicate which upper bound of the projected SLR range is taken.

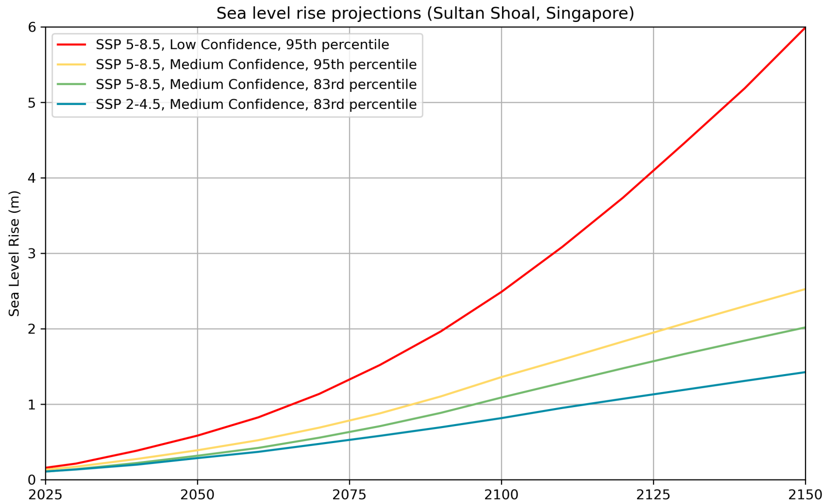


Figure 1.3: Four different SLR projections until 2150, based on SSP, confidence level, and percentile for Sultan Shoal, Singapore (Meteorological Service Singapore, 2020).

Flood management policies are often optimized based on a single climate scenario, similar to the strategy employed by the PUB. However, the actual climate trajectory may deviate significantly from the chosen scenario. This could result in unnecessary investments when planning for the worst-case scenario, whilst the actual SLR trajectory is lower than expected. On the other hand, higher SLR than expected could lead to functional failure of the design. A dynamic optimization approach, such as the adaptive pathway method, can address this by planning for multiple scenarios and incorporating flexibility into the decision-making process (Haasnoot et al., 2013).

However, the adaptive pathway approach has limited applicability in the optimization of Long Island's overall design. For example, the approach cannot be applied to platform level optimization, as the platform level cannot be adapted in the future due to urban development. The same accounts for the foundation design or the sea defense type: alterations can be made, but at a high cost.

1.2.4. Problem statement

Long Island has a multifunctional purpose: to reduce flood risk for East Coast Park and its hinterland, enhance freshwater resources, and support urban development. This multi-functionality, in combination with climate change uncertainty, high-quality standards, and limited material supplies, requires an integral approach for Long Island's conceptual design. Climate change requires the design of Long Island to be future-proof. The Singaporean government aims to do this by raising the platform level requirement for land reclamation projects. But the sand required to do this is significant, even though this material is scarce. Moreover, the amount and speed of SLR is unknown, whilst the design lifetime of Long Island is long, further challenging the decision-making process in the design phase.

1.3. Objective and scope

1.3.1. Design objective

The goal of this thesis is to develop a resilient conceptual design for Long Island using an integrated design approach, with a primary focus on sea defense design and platform level determination. To achieve this, multiple conceptual land reclamation variants will be developed, each incorporating a different sea defense option and/or platform level. The adaptability of the sea defense will be assessed by including long-term SLR. The design aims to provide a detailed comparison of these variants and includes considerations of both coastal and pluvial flooding processes. Each variant will be evaluated based on economic effectiveness and its long-term system performance. A sensitivity analysis will also be conducted to assess how changes in factors such as SLR projections and the Social Discount Rate (SDR) affect the evaluation outcomes.

1.3.2. Scope

The platform levels range from polder reclamations to conventional landfills. For the sea defense options, a dike and caisson structure are considered. The research addresses the adaptation of the sea defenses for up to 5 m SLR. Differences in effective land area and settlements between a polder and landfill reclamations are corrected for. Basic reservoir modeling is performed so that the required material volumes and investments for the reservoir defense is estimated. Further optimization of the reservoir design is not considered in this research. The land reclamation alternatives are further worked out in three adaptive pathway strategies. These alternatives and strategies are evaluated in a cost analysis and Multi Criteria Analysis (MCA). In the MCA, residual flood risk is assessed qualitatively, extensive flood risk analysis is not performed. This scope ensures an integral, yet comprehensive evaluation of the conceptual design of Long Island.

1.3.3. Research questions

The main research question guiding this thesis is:

> **What is a resilient conceptual design of Long Island's land reclamation based on system performance and economic effectiveness?**

This question is supported by the following sub-questions, each corresponding to chapters in the thesis:

1. What are the main drivers influencing the optimization of Long Island's land reclamation design?
2. What are the conceptual land reclamation designs based on the platform levels and sea defense options considered?
3. How can the land reclamation design be adapted to address sea level rise, and what initial robust design choices are required?
4. Which land reclamation design is most favorable when considering both system performance and long-term cost-effectiveness?
5. What are the key sensitivities influencing the evaluation of the land reclamation designs?

1.4. Approach and thesis outline

The approach of this thesis is explained per chapter. In Figure 1.4 an overview of this approach is visualized, indicating the main elements per chapter and the connection between them.

Chapter 2: System analysis First, an area analysis is performed in which the natural processes occurring in Long Island and Singapore are described. The function analysis gives insight into the main functions that Long Island should provide. The relationship analysis explains the differences between polder reclamations and conventional reclamations and presents several reference projects.

Chapter 3: Basis of the design The first part of the Basis of the design consists of the program of requirements, including functional and general structural requirements. Next, the program of evaluation criteria is defined. Then, the design lifetime of infrastructure, the adaptation pathways, and the costs are defined. Finally, the boundary conditions are quantified based on the area analysis.

Chapter 4: Concept design First, the sea defense design is presented per defense type, including the adaptations needed for rising sea levels. Afterwards, some additional land reclamation components are described or designed, such as the reservoir defense, the effective land area correction, the polder pumping system, and settlements.

Chapter 5: Evaluation The cost analysis is split up in two parts: first, the conceptual designs are evaluated based on initial capital expenditures, next, the adaptation pathways are evaluated based on Present Value calculations. Afterwards, a Multi Criteria Analysis is performed based on the evaluation criteria defined in Chapter 3. In the Multi Criteria Analysis, the value-cost ratio is calculated to summarize the evaluation of the concept design and adaptation pathways. Finally, a sensitivity analysis is performed, including variations in design parameters, social discount rates, and different sea level rise scenarios.

Chapter 6: Discussion In this chapter, the approach and findings of the thesis are discussed.

Chapter 7: Conclusion Based on the research questions, the thesis is concluded in this chapter.

Chapter 8: Recommendations In the final chapter, strategic and design recommendations are made.

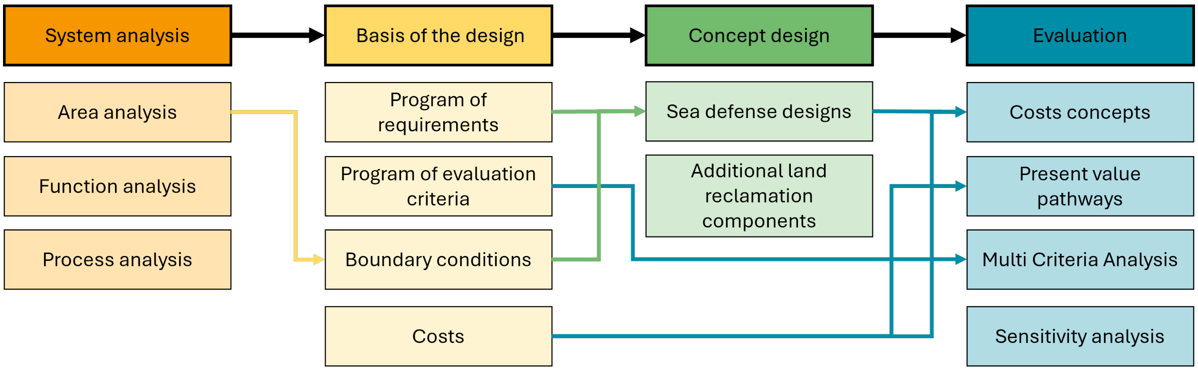


Figure 1.4: Chapter outline of this thesis, indicating the elements of each chapter and the main connection between them.

2

System analysis

This system analysis provides an orientation of Long Island's environment, its functions, and associated processes. In Section 2.1, the area analysis is found. Next, the main functions of Long Island are listed in the function analysis in Section 2.2. In Section 2.3, the hydrological processes at Long Island are described, and reference projects are presented.

2.1. Area analysis

2.1.1. Location Singapore

Singapore has an area of 740 km^2 and is located in South East Asia, on the South of Malaysia (Singapore Department of Statistics, 2025). Singapore is due to its position sheltered from high energy coastal environments: Sumatra protects Singapore from the Indian Ocean, and Malaysia protects Singapore from waves coming from the South Chinese Sea. Due to this unique position, combined with sufficient depths in the Singapore Strait, results in Singapore has been the center of trading for centuries. Still to this day, this is the case: the Port of Singapore is one of the largest in the world, situated in the West of Singapore.

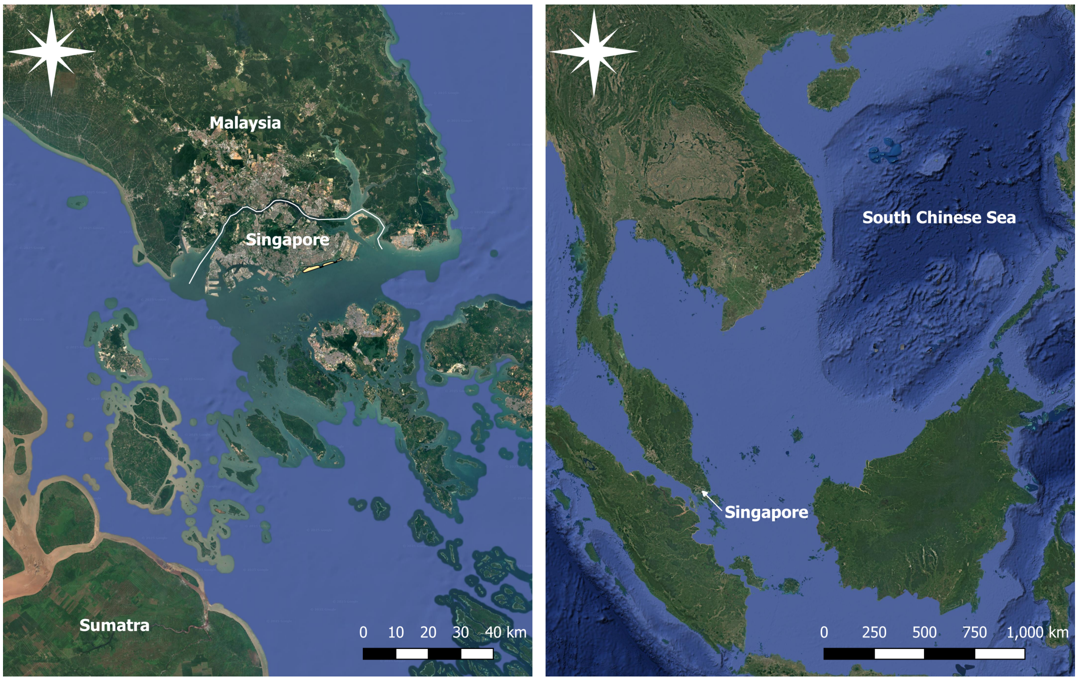


Figure 2.1: Left: the scale of Singapore and the location of Long Island. Right: Location of Singapore in South East Asia (Google Earth, n.d.).

2.1.2. Climate

Singapore's climate is characterized by its monsoon climate. There are two monsoon seasons: the northeast monsoon and the southwest monsoon. In between the monsoon seasons, there are inter-monsoon periods in which the winds are variable and afternoon thunderstorms are common. The northeast monsoon has prevailing winds originating in the north-northeast direction blowing over the South Chinese Sea. The southwest monsoon has prevailing winds coming from the direction of Sumatra, in the southwest to south. The monsoon climate is associated with certain weather characteristics, which are described in Table 2.1. The historical monthly average rainfall per month is shown in Figure 2.2.

Season	Period	Prevailing winds	Weather characteristics
Northeast monsoon	Dec. - early Mar.	N-NE	Wet phase (Dec.-early Jan.): moderate to heavy rain. Dry phase (Jan.-early Mar.): windy, relatively dry.
Inter-monsoon	Late Mar. - May	Light and variable	Hot afternoons. Thunderstorms in early evening.
Southwest monsoon	Jun. - Sep.	SW-S	Sumatra squalls: wind gusts in the morning. Short duration of showers and thunderstorms in the afternoon.
Inter-monsoon	Oct. - Nov.	Light and variable	Thunderstorms in early evening. Wetter than the other inter-monsoon period.

Table 2.1: Weather characteristics of Singapore, split up into four periods.

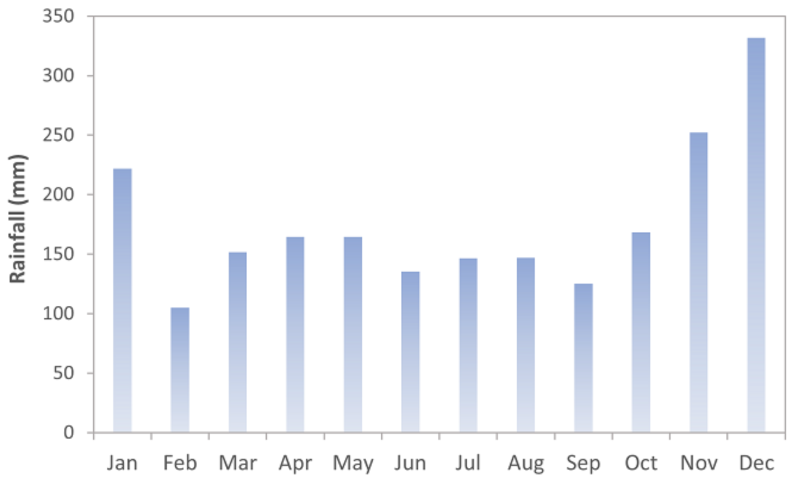


Figure 2.2: Monthly rainfall based on historical data from Changi Climate Station from 1991 to 2020 (Ministry of Transport Singapore, 2024).

2.1.3. Hydraulic conditions

Due to the location of Singapore at the Southern end of the South China Sea, extreme high water levels in Singapore occur during prolonged northeast winds (Cannaby et al., 2016), so during the Northeast monsoon. In contrast to the water levels, the largest wave heights occur during southwest winds because this orientation results in the longest fetch. In this way, the largest waves and highest water levels cannot coincide.

2.1.4. Lay-out Long Island

The Long Island project consists of the construction of three islands and a coastal reservoir, which can be seen in Figure 2.3. In Table 2.2, the dimensions of the islands and the defense lines are listed.

In the figure, the three islands are indicated in light yellow, the reservoir is closed off by the islands and by the barrages. The islands are labeled A, B and C. Island A is the western island and the biggest one, B is in the middle, and C is on the eastern side. The islands are on the south side, protected against the sea by sea defenses, indicated in white. On the northern side, the islands are also protected by the reservoir, indicated in light gray.

Water flows into the reservoir by several streams, which are fed by rainfall. The watersheds of the reservoir are the area of land on which the rain falls that will drain into the reservoir. Watershed boundaries are identified by the highest elevation points, such as mountain ridges or hills, which act as natural dividers between different watersheds (Earth Site Education, 2025). The watersheds of the Long Island reservoir are determined by analyzing the topography of Singapore. A Digital Elevation Model (DEM) is utilized based on data from the Shuttle Radar Topography Mission (SRTM) in 2000 (NASA Earthdata, 2025). The application is further explained in Appendix **??**. The resulting area of the watersheds is $A_{ws} = 4250\ ha$ and is shown in the gray-blue shade in the Figure. The water flowing out of the reservoir into the sea is regulated at the barrage using gates or pumps. The total length of the barrages is about 1.5 kilometers and the area of the reservoir is $A_r = 500\ ha$.

The land use around Long Island is quite diverse. On the east side, the Changi airport is situated. On the Northern side of Long Island, East Coast Park is found, with a coastline length of 9.6 kilometers. The Marina Barrage is just on the west side of Long Island, with Marina Bay in the hinterland. The proposed location of Long Island is where the anchorage for the Port of Singapore is now placed, this will have to be moved to another location or further offshore.

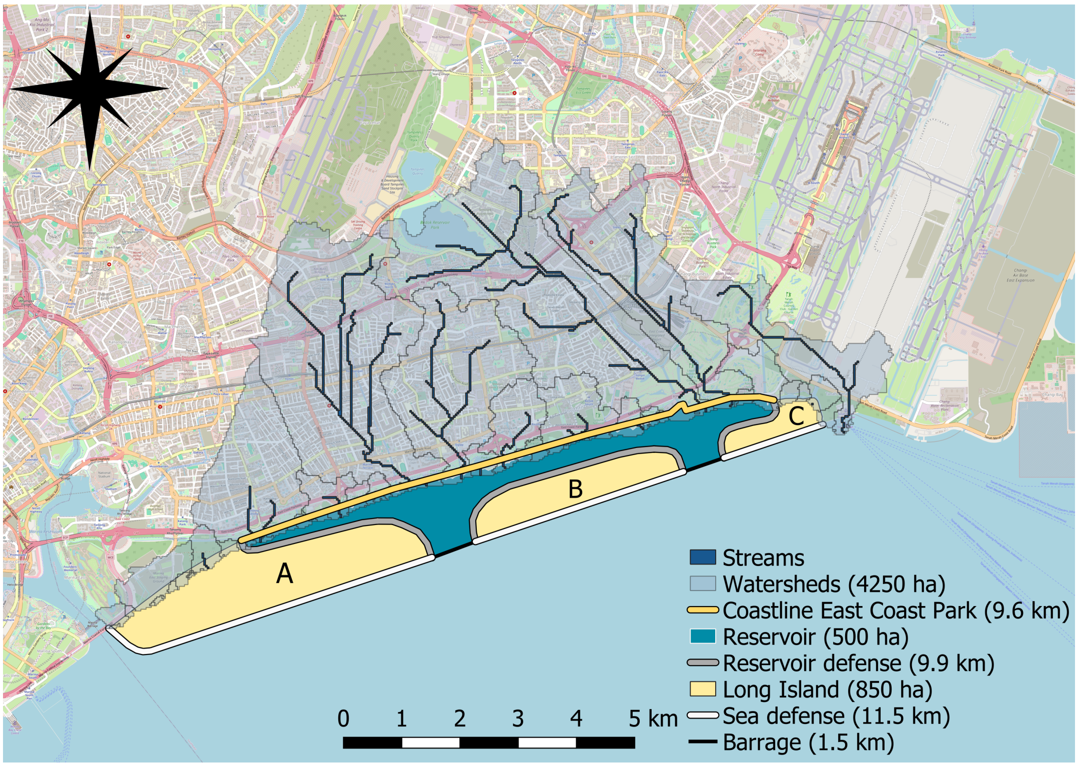


Figure 2.3: Top view of Long Island, indicating key elements and their associated dimensions.

Island	Area [ha]	Length reservoir defense [km]	Length sea defense [km]
A	530	3.7	5.9
B	235	4.6	3.8
C	85	1.6	1.8
Total	850	9.9	11.5

Table 2.2: Dimensions of the islands.

2.1.5. Bathymetry and soil properties

The depths of the sea bottom at Long Island are based on the publicly available bathymetry map of Garmin (Garmin, n.d.). In Figure 2.4, the bathymetry at Long Island is shown, with depths in meters Chart Datum. In this thesis, all levels are configured in m SHD. The difference between m CD and m SHD is: 0 m CD = - 1.652 m SHD. The depths vary locally and reach up to -20 m SHD along the southern boundary of Long Island.

The subsoil at the location of Long Island is characterized by the Kallang Formation, which consists of soft deposits such as marine clay (Sharma et al., 2000). These soft soils can result in geotechnical challenges during the construction and lifetime of Long Island, such as excessive consolidation or instability of sea defenses. Further information on the local soil properties is yet unknown, but geotechnical research performed for the Changi Airport reclamation project can be used as a first estimation due to its close proximity. For this project, extensive research was conducted into the soil properties, settlement estimations, and soil improvement methods (Bo et al., 2018).

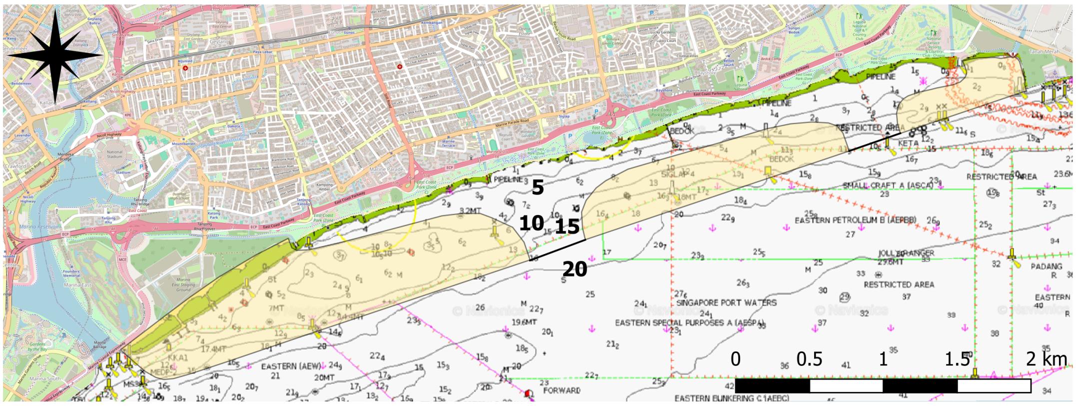


Figure 2.4: Bathymetry of Long Island given in mCD, showing contour lines per 5 meters (Garmin, n.d.).

2.1.6. Environmental impacts Long Island

The Long Island project causes environmental damage in sand-exporting SEA countries, increases greenhouse gas emissions, and affects Singapore's coastal ecosystem.

The sand required for the construction of Long Island is imported from SEA nations, and these sand imports are associated with environmental impacts. The sand mining process can have significant consequences on water quality and can lead to coastal erosion and destabilization of riverbanks (Jaya et al., 2011). In Singapore, most sand is imported from Southeast Asia, where the sand market is characterized by a lack of transparency and regulation (Global Witness, 2010), which has led to illegal activities within the industry (Harriss-White & Michelutti, 2019). The environmental impacts of illegal sand mining are often even more severe than those associated with legal operations (Filho et al., 2021). Additionally, the long transport distances between borrow sites and the project location contribute substantially to greenhouse gas emissions. For instance, sailing distances of 47–260 km accounted for 37–55% of total emissions during dredging operations in Indonesia (Slamet et al., 2020). In Singapore's case, similar distances apply to sources such as Sumatra and Malaysia, but emissions increase significantly when sourcing from more distant SEA nations.

The construction of Long Island will divide the current coastline from the sea, impacting the natural environment and the related ecological value of East Coast Park. This disconnection will eliminate natural flows, seawater, waves, and tides, potentially jeopardizing the survival of flora and fauna, which are sensitive to such changes. For instance, the critically endangered Hawksbill turtles nest at East Coast Park (Tan, 2023). These turtles require sandy shorelines for egg-laying and return to the place where they hatched. The altered coastline may prevent the turtles from finding suitable nesting sites or may no longer provide the necessary conditions for egg-laying.

2.2. Function analysis

Based on the layout of Long Island and the area analysis, a function analysis can be performed. This analysis gains insight into the main functions that Long Island should provide. There are three types of functions: principal, preserving, and additional functions. Principal functions originate from the motivation to create Long Island. The preserving functions are functions that come from Long Island interfering with other systems. Additional functions are possibilities and opportunities that the development of Long Island provides.

Principal functions:

- Provide flood protection along the East Coast of Singapore, consisting of Long Island itself, East Coast Park, and its hinterland.
- Provide space for urban expansion.
- Increase the freshwater supply.

Preserving functions:

- Reduce saltwater intrusion into the freshwater reservoir.
- (Re)create a nature-friendly coastline.
- (Re)create a recreational coastline.

2.3. Process analysis

In the process analysis, the focus mainly lies on the hydrological processes. The platform level influences Long Island's hydrological processes significantly. Therefore, these processes are illustrated and described per reclamation type, being a landfill reclamation and a polder reclamation. Additionally, reference projects are provided to offer real-life examples.

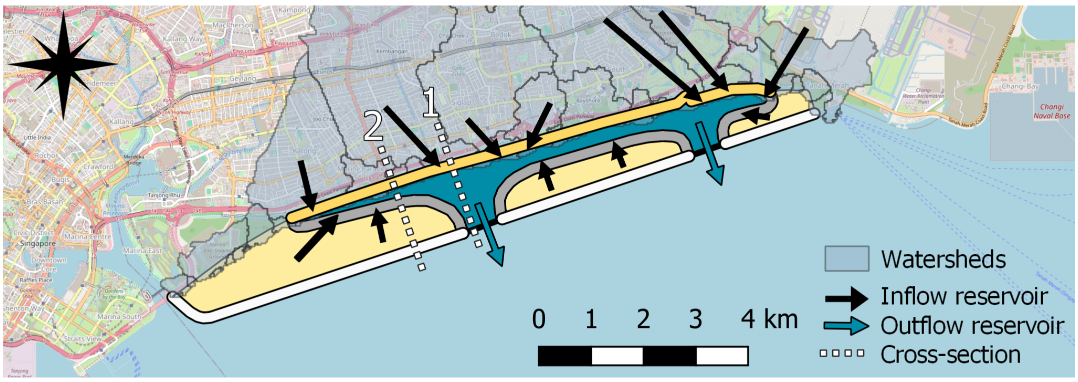


Figure 2.5: Top view of Long Island showing inflow from watersheds and the island into the reservoir, outflow via barrages, and cross-section locations.

Reservoir

In Figure 2.6, the other water processes at the reservoir are illustrated. The rainfall falling on top of the reservoir is again indicated as the rainfall depth D. There are also river streams draining into the reservoir ($Q_{streams}$), which are also illustrated in the top view of Long Island (Figure 2.3). The freshwater flowing from Long Island into the reservoir (Q_{LI}) is also indicated. All the water collected into the reservoir flows out via the barrages ($Q_{barrage}$), which can be done using pumps or by opening gates.

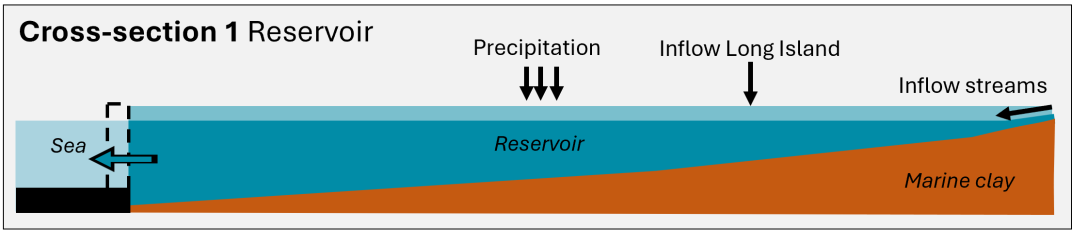


Figure 2.6: Cross-section 1: Main water processes of the Long Island reservoir.

Landfill reclamation

Often, a land reclamation is completely filled so that the platform level is well above sea level. This reclamation type is referred to as a landfill reclamation and results in a relatively simple water balance, which is illustrated in Figure 2.7. Rainfall that falls on the land reclamation is collected in sloped drainage canals to allow for gravitational flow. The canals drain out into the reservoir. While it is possible to drain water into the sea, it is assumed that the freshwater collected on Long Island is stored in the reservoir, allowing it to be reused for drinking water purposes.

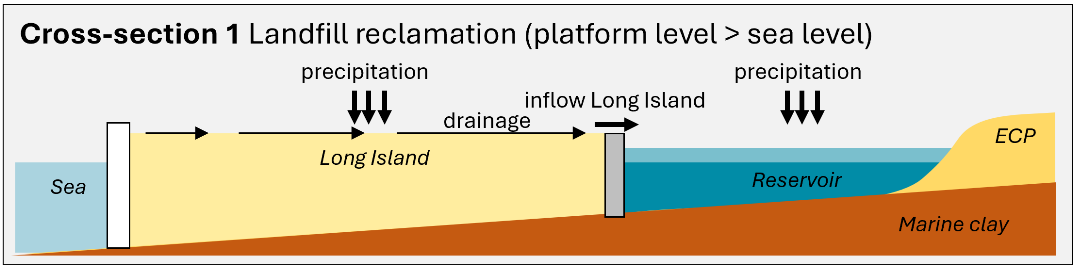


Figure 2.7: Cross-section 2: Long Island as landfill reclamation, indicating the main water processes. Sloped drainage canals result in gravitational flow towards the reservoir.

A reference project comparable with landfill reclamations is the Eko Atlantic project situated in Lagos, Nigeria (see Figure 2.8). This project facilitates urban expansion and flood protection, and the reclaimed area will be about 1,000 hectares (Royal HaskoningDHV, 2025). The area is a high wave energy environment, so the flood defenses are a breakwater-type structure.

Figure 2.8: Eko Atlantic project in Lagos, Nigeria. Left: breakwater as flood defense. Right: aerial view of reclamation. Images: (Eko Atlantic, 2025).

The second landfill project is an example from Singapore: the Tuas Port project (see Figure 2.9). In this case, the perimeter of the reclamation is constructed with caissons, which function as a quay wall. The vertical caisson structure performs well in low-wave climates, such as in port areas. Phase 1 consists of 294 hectares of reclaimed land and the fabrication and installation of 221 caissons (Maritime and Port Authority of Singapore, 2025). Each caisson measures 28 meters in height, 28 meters in width, and 40 meters in length (Maritime and Port Authority of Singapore, 2019). The platform level is at +5 m SHD (Maritime and Port Authority of Singapore, 2023), resulting in water depths of about -23 m SHD, which is comparable with the depths at the location of Long Island's sea defense.

Figure 2.9: Tuas Port, Singapore. Left: towing of a caisson. Right: outline of caissons. Images: (Royal Boskalis, 2025).

The Maasvlakte 2 project in the Port of Rotterdam, Netherlands, is another large-scale landfill reclamation (see Figure 2.10). This port expansion added approximately 2,000 hectares of land, of which 1,000 hectares are designated for company sites and terminals (Port of Rotterdam Authority, 2021). The first phase, constructed between 2008 and 2014, involved the placement of 240 million m^3 of sand. This phase cost €1.55 billion (Port of Rotterdam Authority, 2021), equivalent to around 2.9 billion SGD or 12 SGD/m^3 after inflation adjustment. The terminals are built on two platform levels:

- NAP +5.00 m: the terminals constructed in the initial phase, consisting of container handling. NAP is equal to MSL.
- NAP +6.00 m: second terminal area, which is still under development. This elevation was chosen in response to updated insights into sea level rise and flood risk, in combination with the proposed accommodation of high-risk industrial sites (Port of Rotterdam Authority, 2022).

Coastal protection includes a detached breakwater of reused concrete blocks and a pebble beach on the northwest side (see left image in Figure 2.10), while the remaining shoreline is protected by dunes and sandy beaches.

Figure 2.10: Maasvlakte 2. Left: Coastal defense on northwestern side (PUMA: Boskalis & Van Oord, 2025). Right: Aerial photo of the south side (Cornelissen, 2020).

Polder reclamation

Other options are to lower the platform level and reinforce the defense line along the reclaimed land, to protect the low-lying area from flooding. This land reclamation system can be seen as a polder reclamation, where the platform level is below sea level.

The water processes found in a polder system are illustrated in Figure 2.11. Again, the rainfall collected on the island is defined as the rainfall depth D. Again, the sloped drains are constructed to allow for gravitational flow, but now pumps are required to drain the water into the reservoir, this pumping discharge is indicated as $Q_{LI,pump}$. Seepage flow is the flow of water through the pores of soil and occurs when there is a difference in pressure (Sivakugan, 2005). As the polder has a platform level below sea level and a reservoir water level, water tends to flow into the polder. The seepage flow from the seaside is defined as $Q_{s,sea}$. The reservoir side is defined as $Q_{s,reservoir}$.

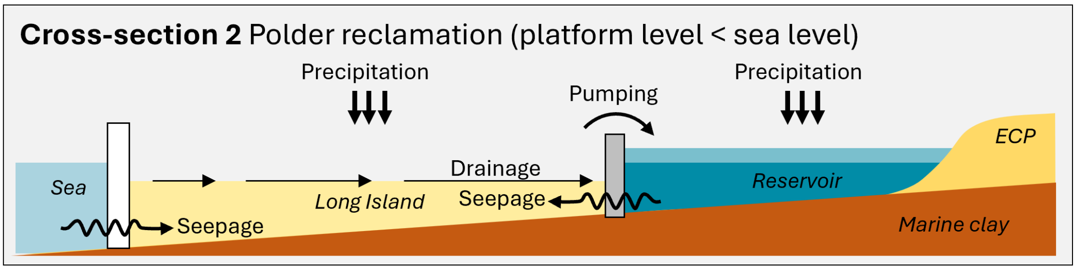


Figure 2.11: Cross-section 2: Long Island as polder reclamation, indicating the main water processes. Sloped drainage canals result in gravitational flow towards the reservoir, from where it is pumped over the reservoir defense.

In Figure 2.12 and 2.13, two examples of polder reclamations are given. The Flevopolder is the largest land reclamation in the world and is located in the Netherlands. The total area is 97,000 hectares and mainly has an agricultural function (Stichting Polderpioniers, 2025). Dikes are used for flood protection.

Figure 2.12: Flevo polder, the Netherlands. Left: dike structure with grass-covered crest. Right: aerial view of agricultural land.

Another example is Pulau Tekong, a polder reclamation located in Singapore. This island has a military function and has an area of 810 hectares (Royal HaskoningDHV, 2016). In this case, the flood defenses also consist of a dike structure.

Figure 2.13: Pulau Tekong, Singapore. Left: construction of dike (Boskalis, 2023). Right: aerial view of project (The Straits Times, 2022).

3
Basis of the design

This chapter outlines the basis for the conceptual design of Long Island by defining the key design requirements, evaluation criteria, and boundary conditions. Section 3.1 outlines the program of requirements, including flood protection standards and functional objectives. Section 3.2 introduces the evaluation criteria used in the Multi-Criteria Analysis. The design lifetime of infrastructure is discussed in Section 3.3, followed by the sea defense adaptation pathways in Section 5.2. Section 3.5 explains the cost analysis methodology, while Sections 3.6 to 3.8 detail the cost components and site-specific boundary conditions.

3.1. Program of requirements

The program of requirements outlines the essential criteria that the design must fulfill. The requirements are based on flood protection standards set by the PUB and insights from the function analysis.

1. Provide flood protection along the East Coast of Singapore, consisting of Long Island itself, East Coast Park, and its hinterland.

    The protection standard in Singapore is based on platform level requirements (PUB, 2018). In this thesis, the flood protection standard is formulated differently: the sea defenses should be designed to withstand hydraulic loads with a return period RP is 10,000 years. For the polder system, an additional requirement is set: the pumping system should have sufficient capacity for a rain event with a return period RP of 50 years with a duration t of 4 hours (PUB, 2018). In Section 3.1.1, this is further explained.

2. Provide urban expansion.

    The effective land area of Long Island should be 850 hectares.

3. Increase the freshwater supply.

    The reservoir area should be 400 hectares.

4. Minimize saline water intrusion into the freshwater reservoir.

    Salt intrusion is defined as the movement of saline water into freshwater aquifers. Salt intrusion should be prevented as it can result in a decline of the water quality of Long Island's reservoir. The amount of salt intrusion that occurs under certain conditions is not within the scope of this research. But measures against salt intrusion are applied: for polder systems, a seepage screen is required which penetrates the subsoil at least 3 meters.

3.1.1. Considerations flood protection

There are various types of flooding and related failure mechanisms that can occur at Long Island. These are summarized in a fault tree in Figure 3.1. The flooding type and failure mechanisms considered are indicated in blue. In the paragraphs below, further explanation is given per flooding type.

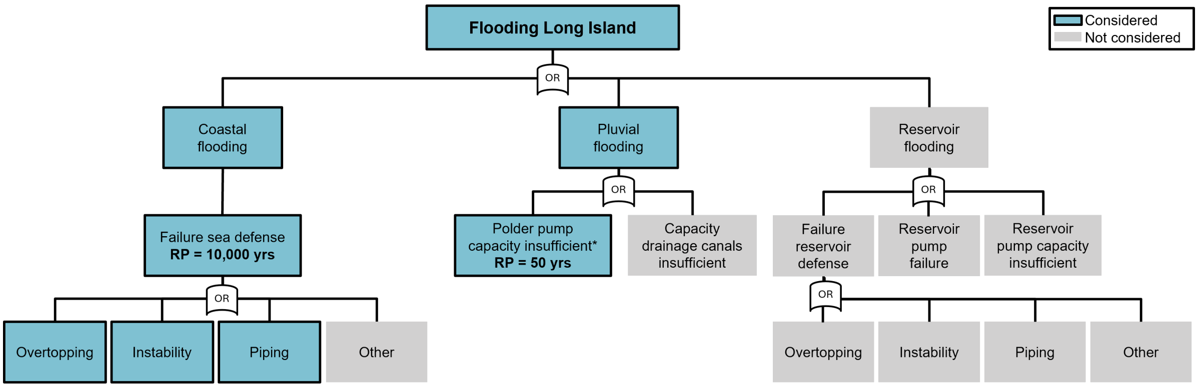


Figure 3.1: Fault tree Long Island. The considered flooding types and failure mechanisms are indicated in blue.

Coastal flooding

One of the most critical flooding scenarios involves the failure of the sea defense, which can result in large-scale flooding. Several mechanisms could lead to sea defense failure. Overtopping, instability and piping are considered in the design and further described in Chapter 4. For overtopping, a general requirement is formulated.

Overtopping refers to the phenomenon where waves exceed the height of the structure, potentially damaging the crest or inner slope. The design approach adopted in this thesis follows the EurOtop II Manual (Van der Meer et al., 2018), with a focus on the mean overtopping discharge. The corresponding design criterion, based on a well-maintained grass-covered crest and inner slope, is defined in Equation 3.1.

$$q_{mean} = 5 \cdot 10^{-3} \; m^3/s/m \tag{3.1}$$

Other potential failure mechanisms, such as deliberate attacks using explosives to breach the sea defense or accidental collisions with ships, are not considered.

Pluvial flooding

For pluvial flooding, the focus lies on flooding associated with the pumping system, which is only applicable for the polder systems. The pumping system requirements are split up into three aspects:

1. Minimal pumping capacity: The system should be capable of handling the average rainfall intensity associated with a return period of $RP = 50$ years (see Section 3.8.6 for details). Additional seepage volumes should also be considered, if applicable.
2. Full redundancy: Every critical component in the pumping system must have a backup to ensure continued operation in case of failure (PUB, 2018).
3. Storage capacity: To accommodate peak rainfall, the system should provide storage volume equal to at least half of the total rainfall from the design event.

Note: The design approach presented in this thesis does not strictly follow the PUB Code of Practice for the first and third requirements (pumping and storage capacity). However, the adopted design methodology has been reviewed and validated in consultation with Haskoning.

Pluvial flooding related to the capacity of the drainage canals is not considered, as this does not differ when varying the platform level, and is therefore not relevant for this research.

Reservoir flooding

Thirdly, flooding of the island can occur due to failure of the reservoir system, which can occur for several reasons: the reservoir defense or the pump of the reservoir may fail, or the pump may lack sufficient capacity. Flooding related to the reservoir is not considered in this thesis, but the maximum allowable reservoir water level is estimated, which is used to determine the crest height of the reservoir dike.

3.2. Program of evaluation criteria

Evaluation criteria are used for the comparison of design options, which is done with a Multi Criteria Analysis. The evaluation criteria are based on the problem analysis (Section 1.2) and system analysis (Chapter 2), and are listed below:

1. Minimal required resources
2. Adaptability sea defense
3. Integration into surroundings
4. Nature-friendly, recreational coastline
5. Residual flood risk

3.3. Design lifetime of infrastructure

It is assumed that all infrastructure of Long Island will be completed by 2040, marking the beginning of the design lifetime of each structure. The design lifetime depends on the type of infrastructure and is defined as follows:

- Flood defenses: More than 110 years.
- Civil works of the pumping station: 100 years, after which large-scale maintenance and repair is needed.
- Mechanical installation of the pumping station: 25 years, after which replacement is required.
- Electrical installations of the pumping station: 10 years, after which replacement is required.

3.4. Pathways: sea defense adaptation

To address projected sea level rise at Long Island, three adaptation strategies for the sea defense are evaluated. These strategies are designed to accommodate up to 5 meters of SLR and are detailed in Table 3.1, supported with a visualization in Figure 3.2. The three pathways differ in timing and scale of construction:

- Pathway A follows a step-wise, incremental approach.
- Pathway B uses a two-step upgrade.
- Pathway C constructs defenses immediately for the full 5-meter SLR.

Incremental build of the sea defense, as in Pathway A, may reduce upfront costs by delaying investments, benefiting from the decreasing value of money over time. However, each construction phase results in start-up costs. Therefore, the pathway strategy is evaluated based on Present Value calculations.

SLR [m]	Pathway A: incremental build	Pathway B: 2-step build	Pathway C: straight to 5 m SLR
0.00	1.15m SLR design	3.00m SLR design	5.00m SLR design
1.15	2.00m SLR design		
2.00	3.00m SLR design	5.00m SLR design	
3.00	4.00m SLR design		
4.00	5.00m SLR design		
5.00			

Table 3.1: Definition of adaptation pathways until 5 meters sea level rise.

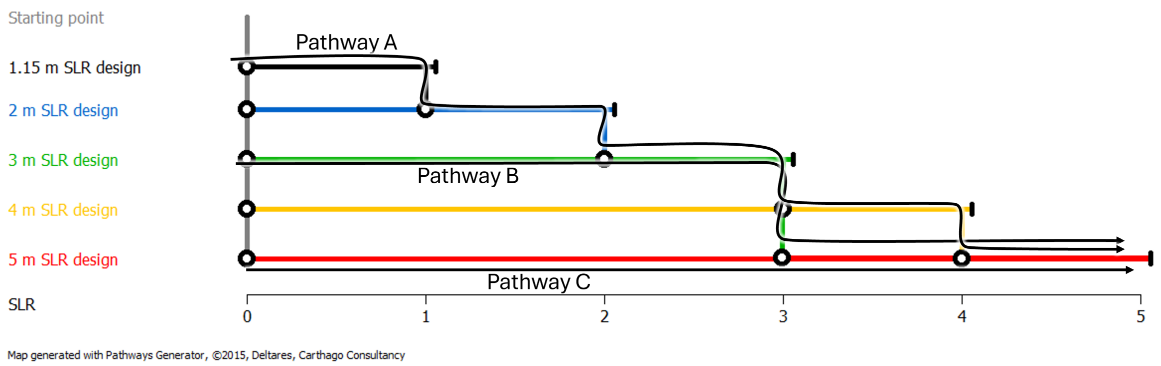


Figure 3.2: Visualization of adaptation pathways (Deltares, 2017).

3.5. Approach cost analyses: evaluation period and discount rate

Two cost analyses are performed to evaluate the land reclamation variants and related sea defense adaptation strategies:

- Initial Capital Expenditures: This evaluation focuses on the Capital Expenditures (CAPEX) associated with the construction of Long Island. No time-based evaluation is involved, so the evaluation period is not applicable.
- Present Value analysis: This evaluation focuses on the total costs over time, accounting for the changing value of money. The Present Value represents the current worth of future costs, calculated using a discount rate. It includes both CAPEX and operating expenditures (OPEX) over a 110-year period, from the base year 2040 to 2150.

The Present Value is calculated using:

$$PV_{t_0} = \frac{C_t}{(1 + SDR)^{t - t_0}} \tag{3.2}$$

Where:

PV_{t_0}	=	present value at base year t_0 [SGD]
C_t	=	future costs at time t [SGD]
SDR	=	Social Discount Rate [-]
t	=	future year [years]
t_0	=	base year (2040) [years]

The social discount rate is used to account for the changing value of money over time. For the pathway evaluation, an SDR of 2% is used. This rate is based on research by Drupp et al., who surveyed over 200 experts on long-term discounting and found that more than 75% of the respondents supported an SDR of 2% (Drupp et al., 2015). The SDR can vary significantly between countries and may fluctuate over time, depending on factors such as economic conditions. Therefore, a sensitivity analysis is conducted to evaluate the impact of different SDR values on the pathway assessment. Specifically, SDR values of 2% and 0.5% are analyzed.

3.6. Costs

The cost components relevant for the Long Island development are categorized as follows:

- Land reclamation: Costs related to construction and maintenance of the land reclamation, including the flood defenses (see Section 3.6.1).
- Polder pumping station: Costs related to construction of the pumping station and its associated operations, maintenance, and replacement of elements (see Section 3.6.2).

- Project development: Costs related to planning and mobilization of the Long Island project and future projects due to adaptations.(see Section 3.6.3).

Each cost category is further divided into:

- Capital Expenditures (CAPEX): One-time investments for construction, project development, or infrastructure replacement.
- Operational Expenditures (OPEX): Recurring annual costs for operations and maintenance.

3.6.1. Land reclamation

The CAPEX of the land reclamation, including the flood defenses, is based on material costs, using market prices from June 2024 as reported by the Building and Construction Authority of Singapore (Townsend, 2024). These prices reflect only the purchase cost of materials; construction and placement costs are excluded. To estimate total project costs, a multiplication factor of 4 is applied to most materials, except for sand and sheet piles, where installation costs are relatively low. For cement bentonite, where no public pricing data is available, unit rates are estimated based on the Pulau Tekong project in Singapore and adjusted for inflation. These unit rates were validated by experts from Haskoning and TU Delft and are presented in Table 3.2.

Comparable Material	Market Price [SGD/m^3]	Actual Material	Factor	Unit Rate [SGD/m^3]
Aggregate	55	Rock	4	220
Concreting sand	45	Sand	1	45
Concreting sand	45	Sand key	4	180
Ready-mixed concrete and steel	430	Reinforced concrete	4	1720
Steel	6330	Sheet pile	1	6330
—	—	Cement bentonite	—	600

Table 3.2: Unit rates for construction materials based on June 2024 market prices (Townsend, 2024).

The OPEX involves the maintenance of the sea defenses, which is defined as a percentage of the CAPEX of the construction of the sea defense. The maintenance costs vary depending on the sea defense type:

- OPEX dike: 2% of the CAPEX annually.
- OPEX caisson: 1% of the CAPEX annually.

Maintenance of other elements, such as the reservoir dike, is not considered, as it does not affect the comparison between land reclamation variants. Moreover, the design lifetime of the flood defenses is larger than the considered evaluation period, so replacement of elements of the flood defenses is not considered in the cost analysis.

3.6.2. Pumping station

The CAPEX of the pumping system is derived from the Permanent Canal Closures and Pumps (PCCP) project in New Orleans, USA, which was completed in 2017. The total cost of the project was 615 million USD (Flood Protection Authority, 2018). After adjusting for inflation and currency exchange rates, this equates to approximately 1.05 billion SGD. With a total pumping capacity of 24,300 cubic feet per second (688 m^3/s), the unit cost of pumping capacity, C_p is 1.5 million SGD/m^3/s. This number is verified with Haskoning and is similar to unit cost estimates for large pumps in the Netherlands.

$$C_p = \frac{1.05 \text{ billion SGD}}{688 \text{ m}^3/\text{s}} \approx 1.5 \text{ million SGD/m}^3/\text{s}$$

The replacement of the mechanical and electrical installations is needed every 25 and 10 years, respectively. The replacement costs are based on the initial CAPEX of the pumping system. The values are listed in Table 3.3.

Pumping system element	Replacement frequency	Replacement costs
Civil works	100 years	50% of CAPEX
Mechanical installation	25 years	35% of CAPEX
Electrical installation	10 years	15% of CAPEX

Table 3.3: Replacement costs of the pumping system.

The OPEX of the pumping system is estimated at 5% of the CAPEX annually, based on typical operation costs (Smith & Loveless, 2018). This estimate is assumed to include labor expenses for system operation, electricity, and annual maintenance checks.

3.6.3. Project development

The project development costs apply to both the initial construction of Long Island and any future adaptations of the sea defense system. The capital expenditures for project development includes:

- Additional costs: These cover engineering and contingency expenses, estimated at 20% of the material costs (Schoemaker et al., 2025).
- Mobilization costs: These include startup activities such as consultancy, tendering, and permitting, and are assumed to be 20% of the combined material and additional costs (Schoemaker et al., 2025).

3.7. Platform levels

Three polder systems and three landfill systems are considered, being listed in Table 3.4. The platform level of the landfill systems are based on possible platform level requirements of the PUB.

Platform level z_{LI} [m SHD]	Description
-4	Polder, below MSL and reservoir water level.
-2	Polder, below MSL and reservoir water level.
1.5	Polder, just above MSL until 2 m SLR.
4	Landfill, standard platform level for reclamation projects in Singapore.
4.5	Landfill, current platform level requirement specific for Long Island.
5.1	Landfill, elevated platform level currently under discussion for Long Island.

Table 3.4: Considered platform levels of Long Island.

3.8. Boundary conditions

In this section, the site-specific parameters are listed that will be used to develop the conceptual design of Long Island. In Figure 3.3, an overview is given of the boundary conditions, with the parameter description found in Table 3.5.

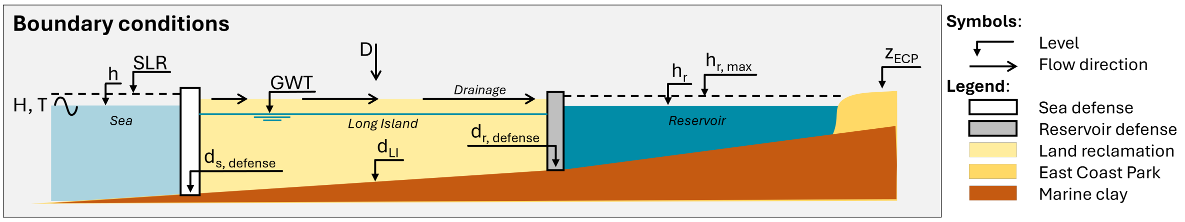


Figure 3.3: Schematization of Long Island, showing a cross-section and indicating the boundary conditions used for the design of Long Island.

Parameter	Description	Unit
$d_{s,defense}$	Bathymetry at sea defense	[m SHD]
$d_{r,defense}$	Bathymetry at reservoir defense	[m SHD]
d_{LI}	Bathymetry at Long Island	[m SHD]
h	Sea water level	[m SHD]
SLR	Sea level rise	[m]
H	Wave height	[m]
T	Wave period	[s]
D	Rainfall depth	[mm]
Z_{ECP}	Elevation East Coast Park	[m SHD]
h_r	Target water level reservoir	[m SHD]
$h_{r,max}$	Maximum water level reservoir	[m SHD]
GWT	Ground Water Table	[m SHD]

Table 3.5: Parameter description of the boundary conditions.

3.8.1. Bathymetry

The depths of the sea bottom at Long Island d_{LI} determine the required volume of sand for the land reclamation. The depths are estimated by splitting the island up into smaller parts with comparable depths. This can be seen in Figure 3.4. Per island part, the area is determined, and the depth is estimated. The resulting numbers are given in Table 3.6. The depths at the reservoir defense ($d_{r,defense}$) and sea defense ($d_{s,defense}$) are also determined and are given in Table 3.7.

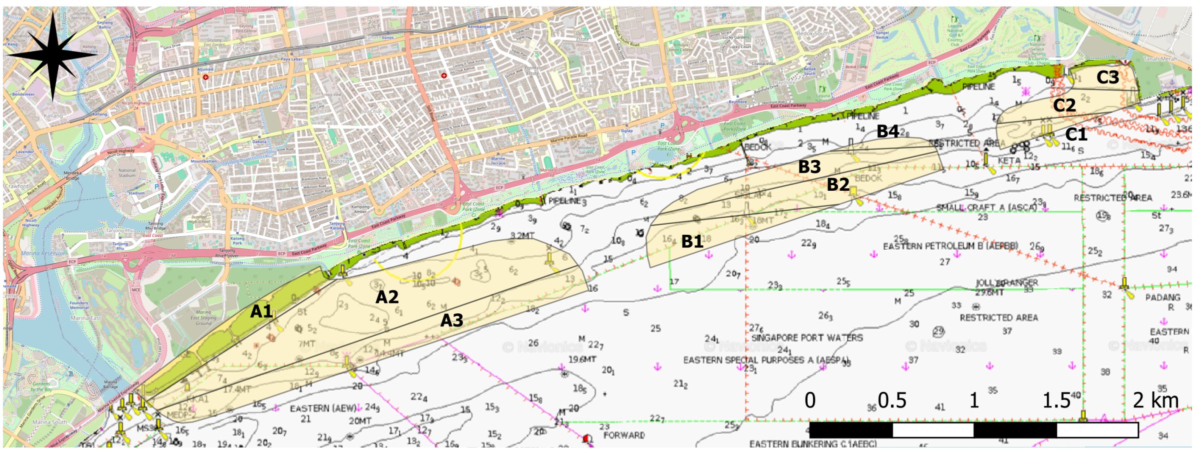


Figure 3.4: Depth of sea bottom d_{LI} given in m CD (0 mCD = -1.652 m SHD).

Island part	A1	A2	A3	B1	B2	B3	B4	C1	C2	C3
Area [ha]	77	297	159	57	96	72	8	20	43	22
Depth [m SHD]	-1.6	-7.6	-13.6	-18.6	-14.6	-8.6	-3.6	-7.6	-3.6	-4.6

Table 3.6: Sectioning of Long Island for volume calculation. Depth of sea bottom per part of the island d_{LI} given in m SHD (see Figure 3.4).

Island	A	B	C
Depth at reservoir defense [m SHD]	-7	-11	-5
Depth at sea defense [m SHD]	-20	-16	-8

Table 3.7: Depth of sea bottom at the location of the reservoir defenses $d_{r,defense}$ and sea defenses $d_{s,defense}$ per island, given in m SHD.

3.8.2. Sea water levels

The extreme water levels h at Long Island are based on the MSc Thesis of Trommelen, which used a Gumbel distribution to describe extreme water levels at Tanjong Pagar, a tidal gauge at the East Coast of Singapore (Trommelen, 2022). The resulting extreme water levels are summarized in Table 3.8. These extreme water levels consider tidal variation, seiches and surges, but exclude sea level rise (Trommelen, 2022).

Return period RP [yrs]	Sea water level h [m SHD]
1	1.55
10	1.84
100	2.00
1,000	2.17
10,000	2.33

Table 3.8: Design sea water level h per return period in m SHD, based on Gumbel-distribution (Trommelen, 2022).

3.8.3. Sea level rise

Sea level rise projections form a critical basis for the concept design and the evaluation of the pathway strategies. Therefore, three SLR scenarios are considered, as detailed in Table 3.9.

The concept design incorporates a sea level rise of 1.15 meters, based on the assumption that Long Island will be completed by 2040 and that sea levels will have already risen to some extent by then. For the evaluation of the pathway strategies, the SSP5-8.5 83rd percentile scenario is used as the base case, with values derived from (Meteorological Service Singapore, 2020). A sensitivity analysis is also conducted, considering a faster SLR scenario (SSP5-8.5, 95th percentile) and a slower SLR scenario (SSP2-4.5, 83rd percentile).

Since current IPCC projections extend to 2150, an extrapolation is performed to extend the projections to 2250. Figure 3.5 presents the SLR projections and extrapolations, including the key SLR points used in the design of Long Island. These values are summarized in Table 3.9.

	*SSP5-8.5, 83rd percentile	**SSP5-8.5, 95th percentile	**SSP2-4.5, 83rd percentile
SLR [m]	Year	Year	Year
0.00	2025	2025	2025
1.15	2100	2090	2125
2.00	2150	2125	2185
3.00	2185	2165	2240
4.00	2220	2195	
5.00	2250	2220	

Table 3.9: Projected years of reaching specific SLR thresholds under different climate scenarios. *Scenario used as base case. **Scenarios used in sensitivity analysis.

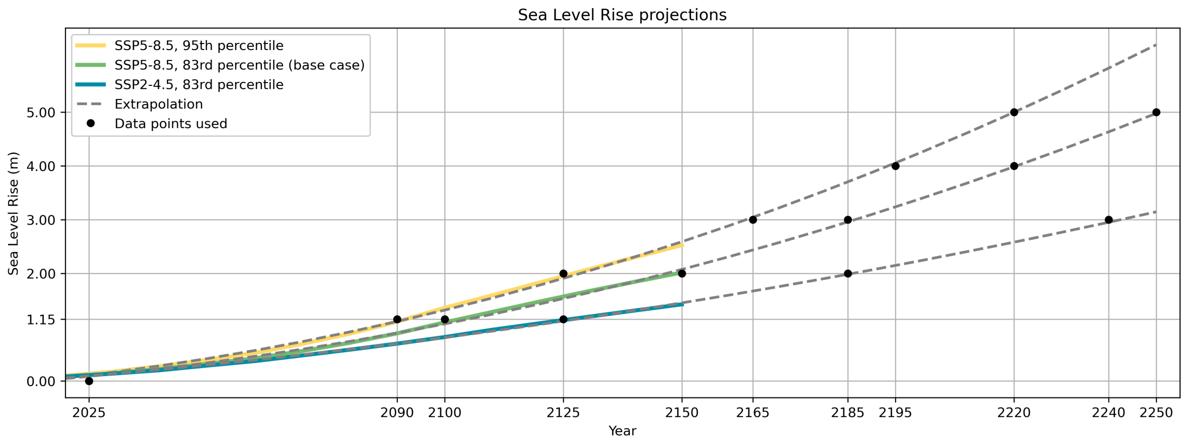


Figure 3.5: SLR projections under different climate scenarios, indicating the extrapolation to 2250 and the resulting data points used.

3.8.4. Wave conditions

The wave characteristics are estimated using wind data, as there are no public regional data available on the wave climate in Singapore. The resulting wave characteristics are summarized in Table 3.11. The critical wind orientation is shown in Figure 3.6, which is based on a comparison performed in Appendix B.

The design wind speeds that should be used are the hourly mean at 10 meters above the ground, because this continuous flow of wind creates waves. So, wind gusts do not have a large influence on wave formation. The resulting design wind speeds are retrieved from weather station Semakau (Copernicus Climate Change Service (C3S), 2017), which are summarized in Table 3.10.

Return period RP [yrs]	Wind speed U_w [m/s]
1	10.7
10	12.8
100	14.9
1,000	16.8
10,000	18.7

Table 3.10: The hourly mean wind speed 10 m above ground at Semakau Station (Copernicus Climate Change Service (C3S), 2017).

The wave characteristics are determined using the Bretschneider formulation (US Army Corps of Engineers, 1984). In this formulation, the parameters used are the fetch, the depth and the average wind speed. The longer the fetch and faster the wind speeds, the higher the waves. The longest fetch at Long Island is 65 kilometers, with a Southwest orientation (see Figure 3.6). The depth is averaged to 30 meters, based on the public bathymetry map (Garmin, n.d.).

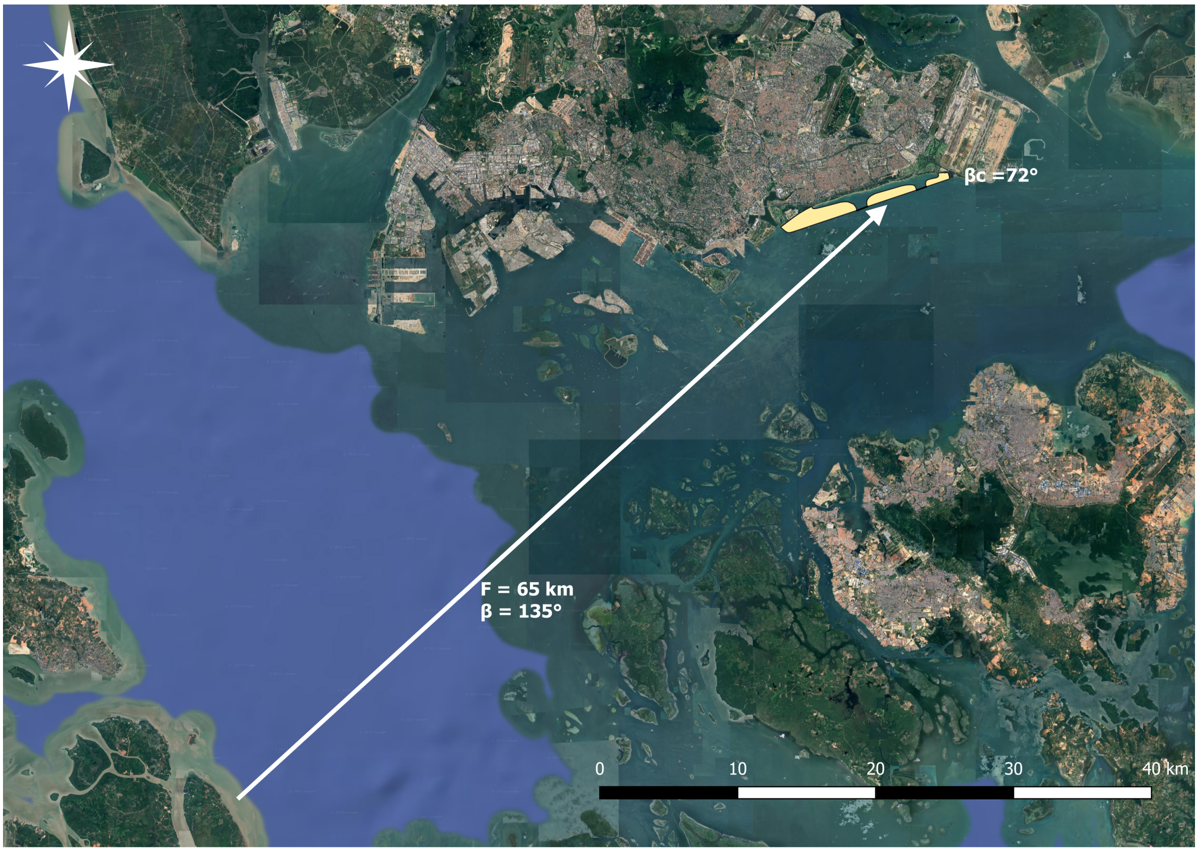


Figure 3.6: The longest fetch results in the highest wave conditions at Long Island. The fetch is 65 kilometers and has an angle of incidence of $\beta = 135° - 72° = 63°$ with respect to the coastline.

The Bretschneider wave heights are based on shallow water or deep water. In this case, the relative wave conditions are small in comparison with the water depth, therefore deep water conditions should be applied. Since the fetch is in the order of 10th kilometers, the fetch is limited. The fetch-limited deep-water wave forecasting equations are presented in equation 3.3, 3.5 and 3.4.

$$H_{m0} = 5.112 \cdot 10^{-4} \cdot U_w \cdot F^{1/2} \tag{3.3}$$

$$t = 3.215 \cdot 10^{1} \cdot (F^2/U_w)^{1/3} \tag{3.4}$$

$$T_m = 6.238 \cdot 10^{-2} \cdot (U_w \cdot F)^{1/3} \tag{3.5}$$

Where:

H_{m0}	=	mean wave height [m]
t	=	duration of wind condition [s]
T_m	=	mean period [s]
U_w	=	hourly mean wind speed at 10 meters above ground [m/s]
F	=	fetch [m]

For some design elements the peak period (T_p) and mean energy period ($T_{m-1,0}$) are used instead of the mean period. The peak period is determined by the ratio of T_m over T_p being equal to a constant between 0.79 and 0.96. The factor 0.79 results in the largest peak period, and is therefore taken as the constant (see equation 3.6). The mean energy period is determined by the energy density spectrum (see Equation 3.7). For deep-water waves, two spectra are widely used: the Pierson-Moskowitz (PM) spectrum and the JONSWAP-spectrum. The PM spectrum is applicable for a fully developed sea, but as the fetch is relatively short, the JONSWAP-spectrum is a better match, which describes a growing sea-state (CIRIA & CETMEF, 2007).

$$T_p = T_m/0.79 \tag{3.6}$$

$$T_{m-1,0} = T_p/1.1 \tag{3.7}$$

Where:

T_p = peak period [s]

$T_{m-1,0}$ = mean energy period [s]

The resulting wave conditions per return period RP, based on Southwest winds, are listed in Table 3.11.

Return period RP [yrs]	Wave height H_{m0} [m] (3.3)	Wind duration t [hrs] (3.4)	Wave period T_m [s] (3.5)	Mean energy period $T_{m-1,0}$ [s]	Peak period T_p [s]
1	1.39	6.55	5.53	6.36	7.00
10	1.67	6.17	5.87	6.75	7.43
100	1.94	5.87	6.17	7.10	7.81
1,000	2.19	5.64	6.42	7.39	8.13
10,000	2.44	5.44	6.66	7.66	8.43

Table 3.11: Wave conditions based on the return period of occurring wind conditions (US Army Corps of Engineers, 1984), angle of incidence is $\beta = 63°$.

3.8.5. Design cases

In Singapore, extreme water levels and wave heights do not occur simultaneously, resulting in the development of specific design cases (see Table 3.12). The design cases are based on the direction of prolonged winds, which influence either wave heights or water levels:

1. Southwest monsoon: Winds from the southwest generate the largest waves. A 1/10,000-year wave height is combined with a 1/1-year water level.
2. Northwest monsoon: Winds from the northeast result in the highest water levels. A 1/1-year wave height is combined with a 1/10,000-year water level.

Additionally, SLR must also be considered in extreme water levels. But in some cases, the addition of SLR would lead to a less critical design scenario, such as the required rock diameter for the toe of a dike. Therefore, the addition or exclusion of SLR should also be integrated into the design cases.

	Wave				Water level	
Case	RP [yrs]	H_{m0} [m]	t [hrs]	T_m [s]	RP [yrs]	$h + SLR$ [m SHD]
SW: largest wave	10,000	2.44	5.44	6.66	1	1.55
SW: largest wave + SLR	10,000	2.44	5.44	6.66	1	2.70
NE: largest water level	1	1.39	6.55	5.53	10,000	2.33
NE: largest water level + SLR	1	1.39	6.55	5.53	10,000	3.48

Table 3.12: Design cases for the concept design ($SLR = 1.15m$). For the pathway design SLR will be changed accordingly.

3.8.6. Rainfall depth

Rainfall determines the pumping capacity required for the polder reclamation. The design rainfall event has a return period RP of 50 years with a duration t of 4 hours and is based on the Code of Practice on Surface Water Drainage (PUB, 2018). The present design rainfall event would be 165 mm in 4 hours. But due to climate change, the intensity of rainfall events will increase over time (Meteorological Service Singapore, 2023). For the year 2100, the increase of rainfall depth is assumed to be +25%. The resulting design rainfall event D is 207 mm and is summarized in Table 3.13.

	Rainfall event		
Case	RP [yrs]	D [mm]	t [hrs]
Year 2100	50	207	4

Table 3.13: Design rainfall event considering +25% increase due to climate change (PUB, 2018).

3.8.7. Elevation East Coast Park

The elevation of East Coast Park z_{ECP} is used to determine the maximum water level in the reservoir $h_{r,max}$, which in turn will determine the reservoir crest height. In Figure 1.1, it can be seen that East Coast Park is lower than 5 m above MSL (so +5 m SHD). In this thesis, it is assumed that the elevation of East Coast Park is equal to +2 m SHD.

3.8.8. Reservoir water levels

Based on the elevation of East Coast Park z_{ECP}, the maximum allowable water level in the reservoir $h_{r,max}$ is assumed to be +1.5 m SHD, providing a safety buffer of 0.5 m. The target water level in the reservoir h_r is assumed to be at MSL, which is 0 m SHD. Therefore, the reservoir can accommodate a water level increase of up to 1.5 m.

3.8.9. Ground water table

The groundwater table GWT at Long Island influences seepage processes, the loading on flood defenses and settlement processes. The groundwater table is dependent on the platform level and is defined in Table 3.14.

Platform level z_{LI} [m SHD]	Ground water tableGWT [m SHD]
-4	-5
-2	-3
1.5	0.5
4	2.5
4.5	3
5.1	3.6

Table 3.14: Groundwater table at Long Island per platform level in m SHD.

3.8.10. Material density

In Table 3.15, the density of the construction materials is listed.

Material	Density ρ $[kg/m^3]$
Fresh water	1000
Salt water	1025
Unsaturated sand	1900
Saturated sand	2100
Rock	2650
Concrete	2500

Table 3.15: Density of materials $[kg/m^3]$.

4 Concept design

In this Chapter, the conceptual design of Long Island's land reclamation is presented. First, the design approach is explained in Section 4.1. Next, the sea defense design is developed. The dike and caisson design is found in Section 4.2 and 4.3, respectively. Lastly, the additional land reclamation components are presented in Section 4.4, which consist of the reservoir defense design, the effective land area correction, the polder pumping system, and settlement corrections.

4.1. Design approach

The goal of the conceptual design approach is to enable a comprehensive and comparative evaluation of different reclamation variants. The focus is on the design of sea defenses, their adaptation to sea level rise, and the elements that change significantly with different platform levels. The conceptual design is developed in five steps:

1. Sea defense design and adaptation to SLR: First, the sea defense of Long Island is designed. Two flood defense types are considered: a dike and a caisson to explore the trade-offs between material use, adaptability, and integration with the urban environment. Although the public favors a more nature-friendly coastline, nature-based solutions are excluded to maintain focus on the platform-level optimizations and climate robustness.
2. Reservoir defense design: A dike-type structure is used for the reservoir defense. The reservoir defense design is included because its crest height is influenced by the platform level. But due to its smaller scale and shallower depth compared to the sea defense, less emphasis is placed on its design.
3. Correction for effective land area: The platform level affects the effective land area due to the inner slopes of the flood defenses and the need for water storage in polder systems. This reduction is compensated by expanding the surface area through a seaward shift of the sea defense.
4. Polder pumping system: At lower platform levels, a pumping system is required to discharge the collected water into the reservoir. The volume of water depends on Long Island's surface area, which is influenced by the platform level. First-order estimates of pumping capacity and required storage volumes are made.
5. Correction for settlement: In the final step, settlement of the land reclamation is considered, as the process is dependent on the soil pressures, which are determined by the platform level. Due to the presence of soft marine clay, settlement can differ significantly between platform levels.

Together, these five components define the sea defense design and its adaptation potential, while also capturing the influence of the platform level on Long Island's design. This enables a detailed comparison of the different reclamation variants.

4.2. Sea defense design: dike

In this Section, the dike option as a sea defense is worked out. First, the general outline of the dike is presented, from which failure mechanisms and structural criteria are explained. Based on that the structural design is developed and visualized in an overview.

4.2.1. Outline of design

In Figure 4.1, the general outline of the dike design is given, indicating the several elements considered, which are further described below.

Core Inner part of the dike, which is often made out of layers of sand and clay. In some cases, also quarry run is used. In this thesis, it is assumed that the whole core is constructed of sand.

Bentonite wall In case of the lowest platform levels (-4 m SHD and -2 m SHD), a cement bentonite wall is constructed. This reduces saltwater intrusion.

Armor protection The armor material protects the dike from wave attack and often consists of rock, concrete cubes, basalt, or asphalt. In this design, the armor protection consists of an armor layer and under layer. Underneath, a geotextile is placed to prevent sand particles from being conveyed through the pores of the armor protection.

Toe The waves that hit a sea defense will reflect partially, depending on the slope of the sea defense and its roughness. The reflected wave can result in scour in front of the outer slope. The scour could result in instability of the slope, and when progression occurs, the dike can collapse. To prevent this from occurring, a toe is placed at the bottom of the outer slope. The toe is constructed of rock.

Sand key The subsoil consists mainly of marine clay, a soft material with a low bearing capacity. Therefore, a foundation underneath the dike is required. In this case, a sand key is opted for with its geometry based on the design of Pulau Tekong.
Elements such as berms or submerged detached breakwaters are not considered, because these elements only prove to be functional with a relatively shallow foreshore with relatively large wave heights. In the case of Long Island, this is not applicable.

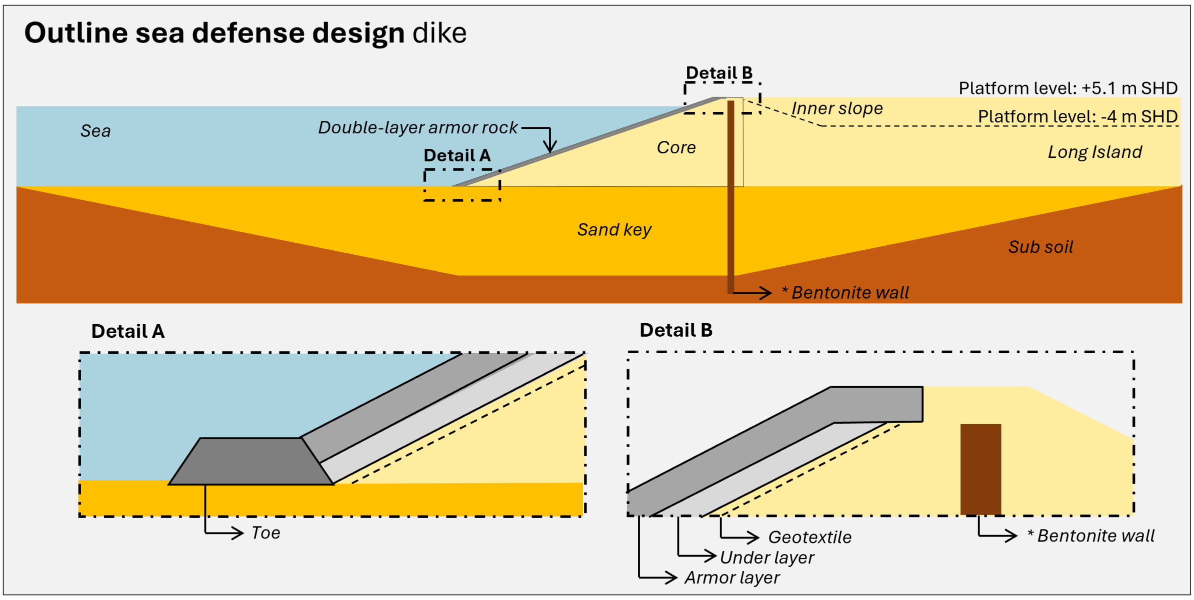


Figure 4.1: General outline of sea defense design for Long Island. *Bentonite wall is only applicable for polder reclamations.

4.2.2. Structural requirements

In this section, the structural requirements are defined, which are based on failure mechanisms. In Figure 4.2, common failure mechanisms of the dike are shown. Most failure mechanisms shown are considered in this design, except for shearing, erosion first bank, settlement, drifting ice, and collision.

The piping failure mechanism is not applicable for this dike design. Piping is the process by which groundwater flow gradually erodes loose material from the foundation of dikes, thereby forming shallow pipes at the interface of the loose material and the cohesive cover layer (Robbins, 2015). This groundwater flow occurs when there is a potential difference across the structure. In the case of Long Island, this would be the case when the platform level is below MSL. However, piping is not relevant for this type of dike, as no cohesive cover layer is available due to the core being completely made out of sand.

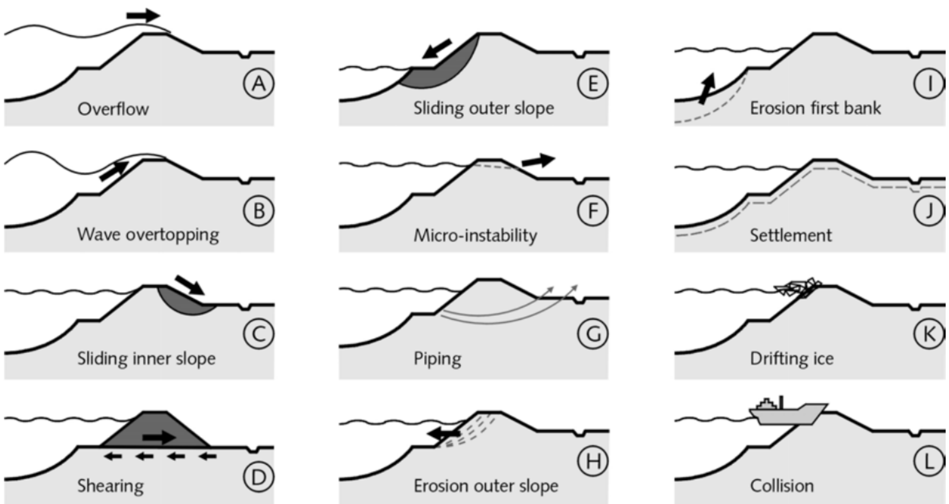


Figure 4.2: Failure mechanisms dike (Technical Advisory Committee on Flood Defence, 1998).

In Table 4.1, the structural requirements of the dike can be found. There are sometimes multiple structural requirements linked to one failure mechanism, because multiple aspects can lead to failure. For example, when the dike fails due to sliding of the outer slope, it could be because the toe of the structure is unstable or because of insufficient permeability in the filter layers.

Failure mechanism	Structural requirement	Equation
Overflow, wave overtopping, sliding inner slope	Limited overtopping	4.1
Erosion outer slope	Armor stability	4.8
Sliding outer slope	Toe stability	4.10
Erosion outer slope, sliding outer slope	Filter stability	4.13, 4.14, 4.15
Sliding inner or outer slope	Macro stability	
Salt intrusion	Salinity prevention	

Table 4.1: Structural requirements of dike based on failure mechanisms.

4.2.3. Calculations structural design

In this section, the structural design of the dike is explained based on the structural requirements.

Limited overtopping

The approach to calculate the overtopping at the sea defense is based on the EurOtop Manual (Van der Meer et al., 2018). Overtopping is in general split up into two types: the mean overtopping discharge q and the maximum wave overtopping volume V_{max}. In this research, only the mean overtopping discharge is considered. Overtopping depends on several parameters such as the geometry of the sea defense, the steepness and depth of the foreshore, and the wave height and its relative steepness. These parameters also determine which overtopping formula should be used.

In Equation 4.2, the required freeboard based on overtopping is formulated (Van der Meer et al., 2018). This overtopping formula is applicable for a relatively deep foreshore and a gentle slope of the sea defense, which is in this case 1 vertical over 3 horizontal ($\cot\alpha = 3$). Several types of reduction factors are used in this formula, the two applicable for Long Island are the roughness reduction factor (γ_f) and the oblique waves reduction factor (γ_β).

$$\frac{q}{\sqrt{gH_{mo}^3}} = \frac{0.026}{\sqrt{tan\alpha}}\gamma_\beta \cdot \xi_{m-1,0} \cdot exp[-(2.5\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v})^{1.3}] \tag{4.1}$$

$$R_c = \frac{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}{2.5}\left[-\ln\left(\frac{q\sqrt{\tan\alpha}}{0.026\sqrt{gH_{mo}^3}\gamma_\beta \cdot \xi_{m-1,0}}\right)\right]^{\frac{1}{1.3}} \tag{4.2}$$

Where:

R_c	=	required freeboard [m]
$\xi_{m-1,0}$	=	Iribarren parameter for mean energy period conditions [-]
H_{m0}	=	mean wave height [m]
γ_b	=	1, berm reduction factor, not considered [-]
γ_f	=	0.55, roughness reduction factor, based on 2 rock layers and impermeable core [-]
γ_v	=	1, wave wall reduction factor, not considered [-]
γ_β	=	$1 - 0.0033 \cdot \beta$ if $0° \leq \beta \leq 80°$, else: $\gamma_\beta = 0.824$ oblique waves reduction factor [-] with β in [° relative to coastline]
q	=	$5 \cdot 10^{-3}$, maximum allowed overtopping discharge [m^3/s]
α	=	outer slope of dike [rad]

With the Iribarren parameter defined as follows:

$$\xi_{m-1,0} = \frac{tan(\alpha)}{\sqrt{2\pi/gH_{m-1,0}/T_{m-1,0}^2}} \tag{4.3}$$

Where:

$H_{m-1,0}$	=	mean energy wave height [m]
$T_{m-1,0}$	=	mean energy period [s]

The resulting crest height is determined by the equation defined in Equation 4.4.

$$z_{crest} = R_c + h + SLR \tag{4.4}$$

The resulting minimal crest height required for the sea dike is $z_{crest} = 6.3$ m SHD.

Armor stability

The stability of armor-rock is determined by using the Van der Meer formula for deep water conditions (van der Meer, 1988) and following its approach using Section 5.2 of the Rock Manual (CIRIA & CETMEF, 2007). There are two types of formulae, depending on the wave breaking type that occurs at the structured slope. The wave breaking type is determined by the ratio of the Iribarren parameter (ξ_m) and the critical value (ξ_{cr}). Both are defined in Equation 4.5 and 4.6, respectively.

$$\xi_m = tan(\alpha)/\sqrt{2\pi/gH_s/T_m^2} \tag{4.5}$$

$$\xi_{cr} = [\frac{c_{pl}}{c_s}P^{0.31}\sqrt{tan(\alpha)}]^{\frac{1}{P+0.5}} \tag{4.6}$$

Where:

ξ_m	=	Iribarren parameter [-]
H_s	=	significant wave height [m], H_{m0} used
T_m	=	mean wave period [s]
ξ_{cr}	=	critical Iribarren parameter [-]
c_{pl}	=	6.2 [-], constant for plunging waves
c_s	=	1.0 [-], constant for surging waves
P	=	0.1, notional permeability [-]

The critical case for armor stability is "SW monsoon: largest wave + SLR". As the slope consists of an armor layer, filter layer and geotextile, the notional permeability is $P = 0.1$ (CIRIA & CETMEF, 2007). This results in $\xi_{cr} > \xi_m$, meaning that there are surging waves at the structure during the design wave conditions.

Next to the type of wave breaking, the angle of incidence of the waves have influence on the stability of armor-rock. Oblique waves result in lower damage levels than perpendicular wave attack (Gent, 2014). The angle of incidence for the 1/10,000-year wave condition is equal to $\beta = 63°$ with respect to Long Island's coastline. The resulting reduction factor for obliquity is determined in Equation 4.7, based on van Gent's research (Gent, 2014). The coefficient c_β depends on the type of armor layer and the wave loading type. For rock slopes, $c_\beta = 0.35$ for long-crested waves and $c_\beta = 0.42$ for short-crested waves. Since the sea-state at Long Island can be more described as wind-waves instead of swell-waves, short-crested waves are taken. The resulting reduction factor is equal to $\gamma_b = 0.54$.

$$\gamma_b = (1 - c_\beta)cos^2(\beta) + c_\beta \tag{4.7}$$

Where:

c_β	=	0.42 [-], coefficient dependent on wavelength and type of armor layer
β	=	63 [° relative to coastline] wave direction

The Van der Meer formula for surging waves is written out in Equation 4.8. The damage level parameter is based on the slope and the amount of damage allowed, which is, in this case, as low as possible, resulting in the level equal to 'start of damage'. This leads to a damage level parameter with the value of $S_d = 2$. The number of incident waves N depends on the duration of the wave condition (t [hrs]) and the mean wave period (T_m [s]). In the Van der Meer formula, wave obliquity is not taken into account, therefore the reduction factor γ_b is added.

$$\frac{H_s}{\Delta D_{n50}} = c_s \cdot P^{-0.13} \cdot (\frac{S_d}{\sqrt{N}})^{0.2} \cdot \sqrt{cot(\alpha)} \cdot \xi_m^P \cdot \gamma_b \tag{4.8}$$

Where:

D_{n50}	=	median diameter armor-rock [m]
S_d	=	2 [-], damage level parameter
N	=	$\frac{3600t}{T_m}$ [-], number of incident waves at the toe
γ_b	=	wave obliquity reduction factor [-]

The minimum needed median diameter for the armor-rock layer is $D_{n50} = 0.65\ m$. When looking at the rock classes, this corresponds with a rock class of HMA1000-3000, having a diameter of $D_{n50} = 0.90$ m and a median mass of $M_{50} = 615\ kg$. The required thickness of the armor layer t_a is calculated using Equation 4.9, based on Section 3.5 of the Rock Manual (CIRIA & CETMEF, 2007). The layer coefficient k_t can vary widely due to the wide range of characteristics of different rock and the placement type. In this case, based on Table 3.10 of the Rock Manual, a single armor layer is assumed and an average of two rock gradings (0.5t and 3-6t) is taken. This results in $k_t = 0.90$. For the armor layer porosity n_v, the same method is used, resulting in $n_v = 37$ %. The armor layer porosity is of importance in calculating the amount of rock needed.

$$t_a = n \cdot k_t \cdot D_{n50} \tag{4.9}$$

Where:

t_a	=	armor layer thickness [m]
n	=	2 [-], number of layers
k_t	=	0.90 [-], layer coefficient
n_v	=	37 [%], armor layer porosity

Toe stability

In the Rock Manual, the toe stability approach is limited to depth-limited cases (CIRIA & CETMEF, 2007). This approach is not applicable for cases with large water depths such as at Long Island, where water depths can reach about 20 meters. Therefore, an approach of van Gent and van der Werf is applied, corresponding to Equation 4.10 and 4.11 (Gent & van der Werf, 2014). The amount of acceptable damage to the toe is defined as the damage number N_{od}. Again, almost no damage is acceptable, resulting in $N_{od} = 0.5$ (CIRIA & CETMEF, 2007).

$$D_{n50} = 0.32 \frac{H_s}{\Delta(N_{OD}))^{1/3}} (\frac{B_t}{H_s})^{0.1} (\frac{t_t}{H_s})^{1/3} (\frac{\hat{u}_\delta}{\sqrt{gH_s}})^{1/3} \tag{4.10}$$

$$\hat{u}_\delta = \frac{\pi H_s}{T_{m-1,0}} \frac{1}{sinh(kh_t)} \text{ with } k = \frac{2\pi}{gT^2_{m-1,0}/2\pi} \tag{4.11}$$

Where:

H_s	=	significant wave height [m], take H_{m0}
N_{OD}	=	0.5 [-] start of damage (CIRIA & CETMEF, 2007)
B_t	=	toe width [m]
t_t	=	toe thickness [m]
$\hat{u}_\delta$	=	characteristic velocity [m/s]
$T_{m-1,0}$	=	mean energy period [s]
k	=	wave number [1/m]
h_t	=	height water level above toe [m]

The stability of the toe depends on the water level and the wave height. Therefore, each design case should be tested to determine the critical load. Additionally, the different nautical depths per island are also considered. Calculating the required rock diameter for the toe is an iterative process, as the toe thickness and toe width are input values that depend on the diameter. The resulting $D_{n50} = 0.17$ m, corresponds with LMA5-40 rock class. But since it can be challenging to place rock of smaller diameter at large depths, a larger rock class is taken. For now, HMA300-1000 rock is used for the toe, which has a diameter of $D_{n50} = 0.59$ m.

Filter stability

Filter stability consists of three criteria: the internal stability, the interface stability and permeability. Each criteria is met and the resulting rock gradings are found in Table 4.2

It is advised to use grading curves of local quarries that will be used for the production of rock at the project (CIRIA & CETMEF, 2007). However, it was not possible to retrieve detailed information about the rock grading at quarries located in South East Asia or the Middle East. Therefore, idealized Rosin-Rammler curves are used, which can be found in Appendix A. Based on these curves, the mass of the rock for a certain passing rate could be found. Using Equation 4.12, the resulting diameters can be found, which are listed in Table 4.2.

$$M = \rho_r \cdot D^3 \tag{4.12}$$

Where:

M = mass [kg]
ρ_r = rock density [kg/m^3]
D = diameter [m]

Layer type	Class	D10 [m]	D15 [m]	D60 [m]	D85 [m]
Armor layer	HMA1000-3000	0.76	0.80	0.95	1.00
Under layer	LMA15-300	0.31	0.33	0.42	0.47
Core	Sand	$0.59 \cdot 10^{-3}$	$0.68 \cdot 10^{-3}$	$1.2 \cdot 10^{-3}$	$1.5 \cdot 10^{-3}$

Table 4.2: Definition of rock grading for armor layer and under layer and sand grading for core. Both are based on idealized Rosin-Rammler curves (see Appendix A.

Internal stability The internal stability requires that the rock classes to be used should be well-graded without gaps. The criteria for internal stability are defined in Equation 4.13.

$$d_{60}/d_{10} < 10 \tag{4.13}$$

Where:

d_{60} = diameter of rock grading with 60% passing rate [m]
d_{10} = diameter of rock grading with 10% passing rate [m]

The internal stability criteria are checked for each armor protection layer:

- Armor layer: $d_{60}/d_{10} = 0.42/0.31 = 1.35 < 10$ ✓
- Under layer: $d_{60}/d_{10} = 0.95/0.76 = 1.25 < 10$ ✓

Interface stability Interface stability is defined as the stability at the interface of two rock classes. The finer material used as an underlayer or core in the armor unit is defined as the base layer. The coarser material on top is defined as the filter layer. When too coarse material is used on top of the base layer, the finer materials can get entrained in the water as the pores in between the coarse material are larger than the finer particles. To ensure that the finer materials are enclosed by the coarse material, a geometrically closed filter should be used. The criteria for a geometrically closed system are defined in Equation 4.14.

$$d_{15f}/d_{85b} < 5 \tag{4.14}$$

Where:

d_{15f} = diameter of filter layer with 15% passing rate [m]
d_{85b} = diameter of base layer with 85% passing rate [m]

The interface stability criteria is checked for each interface:

- Filter is the armor layer, base is the under layer: $d_{15f}/d_{85b} = 0.80/0.47 = 1.70 < 5$ ✓
- Filter is the under layer, base is the core: $d_{15f}/d_{85b} = 0.33/0.0015 = 220 < 5 \rightarrow$ criteria not met.

The stability criteria for the interface of the underlayer and the core is therefore not met; a geotextile should be placed between this interface.

Permeability The permeability criteria is based on the prevention of excess pore pressures building up in the layer (Section 5.4 (CIRIA & CETMEF, 2007)). To prevent this the design criteria in Equation 4.15 is defined.

$$d_{15f}/d_{15b} > 1 \tag{4.15}$$

Where:

d_{15f} = diameter of filter layer with 15% passing rate [m]
d_{15b} = diameter of base layer with 15% passing rate [m]

The permeability criteria is checked for each interface:

- Filter is the armor layer, base is the under layer: $d_{15f}/d_{15b} = 0.80/0.230 = 2.42 > 1$ ✓
- Filter is the under layer, base is the core: $d_{15f}/d_{15b} = 0.33/0.00068 = 485 > 1$ ✓

Macro stability

When a dike is constructed on soft soil, it is possible that a part or the whole dike fails along a slip plane. This is comparable with the sliding of the inner or outer slope in Figure 4.2. To prevent this, the dike should be assessed on macro stability. It is known that the subsoil at Long Island is weak, but further geotechnical information is absent, limiting the assessment on macro-stability. As a first estimate, a foundation is designed that is assumed to ensure macro stability.

A sand key is taken as the foundation type for the sea defense of Long Island, which is based on the foundation design of Pulau Tekong, Singapore. A sand key consists of a large layer of high-quality construction sand on which the sea defense is constructed on. This sand key is considered stable by assuming that the slip plane of the dike goes through the sand key, which has a high angle of internal friction. The sand key has a central thickness of 10 meters and a width that extends from the toe of the dike to the crest of the dike. To construct the sand key, the seabed at the location of the proposed dike should first be dredged. The side slopes of the sand key are designed with a gradient of 1 vertical to 5 horizontal.

Salinity prevention

Salt intrusion is the process of saline groundwater flow into a freshwater aquifer. This occurs when there is a potential difference across the structure, which is assumed to happen when the platform level is lower than MSL. As the salt water intrusion is a slow process, mean sea level should be taken instead of extreme water levels that last for a relatively short period of time. SLR should be considered to ensure long-term performance. Based on this assumption, the saltwater intrusion is only applicable for the reclamations with a platform level of -4 m SHD and -2 m SHD. To reduce salt intrusion, a seepage screen is installed.

The design of the seepage screen is based on the Pulau Tekong project in Singapore, where a wall of cement bentonite is constructed. Cement bentonite is a low-permeability material bentonite and will work as a barrier when constructed with sufficient length so that it penetrates through a soil layer of low permeability. It is assumed that the end of the bentonite wall should be 3 meters into the subsoil, which is a clayey, low-permeability material. The thickness of the cement bentonite wall is assumed to be 0.80 m, following the dimensions of Pulau Tekong's design. The top of the bentonite wall is 0.5 underneath the crest of the dike. The resulting lengths of the bentonite wall are summarized in Table 4.3.

Platform level z_{LI} [m SHD]	Island A	Island B	Island C
-4	39	35	27
-2	39	35	27

Table 4.3: Length of bentonite wall L_s for dike in meters.

4.2.4. Structural design overview

In Figure 4.3, the dike design is visualized. For this illustration, Island A is considered, which is the most eastern island and has the deepest bathymetry (-20 m SHD). For the other islands, a different nautical depth is assumed, which results in different depths of the bottom of the dike, the sand key, and the end of the bentonite wall. For Island B, the nautical depth is -16 m SHD. The sand key reaches -26 m SHD, and the bentonite wall until -29 m SHD. For Island C, the nautical depth is assumed to be -8 m SHD, resulting in the sand key reaching until -18 m SHD and the bentonite wall until -20 m SHD.

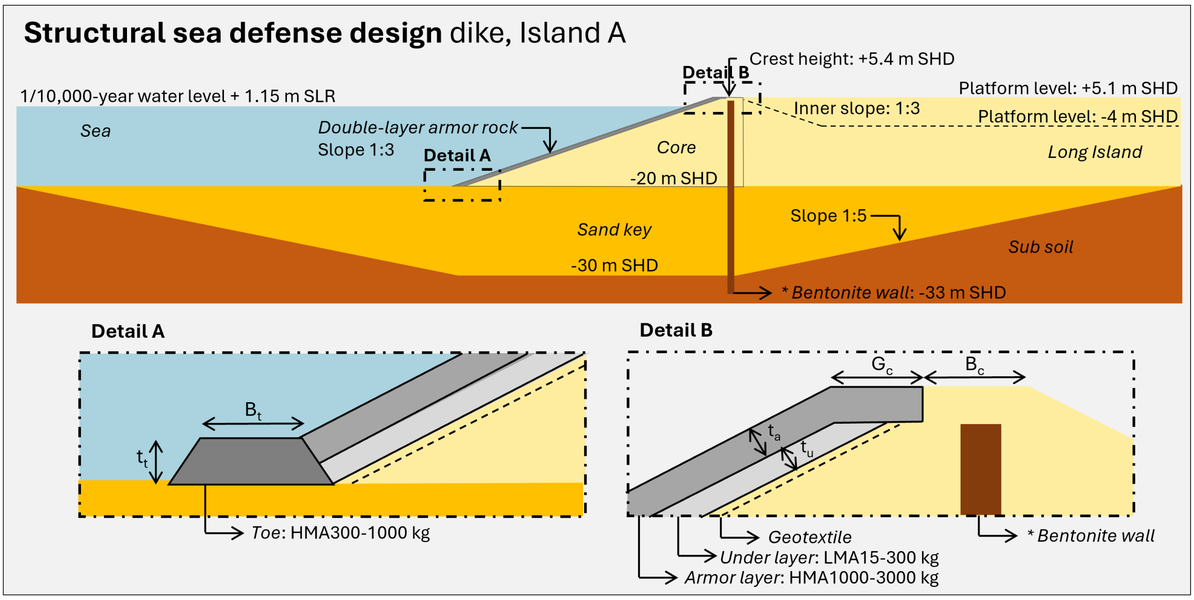


Figure 4.3: Structural dike design Island A. *Bentonite wall only applicable for polder reclamations.

4.2.5. Adaptation and robust design choices

As sea levels rise, the dike must be reinforced to ensure flood protection. Figure 4.4 illustrates the reinforcement of the dike up to 5 meters of SLR, with the corresponding material requirements listed in Table 4.4.

A key robust design choice is ensuring that the dike's seaward slope can be extended during future adaptations. This requires sufficient subsoil strength to maintain slope stability, which can be achieved either by extending the sand key foundation during initial construction or by applying soil improvement techniques at the time of adaptation. In this design, the latter approach is assumed, with the area requiring soil improvement illustrated in Figure 4.4. The associated materials and costs are excluded from this thesis due to their dependency on unknown soil characteristics.

When mean sea level exceeds the platform elevation, a bentonite wall must be installed to prevent saltwater intrusion. For the two lowest platform levels (−4 and −2 m SHD), this wall is already included in the concept design. For higher platform levels, the wall should be constructed when the following SLR thresholds are reached:

- z_{LI} = 1.5 m SHD: install the bentonite wall when SLR = 1.15 m, at the latest.
- z_{LI} = 4 m SHD: install the bentonite wall when SLR = 4 m, at the latest.
- z_{LI} = 4.5 m SHD: install the bentonite wall when SLR = 5 m, at the latest.
- z_{LI} = 5.1 m SHD: install the bentonite wall when SLR = 5 m, at the latest.

Alternative adaptation strategies, such as a seawall on the crest or a landside slope extension, are excluded from the design. A seawall would only provide protection for approximately 1 meter of sea level rise, while a landside extension would reduce the available land area.

SLR [m]	z_{crest} [m SHD]	ΔV_{sand} [10^6 m^3]	ΔV_{rock} [10^6 m^3]
0.00	-	-	-
1.15	6.3	-	-
2.00	7.2	3.03	0.06
3.00	8.2	3.30	0.07
4.00	9.2	3.42	0.07
5.00	10.2	3.56	0.07

Table 4.4: Material requirements for incremental dike adaptation up to 5 m of SLR. Note: the bentonite wall is excluded from the table as its installation timing depends on the platform level. The required volume is $\Delta V_{bentonite}$ = $0.3 \cdot 10^6$ m^3.

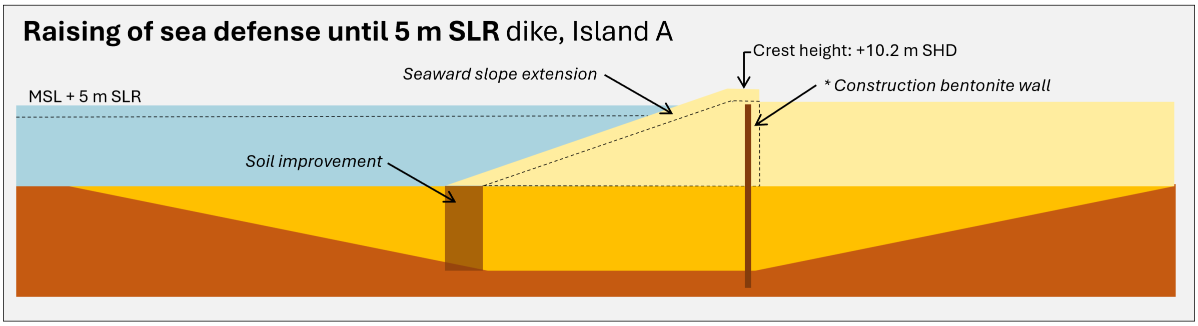


Figure 4.4: Overview of the raising of the dike until 5 m of SLR. Soil improvement is shown illustratively, associated materials and costs are not included. *The construction of the bentonite wall is only applicable for the highest four platform levels (1.5, 4, 4.5, 5.1 m SHD), as the two lowest levels already include a bentonite wall in the initial design.

4.3. Sea defense design: caisson

This section presents the design of the caisson as a sea defense and is structured in the same way as Section 4.2.

4.3.1. Outline of design

In Figure 4.5, the outline of the caisson design is schematized, indicating the several elements considered. Figure 4.6a visualizes the inner structure of the caisson and defines the dimensions. Figure 4.6b shows the concrete structure of a reference project (Tuas Port, Singapore).

Caisson The caisson is a watertight box that is constructed on land and towed to the right location. The length of one caisson is assumed to be $L = 40\ m$, which is the same length as the caissons constructed for the Tuas Port project in Singapore. The sea defense line is 11.5 km, resulting in 288 caissons required. The height H and width B of the caisson differ per island, due to the differences in water depths (see Table 4.6). The bottom slab of the caisson has a thickness of $t_{bottom} = 1\ m$, the outer walls have a thickness of $t_{outerwall} = 0.5\ m$. The caissons are strengthened with inner walls, which results in separate compartments, also known as cells. The cell width and length are $B_{cell} = L_{cell} = 5\ m$ with a thickness of $t_{innerwall} = 0.25\ m$.

Inner slope An inner slope (1V:3H) is added to the caisson design to integrate the sea defense better into its surroundings. This is to make the structure more appealing for residents of Long Island: without the inner slope, the public would have to look at a vertical concrete wall, which could be multiple stories high.

Sheet pile In case of the polder reclamations, a sheet pile wall is along the reservoir side of the caisson. This reduces saltwater intrusion and prevents piping.

Scour protection The scour protection protects the foundation from eroding due to reflected waves.

Sand key The caisson has a sand key foundation, which has the same geometry as for the dike design: the thickness is 10 meters, and the slope is 1:5.

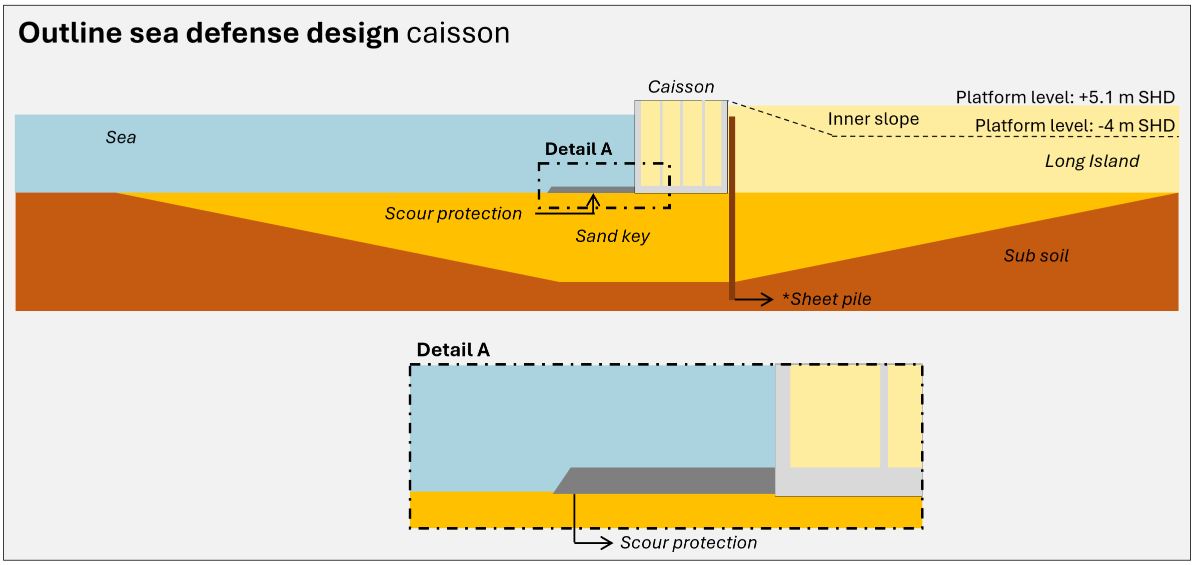
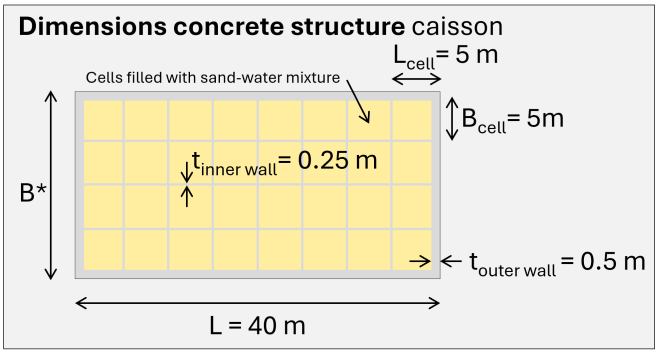
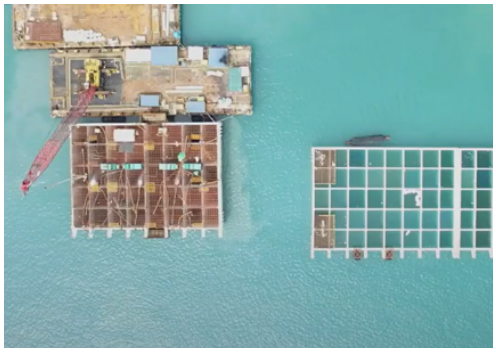


Figure 4.5: General outline of caisson option as sea defense. *Sheet pile is only applicable for polder reclamations.



(a) Top view of the caisson, indicating the dimensions of the concrete structure. *Width B of caisson differs per island.



(b) Caissons placed for the Tuas Port project in Singapore, showing the cell structure (Singapore, 2019).

4.3.2. Structural requirements

The structural requirements of the caisson are based on the relevant failure mechanisms illustrated in Figure 4.7. In Table 4.5, the related design criteria are defined.

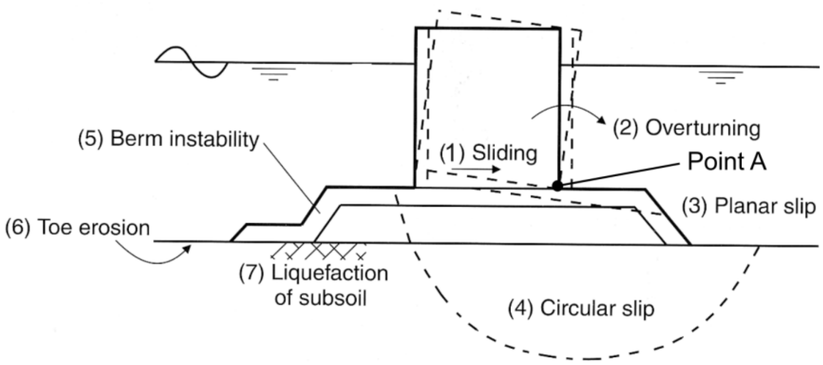


Figure 4.7: Illustration of the failure mechanisms relevant for caisson structures (CIRIA & CETMEF, 2007).

Failure mechanism	Criteria	Equation
Overflow, wave overtopping	Limited overtopping	4.16
Sliding	Shear stability	4.17
Overturning	Rotational stability	4.19
Scour	Scour protection	4.20
Liquefaction subsoil	Vertical bearing capacity	4.21
Circular slip	Macro stability	
Piping	Piping prevention	4.23
Salt intrusion	Salinity prevention	

Table 4.5: Design criteria for the caisson based on failure mechanisms.

For shear stability, rotational stability and vertical bearing capacity, a balance between the forces acting on the caisson needs to be found. These different forces are defined and illustrated in Figure 4.8.

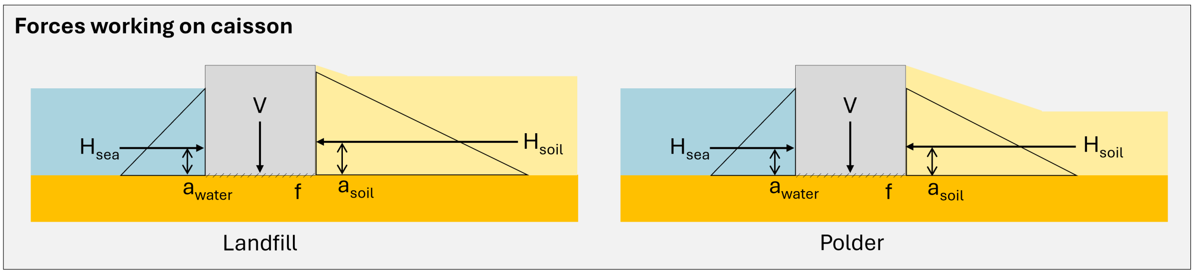


Figure 4.8: Illustration of the forces working on the caisson. Left: landfill reclamation. Right: polder reclamation.

4.3.3. Calculations structural design

Limited overtopping

The freeboard that is required to fulfill the overtopping criteria is based on the EurOtop Manual (Van der Meer et al., 2018). Due to relatively large depths in front of the caisson, no wave breaking occurs, resulting in non-impulsive waves. This results in the application of equation 7.16 of the EurOtop Manual for the calculation of the required freeboard R_c, which is found in Equation 4.16. The resulting crest height is 7.2 m SHD, resulting in the caisson heights H summarized in Table 4.6.

$$R_c = \frac{\gamma_\beta \cdot H_{m0}}{2.78} \cdot \log(\frac{0.05 \cdot \sqrt{g \cdot H_{m0}^3}}{q}) \tag{4.16}$$

Where:

R_c	=	required freeboard [m]
γ_β	=	$1 - 0.0062 \cdot \beta$ if $0 < \beta < 45$, else: $\gamma_\beta = 0.72$. Oblique waves reduction factor [-] with β
	=	in [° relative to the coastline].
q	=	maximum allowed overtopping discharge [m^3/s]

Island	Depth $d_{s,defense}$ [m SHD]	H [m]	B [m]
A	-20	28	15
B	-16	24	10
C	-8	16	10

Table 4.6: Height H and width B of caisson per island.

Shear stability

The criteria is met for each caisson type and each platform level, and can be found in Figure C.1 of Appendix C.

The definition of the shear stability is based on Lecture Notes on Caissons (Voorendt et al., 2016) and is found in Equation 4.17. Shear stability is required to prevent the caisson from sliding, which occurs due to pressure differences between the seaside and landside of a caisson. The friction force between the caisson and the subsoil should be sufficient to overcome these horizontal forces.

$$\sum H < f \sum V \tag{4.17}$$

Where:

$\sum H$	=	total of acting horizontal forces [kN]
f	=	friction coefficient [-]
$\sum V$	=	total of acting vertical forces [kN]

The sum of the horizontal forces $\sum H$ consists of the water forcing on the seaside H_{sea} and the soil forcing on the reclamation side H_{soil}. The platform level determines the loading of the soil, and thus differs per platform level, as indicated in Figure 4.8.

The friction factor f should be based on the lowest friction layer:

- Friction layer caisson-subsoil,
- Internal friction layer subsoil.
- Internal friction layer in deeper soil with a low sliding resistance.

In this case, the possibility that deeper soil layers have a lower sliding resistance is not considered, as also for the caissons, a sand key foundation will be constructed. This means that for several meters of depth, the same sliding resistance is found. The first sliding mechanism, based on the friction layer between the caisson and the subsoil, is critical. The friction factor is calculated with Equation 4.18 and is $f = 0.36$ for sand with an angle of internal friction of $\beta = 30$ °.

$$f = tan(\delta) \approx tan(\frac{2}{3}\varphi) \tag{4.18}$$

Where:

δ	=	friction angle between caisson and subsoil [°]
φ	=	30 [°], angle of internal friction

The sum of the vertical forces $\sum V$ is determined by the total mass of the caisson. The mass of the caisson is calculated based on the volume of concrete and volume of the sand-water mixture. The mass differs per island due to height differences.

Rotational stability

Rotational stability is important to prevent overturning of the caisson. The overturning mechanism is determined by the ratio between the sum of moments over the sum of vertical forces and the geometry of the caisson. The rotational stability criterion is based on the Lecture Notes on Caissons (Voorendt et al., 2016) and is determined by calculating the resulting eccentricity e_R, which is found in Equation 4.19. The rotational stability criterion is checked for each caisson type and platform level. The criteria is met for each case and is found in Figure C.2 of Appendix C.

$$e_R = \frac{\sum M}{\sum V} \leq \frac{1}{6}B \tag{4.19}$$

Where:

e_R	=	resulting eccentricity [m]
$\sum M$	=	total of the moments around center of gravity [kNm]
B	=	width of caisson [m]

Scour protection

Due to the vertical structure of the caisson, incoming waves are reflected and can create scour holes in front of the caisson. Therefore, scour protection should be placed. The dimensions of the protection is determined using equation 4.20, based on the Caissons Lecture Notes (Voorendt et al., 2016).

$$L_{scour} \geq \gamma \cdot n_s \cdot h_{max} \tag{4.20}$$

Where:

γ	=	safety factor [-]
$1 : n_s$	=	average slope of the slide[-], $n_s = 6$ taken for densely packed material.
h_{max}	=	maximum scouring depth [m]

As a first-order estimate, the thickness of the scour protection is assumed to be equal to the incoming wave height ($h_{max} = 2.44\ m$). The resulting length of the scour protection is $L_{scour} = 22\ m$ and is constructed of HMA300-1000 rock ($D_{n50} = 0.59$ m).

Vertical bearing capacity

The subsoil should have sufficient bearing capacity to resist the vertical load of the caisson. The load on the subsoil is calculated using equation 4.21, which is based on the Lecture Notes on Caissons (Voorendt et al., 2016).

$$\sigma = \frac{\sum V}{L \cdot B} + \frac{\sum M}{\frac{1}{6} \cdot L \cdot B^2} \tag{4.21}$$

Where:

σ	=	maximum acting stress on the soil [kN/m^2]
L	=	length of caisson [m]

The caisson is placed on the sand key. As a rule-of-thumb, the bearing capacity of densely packed sand is often assumed to be at least $\sigma_R = 500$ kN/m^2 (Voorendt et al., 2016). Based on this assumption, the vertical load for island A is larger than the bearing capacity. For the other islands, the bearing capacity is sufficient. The bearing capacity can be increased by adding a rock layer on top of the sand key, on which the caisson is placed, or by treating the soil to improve its strength. But for now, it is assumed that the sand key is strong enough to withstand the vertical load. In Figure C.3 in Appendix C the results of the calculations are visualized.

Macro stability

Just like with the dike, macro stability is ensured by constructing a sand key underneath the caisson, the same dimensions are found: the central thickness t is 10 meters, which extends from the toe to the landside of the caisson. The side slopes are 1 vertical over 5 horizontal.

Piping prevention

The piping mechanism can occur at the interface between the concrete bottom of the caisson and the sand key. The piping criteria is checked using the formulation of Lane (see Equation 4.22 and 4.23), which is often used for vertical structures.

$$L \geq \gamma \cdot C_L \cdot \Delta H \tag{4.22}$$

$$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor} \tag{4.23}$$

Where:

C_L	=	Lane constant [-], 5 for coarse sand
ΔH	=	differential head across structure [m]

The differential head ΔH is determined by the water level differences between the sea and the land reclamation. The piping failure mechanism should be checked for a 1/10,000-year water level and should include SLR. The groundwater level at the land reclamation is assumed to be equal to the platform level. The differential head only develops when the water level at sea is higher than the groundwater level, therefore the piping mechanism is only applicable for the two lowest platform levels (-4 and -2 m SHD). For these platform levels, the actual seepage length and the required seepage length are visualized in Figure 4.10. From this Figure, it becomes clear that the piping criteria is not met for the platform levels of -4 and -2 m SHD. For the platform level of 1.5 m SHD, the piping criteria is met.

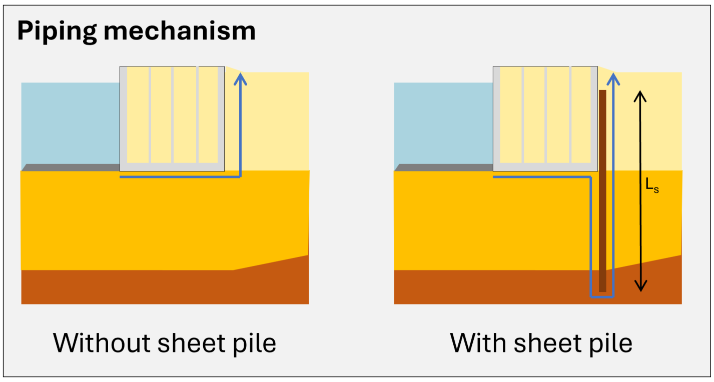


Figure 4.9: Flow path of piping mechanism relevant for the caisson structure. A sufficient length of the seepage screen L_s prevents this from occurring.

The minimal required length of the seepage screen to prevent piping L_s is shown in Figure 4.9 and 4.10, the final values of the seepage screen lengths are given in Table 4.7. In the case of the caisson, the seepage screen is made out of steel sheet piles placed at the landside of the caisson. The construction of a cement bentonite wall is not applicable due to the possibility of damaging the caisson during the digging process of the bentonite wall installation.

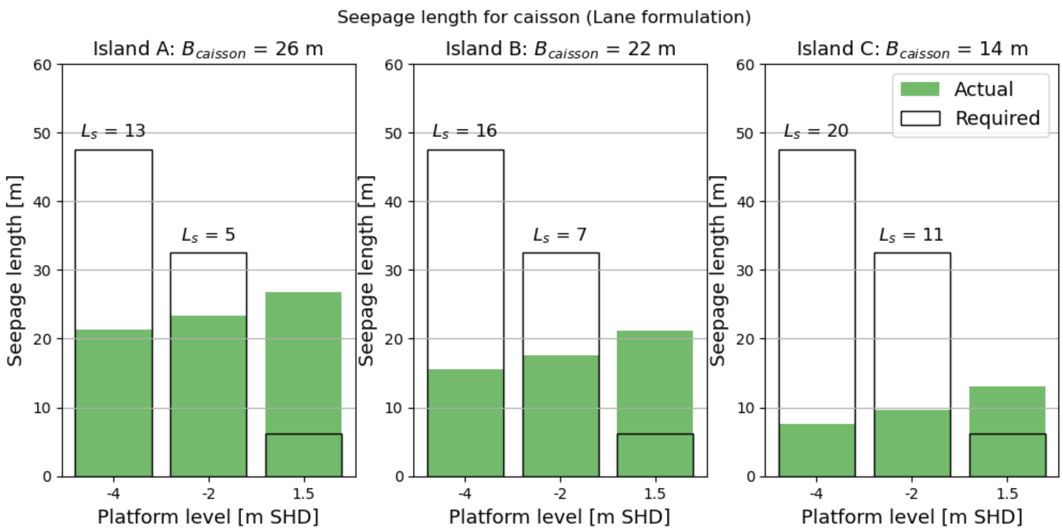


Figure 4.10: Seepage length requirements based on piping for caisson defense design. L_s is the required length of the seepage screen.

Salinity prevention

Next to piping, salt intrusion should also be prevented. Again it is assumed that the end of the sheet pile should reach 3 meters into the subsoil. In combination with the seepage length criteria based on piping, the required length of the seepage screen is summarized in Table 4.7.

Platform level z_{LI} [m SHD]	Island A	Island B	Island C
-4	28	27	23
-2	30	26	18

Table 4.7: Length of seepage screen for caisson L_s in meters.

4.3.4. Structural design overview

In Figure 4.11, a cross-section of the structural design of the caisson is shown. For this, Island A is taken as an example, with its nautical depth of -20 m SHD. For Island B and C, the bathymetry is -16 and -8 m SHD, respectively. The related caisson dimensions are listed in Table 4.6. The sheet pile wall is only present at the two lowest platform levels (-4 and -2 m SHD).

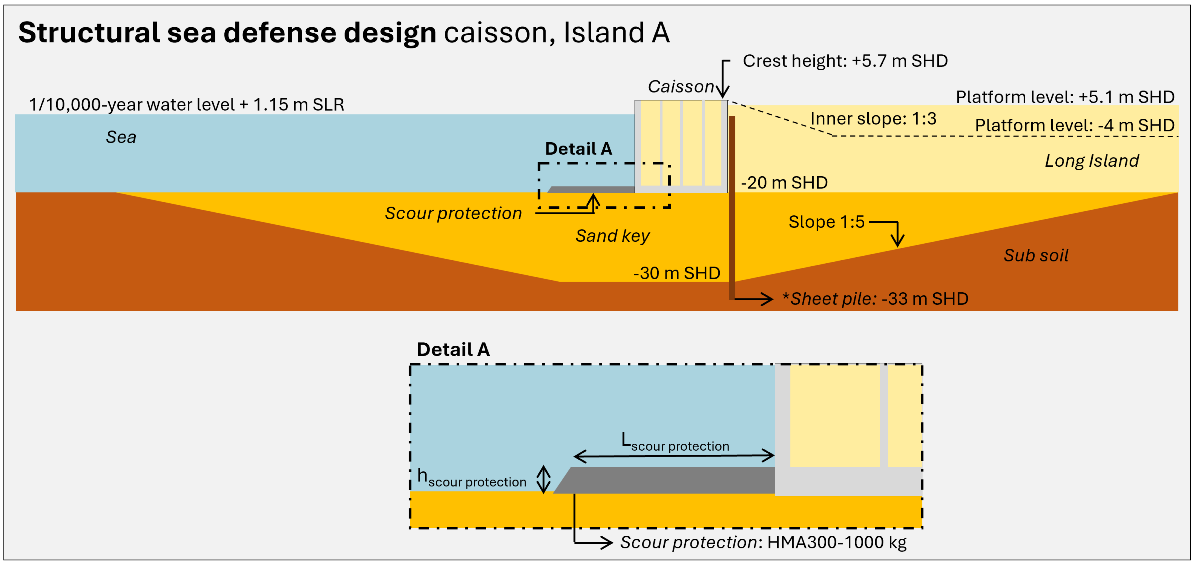


Figure 4.11: Structural caisson design Island A. * Sheet pile only applicable for polder reclamations.

4.3.5. Adaptation and robust design choices

The adaptation strategy for the caisson in response to sea level rise differs from that of the dike, as illustrated in Figure 4.13. The additional sand and concrete volumes required for each adaptation increment are summarized in Table 4.8 and Figure 4.12.

To increase the crest height, modular concrete units are placed on top of the existing caisson. These units must be dimensioned such that a stable structure is developed with watertight joints. To minimize concrete usage, a hollow design filled with sand is applied, using a wall and slab thickness of $t = 0.25$ m. The foundation of the caisson should provide sufficient bearing capacity to withstand the increased vertical loading. The vertical loading increases with $\sigma = 60\ kN/m^2$ for 5 m SLR, which is about 12% of the total vertical bearing capacity assumed for the sand key. For now, it is assumed that the sand key provides sufficient bearing capacity for the increased vertical loading. In future design steps, this aspect requires further research to ensure long-term structural integrity.

The inner slope of the caisson must also be extended to maintain spatial integration. Since the required sand volumes vary by increment and platform level, they are excluded from Table 4.8 and instead visualized in Figure 4.12. Space for these future extensions should be reserved during the development of Long Island, which is further explained in Section 4.4.2.

Similar to the dike adaptation strategy, seepage control becomes necessary when the mean sea level exceeds the platform level. In the caisson design, this is achieved using sheet pile seepage screens. For the two lowest platform levels (-2 m and -4 m SHD), these screens are already integrated into the concept design. For higher levels, the same criteria as for the dike option apply.

SLR [m]	z_{crest} [m SHD]	ΔV_{sand} [10^3 m^3]	$\Delta V_{concrete}$ [10^3 m^3]
0.00	-	-	-
1.15	7.2	-	-
2.00	8.2	120	25
3.00	9.2	85	20
4.00	10.2	52	14
5.00	11.2	17	8

Table 4.8: Material requirements for incremental dike adaptation up to 5 m of SLR. Note: the inner slope extension is excluded from the table due to its dependence on the platform level (see Figure 4.12).

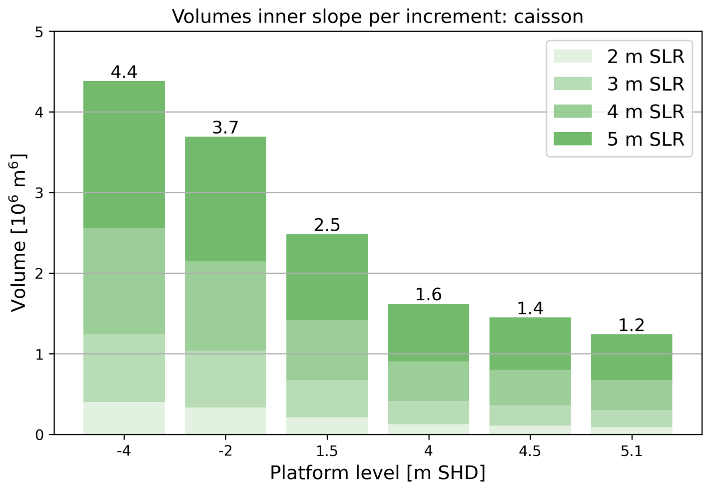


Figure 4.12: Additional volumes for the inner slope extension per SLR step for each platform level.

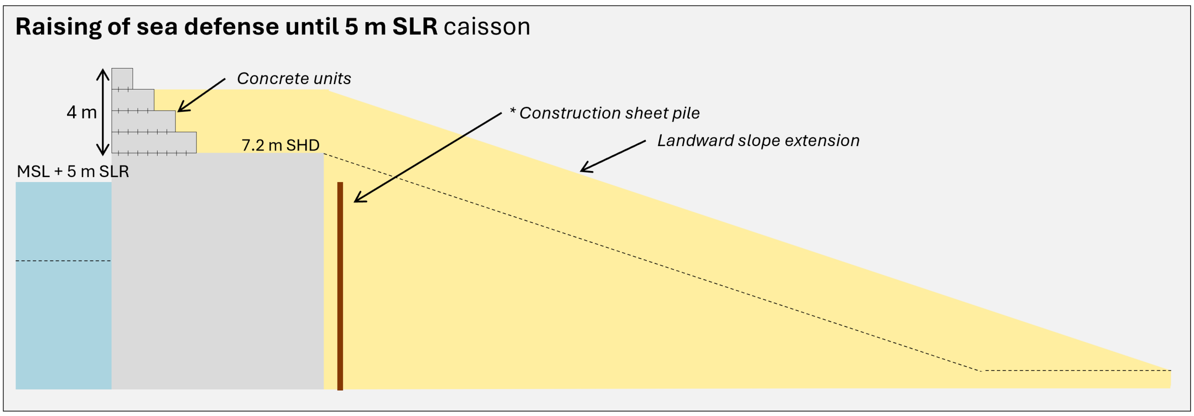


Figure 4.13: Overview of the raising of the caisson until 5 m of SLR. *The construction of the sheet pile is only applicable for the highest four platform levels (1.5, 4, 4.5, 5.1 m SHD), as the two lowest levels already include a sheet pile in the initial design.

4.4. Additional land reclamation components

4.4.1. Reservoir defense design

For the reservoir defense design, a dike option is considered. The design for the reservoir dike has the same characteristics as the sea dike option.

Core The core is made of sand, slope is 1V:3H.

Armor protection The armor protection consists of 1 layer of armor. The rock class is assumed to be LMA15-300, being equal to the under layer of the armor protection of the sea dike. Underneath the armor protection a geotextile is placed.

Toe The toe is constructed of the same rock class as for the sea dike.

Sand key The sand key thickness is reduced to 4 meters. This is because the vertical load and the footprint of the reservoir dike is significantly smaller than for the sea dike, reducing the chance for macro instability.

The crest height of the dike is determined by the maximum reservoir water level $h_{r,max} = 1.5$ m SHD. By taking a safety margin of 0.5 m, the crest height of the reservoir dike becomes 2 m SHD.

Platform level z_{LI} [m SHD]	Island A	Island B	Island C
-4	18.5	22.5	16.5
-2	18.5	22.5	16.5

Table 4.9: Length of the bentonite wall for the reservoir dike in meters.

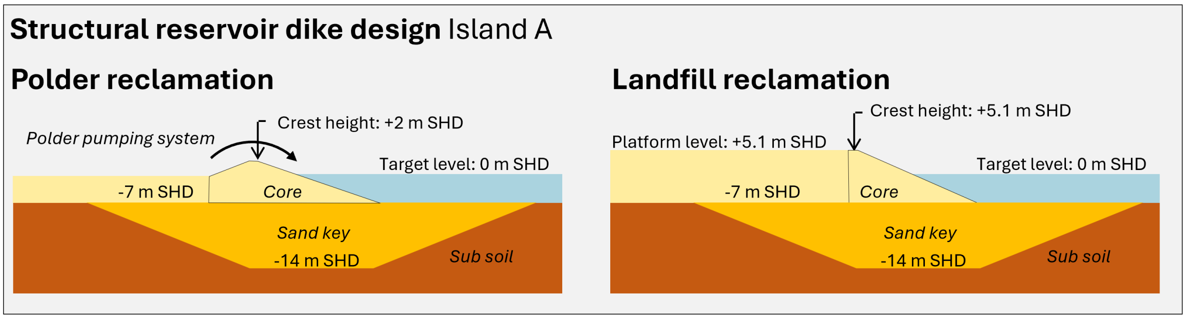


Figure 4.14: Structural design of reservoir dike for Island A. Left: polder reclamations. Right: landfill reclamations, example given for platform level of 5.1 m SHD. (crest height dike raises according to the platform level)

4.4.2. Effective land area correction

The inner slope of the flood defenses cannot be used for urban development, which results in a reduction of the effective land area. To still provide the required 850 hectares of land for urban development, the sea defense is "moved seaward". This results in the surface area of Long Island (including the inner slopes) becoming larger, which is relevant for the design of water management-related elements. The seaward movement and the resulting increased surface area of Long Island are listed in Table 4.10. The associated additional reclamation costs are shown in Figure 5.5.

The reduction of the land area for the dike option is illustrated in Figure 4.15. The inner slope of the sea dike $L_{innerslope,seadike}$ and reservoir dike $L_{innerslope,reservoirdike}$ determine the length of the seaward "movement" of the sea dike (see Equation 4.24).

$$L_{seaward,dike} = L_{innerslope,seadike} + L_{innerslope,reservoirdike} \tag{4.24}$$

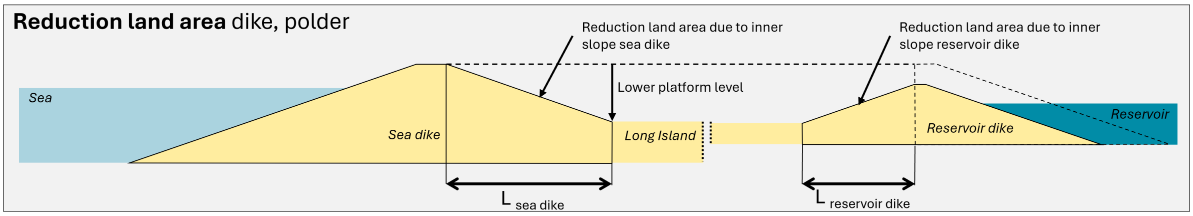


Figure 4.15: Sketch of polder reclamation with dike as sea defense option, illustrating the reduction in effective land area caused by the inner slopes of the sea dike and reservoir dike.

The same approach is applied for the caisson option, which is illustrated in Figure 4.16 and with the length of the seaward "movement" of the caisson defined in Equation 4.25.

$$L_{seaward,caisson} = L_{innerslope,caisson} + L_{innerslope,reservoirdike} \tag{4.25}$$

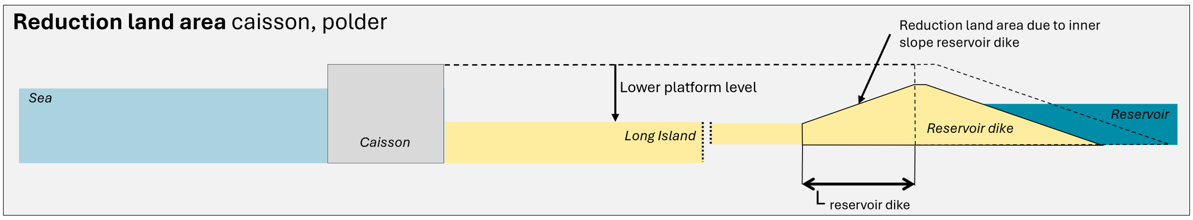


Figure 4.16: Sketch of polder reclamation with caisson as sea defense option, illustrating the reduction in effective land area caused by the inner slope of the reservoir dike only.

The resulting effective land area correction is illustrated in Figure 4.17. The corresponding lengths per sea defense option and the resulting surface area of Long Island are presented in Table 4.10.

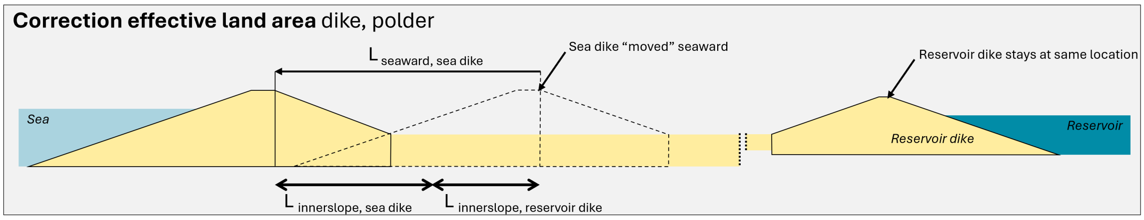


Figure 4.17: Illustration of the correction in effective land area by the seaward movement of the sea defense, using the dike option as an example.

Platform level	Sea dike option		Caisson option	
z_{LI} [m SHD]	$L_{seaward,dike}$ [m]	A_{LI} [ha]	$L_{seaward,caisson}$	A_{LI} [ha]
-4	46	974*	60	989*
-2	34	953*	48	968*
1.5	13	867	27	882
4	4	855	18	870
4.5	3	853	16	868
5.1	1	851	14	866

Table 4.10: Length of seaward movement of sea defense $L_{seaward}$ and resulting surface area of Long Island A_{LI} based on effective land area correction. *Including additional water storage of 50 ha (see Section 4.4.3 for further explanation).

4.4.3. Polder pumping system

The land reclamations with the two lowest platform levels (-4 and -2 m SHD) require a pumping system to discharge water from the islands into a reservoir. The pumping capacity is estimated based on rainfall-induced discharges (Q_r) and the seepage volumes through the reservoir defense (Q_s). In total, the pumping capacity Q_{pump} is 150 and 140 m^3/s, for a platform level of -4 and -2 m SHD, respectively. The detailed values of Q_r, Q_s and Q_{pump} are listed in Table 4.11.

$$Q_{pump} = Q_r + Q_s \tag{4.26}$$

The discharge Q_r required to pump out the rainfall is based on the average rainfall intensity I_{avg} of the design rainfall event and Long Island's surface area A_{LI}. The peak in rainfall intensity can be larger than the determined pumping capacity, as illustrated in Figure 4.18, which could lead to pluvial flooding. This is resolved by reserving additional storage capacity, which is done by increasing the surface area of Long Island by 50 ha.

$$Q_r = A_{LI} \cdot I_{avg} \cdot 10^{-3} \cdot 60 \tag{4.27}$$

With:

$$I_{avg} = D/t \tag{4.28}$$

Where:

Q_{pump}	=	polder pumping capacity [m^3/s]
A_{LI}	=	area Long Island (including effective land area correction and water storage)[m^2]
I_{avg}	=	52, average rainfall intensity [mm/hr]
D	=	207, total rainfall depth [mm]
t	=	4, storm duration [hrs]

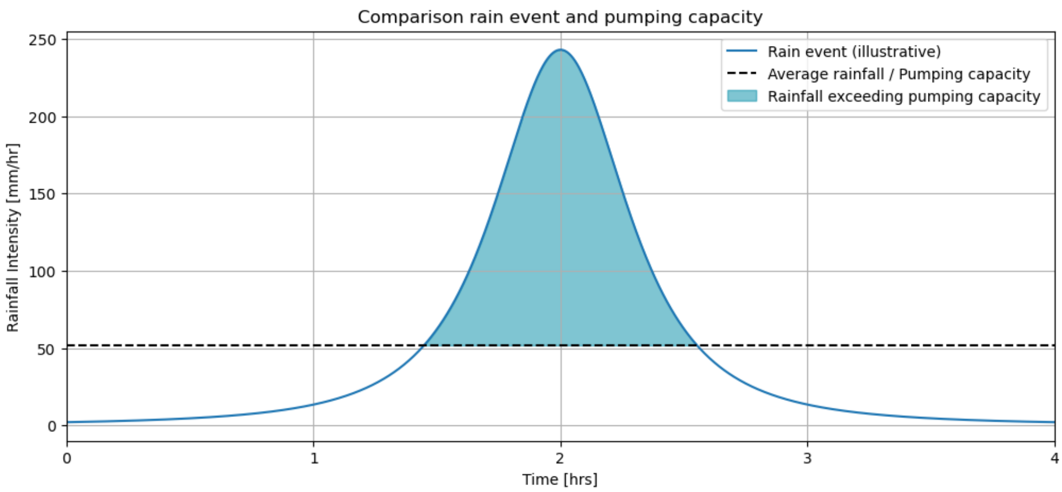


Figure 4.18: Comparison between average rainfall intensity (equal to pumping capacity) and varying rainfall intensity over time. The blue shaded area indicates periods where rainfall exceeds the pumping capacity.

The discharge Q_s needed to pump out the collected seepage volume is determined by using Equation 4.29.

$$Q_s = q \cdot (h_r - GWT) \cdot L_{reservoirdike} \tag{4.29}$$

By assuming a simple, unidirectional flow, the specific discharge q can be determined with Darcy's law defined in Equation 4.30 (Verruijt, 2001).

$$q = k \cdot i \tag{4.30}$$

The hydraulic gradient i is the difference in hydraulic head along the seepage path L.

$$i = (h_r - GWT)/L \tag{4.31}$$

The length of the seepage path L is the shortest path between the reservoir water level and the groundwater table, which is illustrated in Figure 4.20.

$$L = \Delta x^2 + \Delta y^2 \tag{4.32}$$

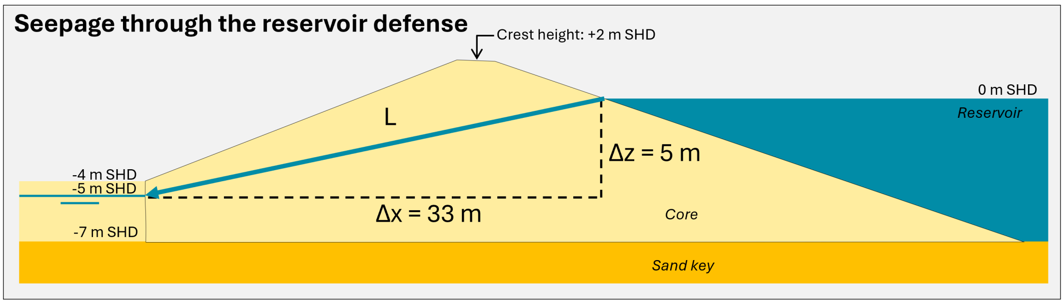


Figure 4.19: Seepage process through the reservoir defense for polder reclamations, a platform level of -4 m SHD is taken as an example. Definition of seepage length L.

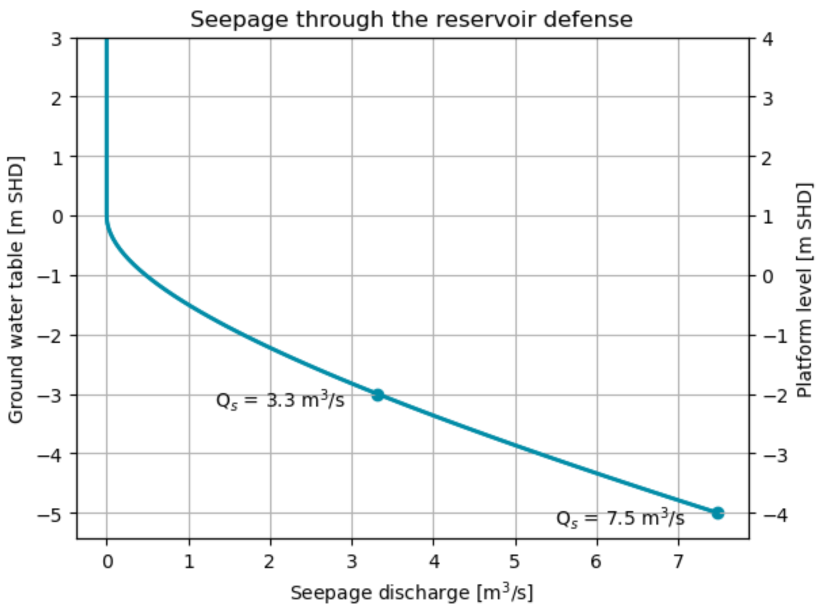


Figure 4.20: Total seepage discharge through the reservoir defense in m^3/s as a function of the platform level.

Platform level	Sea dike option			Caisson option		
z_{LI}[m SHD]	Q_r [m^3/s]	Q_s [m^3/s]	Q_{pump} [m^3/s]	Q_r [m^3/s]	Q_s [m^3/s]	Q_{pump} [m^3/s]
-4	140	7	147	142	7	149
-2	137	3	140	139	3	142

Table 4.11: Pumping capacity requirements for two sea defense options (sea dike and caisson) at different platform levels (z_{LI} in m SHD). The table presents rainfall-induced discharges (Q_r), the seepage flow (Q_s), and the resulting total pumping capacity (Q_{pump}), all in m^3/s. Due to minor differences between the sea defense types, standardized values of $Q_{pump} = 140$ m^3/s at -2 m SHD and $Q_{pump} = 150$ m^3/s at -4 m SHD are taken for design.

4.4.4. Settlement correction

The construction of Long Island will increase pressures in the soft subsoil, causing it to compress. The platform level determines the pressure increase and thus the amount of settlements (see Figure 4.21). These settlements are estimated per platform level (see Table 4.12), and accounted for in the reclamation volume calculations (see Figure 5.5).

The considered settlement is the primary consolidation that occurs during the construction phase of Long Island. The expected settlement can be determined using Terzaghi's equation, formulated in 4.33 (Verruijt, 2001). Due to the unknown local soil characteristics, the compression constant is estimated using geotechnical research at the Changi Airport reclamation project (Chu et al., 2009). See Appendix D for the approach to the determination of the compression constant and further details on the calculation.

$$\varepsilon = -\frac{1}{C} \cdot \ln(\frac{\sigma}{\sigma_0}) \tag{4.33}$$

Where:

C	=	25, compression constant [-]
σ	=	future vertical stress [kN/m^2]
σ_0	=	initial vertical stress [kN/m^2]
ε	=	$\frac{S}{H}$, strain [-]. S is the settlement [m], H layer thickness [m]

Platform level z_{LI} [m SHD]	Average settlement S [m]
-4	-0.2
-2	-0.5
1.5	-0.9
4	-1.2
4.5	-1.2
5.1	-1.3

Table 4.12: Expected settlement of the land reclamation per platform level. The considered settlement is the primary consolidation that occurs during the construction phase of Long Island.

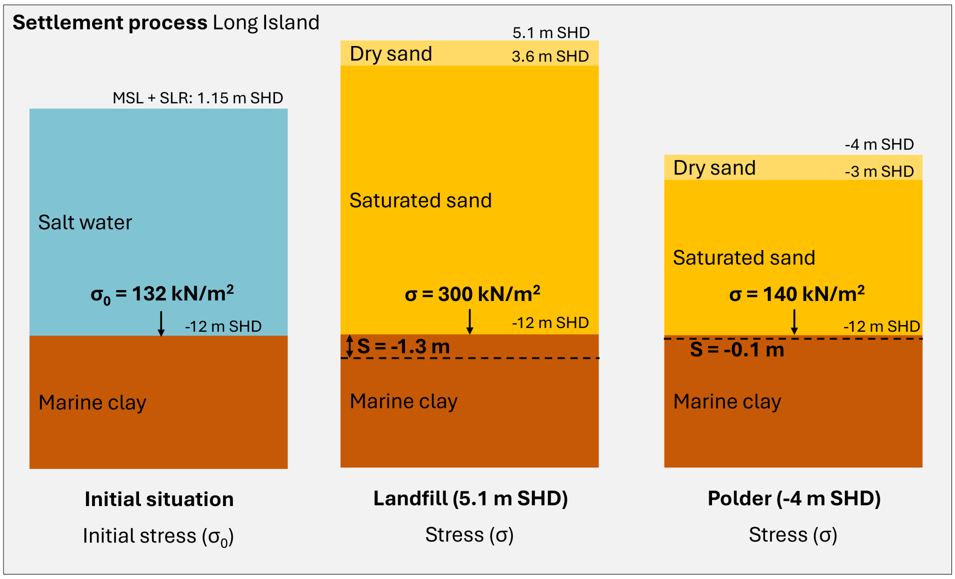


Figure 4.21: Illustration of the settlement process of the land reclamation based on stresses. Left: initial situation, Middle: Landfill reclamation (5.1 m SHD), and Right: polder reclamation (-4 m SHD).

5 Evaluation

The evaluation is split up into four parts. First, in Section 5.1, a cost analysis is performed on the initial capital expenditures. Next, the lifetime costs and adaptation pathways are evaluated based on Present Value calculations, which is found in Section 5.2. Next, a Multi Criteria Analysis is performed in Section 5.3. Finally, in Section 5.4, the sensitivity analysis is presented, which checks the uncertainty of the evaluation outcomes.

5.1. Cost Analysis: initial capital expenditures

In this Section, the initial capital expenditures of each land reclamation variant are presented. The initial CAPEX are the costs associated with the construction of Long Island. First, an overview of the total is given. Next, the details of the cost is given by looking at the costs per design component, consisting of the sea defense (dike and caisson), reservoir defense, polder pumping system, and the land reclamation.

Figure 5.1 gives an overview of the initial capital expenditures in billion SGD, for different land reclamation variants. The dike option (left) has CAPEX ranging from 8.0 billion SGD for a platform level of -4 m SHD to 11.1 billion SGD for a platform level of 5.1 m SHD. The caisson option (right) ranges from 7.8 billion to 11.2 billion SGD. Key findings:

- The total CAPEX for the dike option and caisson option fall within similar ranges, indicating no major cost differences between the considered flood defense types.
- The land reclamation costs account for the majority of the total CAPEX, highlighting the dominance of platform level changes on the overall costs.
- Raising the platform level significantly increases the costs: the difference between the lowest and highest levels is 3.1 billion SGD for the dike and 3.4 billion for the caisson.
- The investments in polder-specific measures (bentonite wall, sheet pile, and the pumping system) are relatively small, being about 0.6 billion SGD, compared to the total expenditures.

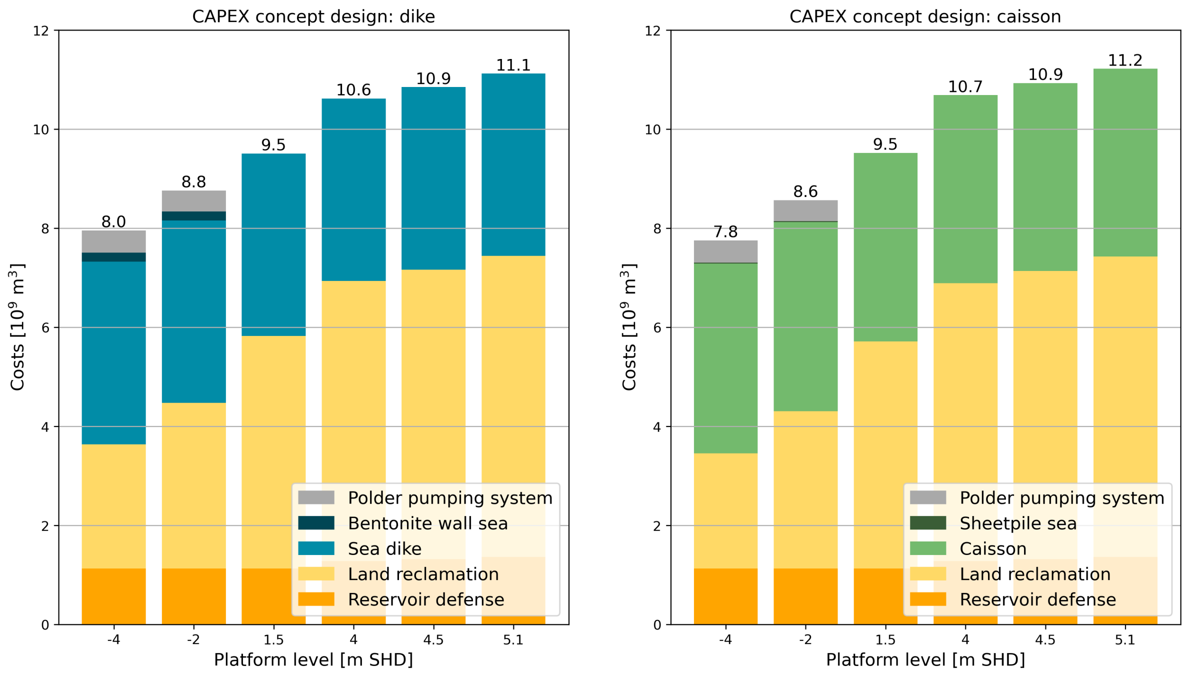


Figure 5.1: Initial capital expenditures of the conceptual designs per platform level in billion SGD. Left: dike option, right: caisson option.

5.1.1. Sea defense: dike

Figure 5.2 presents the material volumes and associated costs for the dike option. Volumes are shown in million m^3 per sea dike element, and costs in billion SGD per platform level. Key findings:

- The majority of material volume is required for the sand key and the core, indicating that optimization of other components (e.g. armor protection) will have limited effect on total costs.
- The bentonite wall is only included at the two lowest platform levels (-4 m SHD and -2 m SHD), with associated costs of approximately 0.2 billion SGD.
- The costs for the dike, excluding the bentonite wall, remain constant at 3.7 billion SGD across all platform levels.

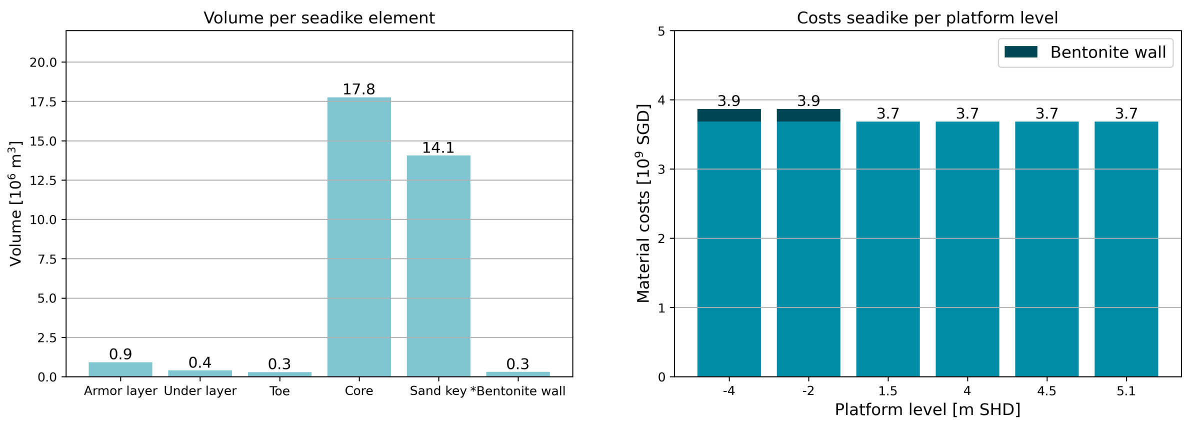


Figure 5.2: Left: Required volume per element for dike in million m^3. Right: Costs per platform level for the dike option in billion SGD. *Sheet pile only applicable for platform levels -4 and -2 m SHD.

5.1.2. Sea defense: caisson

In Figure 5.3, the material volumes and costs for the caisson option are presented. Again, Volumes are shown in million m^3 per caisson element, and costs in billion SGD per platform level.

- Most of the material volume is required for the sand key, with additional volumes needed for caisson filling and the inner slope.
- Total costs remain constant at 3.8 billion SGD across all platform levels. The cost contribution of the sheet pile, used only at the two lowest platform levels (-4 m SHD and -2 m SHD), is negligible.

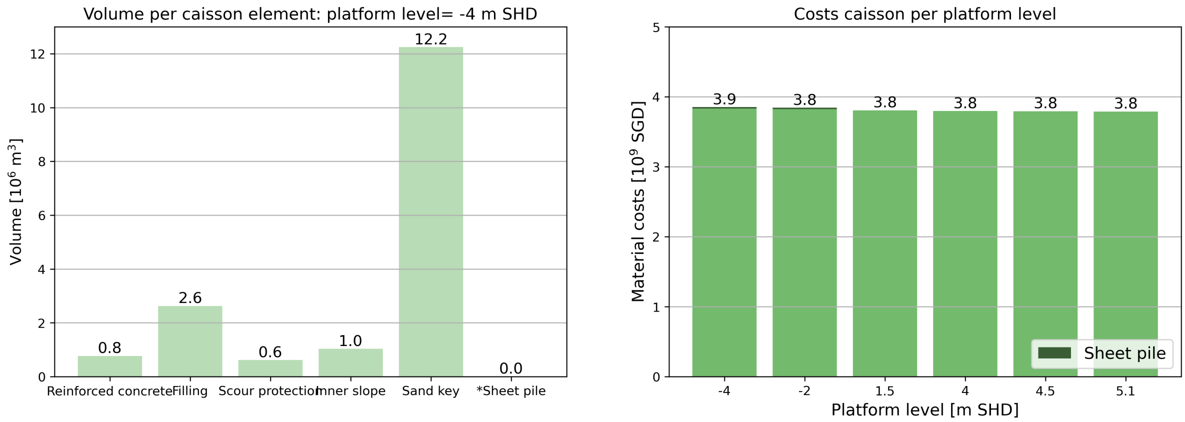


Figure 5.3: Left: Required volume per element for caisson in million m^3. Right: Costs per platform level for the caisson option in billion SGD. *Sheet pile only applicable for platform levels -4 and -2 m SHD.

5.1.3. Reservoir defense

Figure 5.4 shows the reservoir dike costs across platform levels, in billion SGD. Higher platform levels require larger crest heights, increasing material volumes and overall costs. At the highest platform level (5.1 m SHD), costs rise to 1.4 billion SGD. For the three lowest levels, costs remain constant at 1.1 billion SGD.

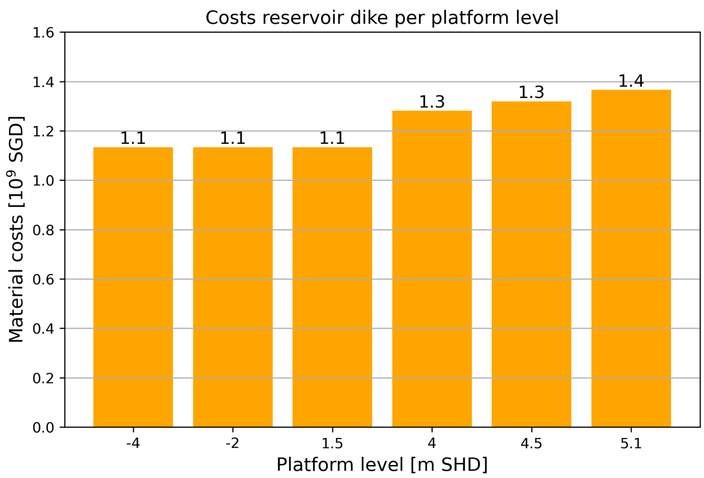


Figure 5.4: Costs per platform level for reservoir dike in billion SGD.

5.1.4. Polder pumping system

The polder pumping system is only required for the two lowest platform levels (-4 and -2 m SHD). Initial capital expenditures are based on the required pumping capacity, including full redundancy, estimated at 300 m³/s and 280 m³/s, respectively. This results in CAPEX values of 450 million SGD for -4 m SHD and 420 million SGD for -2 m SHD.

5.1.5. Land reclamation

In Figure 5.5, the reclamation costs are plotted per sea defense option and per platform level, expressed in billion SGD.

- The reclamation costs per platform level are comparable per defense type.
- The difference in reclamation costs between the lowest and highest platform levels is 3.6 billion SGD, with the lowest level being the most cost-effective.
- Additional costs due to settlement corrections range from 0.1 billion SGD at -4 m SHD to 0.5 billion SGD at 5.1 m SHD.
- Effective land area corrections result in additional costs of 0.3 billion SGD at -4 m SHD, decreasing to nearly zero at 5.1 m SHD.
- Settlement corrections have a greater impact on total reclamation costs than effective land area corrections.
- For the caisson option, part of the reclamation cost includes the construction of an inner slope to better integrate the defense into its surroundings. This adds 0.3 billion SGD at the two lowest platform levels and 0.2 billion SGD at higher levels.

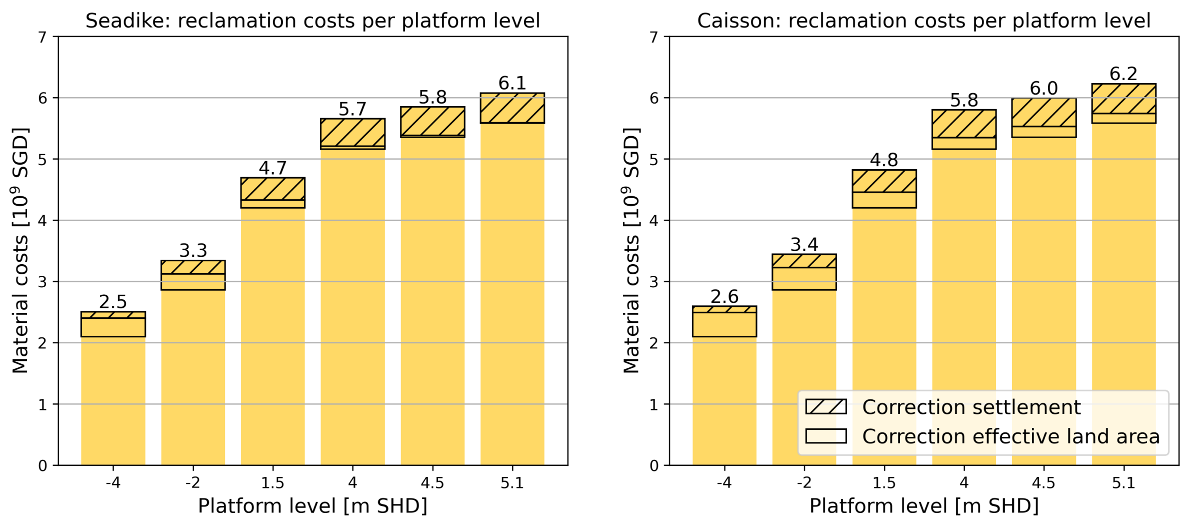


Figure 5.5: Reclamation costs per platform level in billion SGD, indicating the corrections for settlements and the effective land area. Left: dike option, right: caisson option.

5.2. Cost analysis: present value pathways

This section evaluates the pathway strategies for each sea defense option by calculating the Present Value (PV) of associated costs over time. To determine the PV, the costs per timestep are first calculated using Equation 5.1. These costs consist of three main components:

- $CAPEX_{a,t}$: Construction costs for sea defense adaptations, including project start-up costs. These depend on the selected pathway strategy. For the dike, adaptation costs are related to the required sand and rock materials. For the caisson, they include concrete blocks and sand for the inner slope and the filling of the blocks.
- $CAPEX_{p,t}$: Maintenance and replacement costs for pumping system components, including civil works maintenance and replacement of mechanical and electrical installations. These costs are independent of the chosen pathway.
- $OPEX_t$: Annual operating costs for the pumping system and maintenance of the sea defense.

$$C_t = CAPEX_{a,t} \cdot f_a \cdot f_m + CAPEX_{p,t} + OPEX_t \tag{5.1}$$

Where:

C_t = costs at time t [SGD]
$CAPEX_{a,t}$ = capital expenditures for sea defense adaptation at time t [SGD]
f_a = 1.2, factor for additional costs [-]
f_m = 1.2, factor for mobilization costs [-]
$CAPEX_{p,t}$ = replacement and maintenance costs for pumping system components [SGD]
$OPEX_t$ = Operating expenditures at year t [SGD/year]

The costs are implemented in the Present Value calculation, with the formula from Section 3.5 being repeated below (see Equation 5.2. The evaluation period is 110 years, from the base year 2040 to 2150.

$$PV_{t_0} = \frac{C_t}{(1 + SDR)^{t-t_0}} \tag{5.2}$$

Where:

PV_{t_0} = present value at base year t_0 [SGD]
C_t = future costs at time t [SGD]
SDR = Social Discount Rate [-]
t = future year [years]
t_0 = base year (2040) [years]

5.2.1. Present Value: dike

In Figure 5.6, the present value for the dike option is presented per pathway and per platform level. The figure indicates the costs associated with the pumping system separately. Key findings:

- The difference in PV between the highest and lowest platform levels is 3.4 billion SGD, consistent across all pathways.
- Pathway B shows a 0.4 billion SGD increase in PV compared to Pathway A, for all platform levels.
- Pathway C shows a 0.9 billion SGD increase in PV compared to Pathway A, for all platform levels.
- The PV of the pumping system is 1.2 billion SGD for platform level -4 m SHD and 1.1 billion SGD for -2 m SHD.
- Due to the pumping system costs, the PV at platform level 1.5 m SHD is lower than at -2 m SHD, despite the higher elevation.

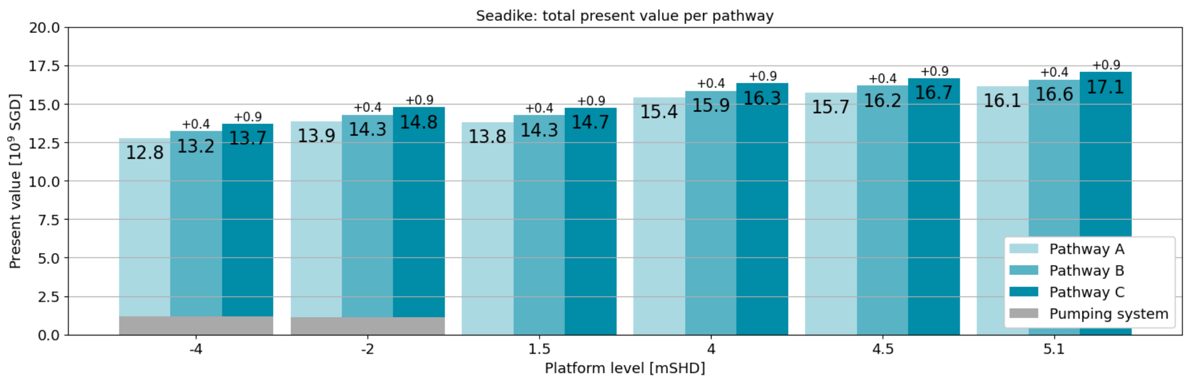


Figure 5.6: Present value for dike option per platform level and per pathway, expressed in billion SGD. Values indicated above the bars indicate the increase in Present Value relative to Pathway A.

5.2.2. Present Value: caisson

In Figure 5.6, the PV for the caisson option is presented in the same manner as for the dike option. Key findings:

- The difference in PV between the highest and lowest platform levels is 3.6 billion SGD, consistent across all pathways.
- Pathway B shows a 0.4 billion SGD increase in PV compared to Pathway A, for all platform levels, which is the same as for the dike option.
- Pathway C shows a 1.1 billion SGD increase in PV compared to Pathway A, for all platform levels, which is slightly higher than the dike option.
- The PV of the pumping system is again 1.2 billion SGD for platform level -4 m SHD and 1.1 billion SGD for -2 m SHD.

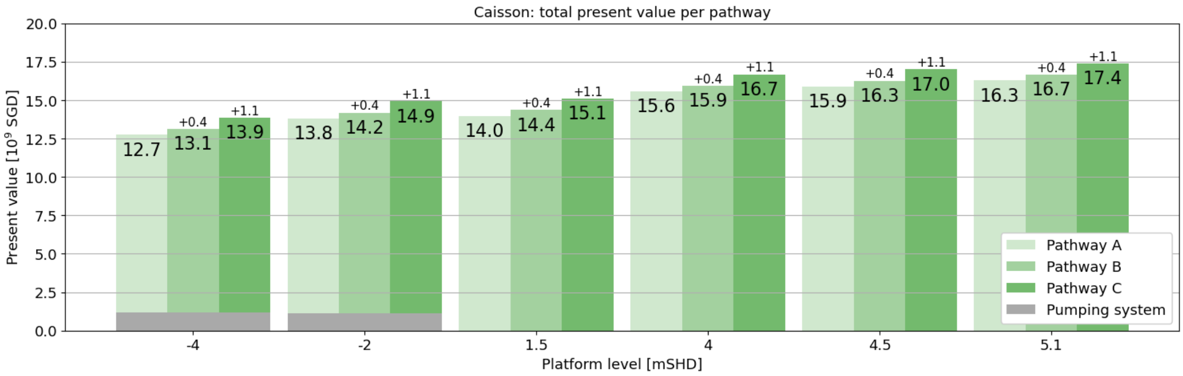


Figure 5.7: Present value for caisson option per platform level and per pathway, expressed in billion SGD. Values indicated above the bars indicate the increase in Present Value relative to Pathway A.

5.3. Multi Criteria Analysis

In this Section, the Multi Criteria Analysis is performed. In this analysis, the evaluation criteria are ranked in importance by assigning a weighting factor to each criteria. Next, the alternatives, being a dike or caisson in combination with a platform level, are scored per criterion and multiplied by the corresponding weighting factor. Next, the value-cost ratio is determined, from which an optimal alternative can be chosen.

5.3.1. Weighting of evaluation criteria

As defined in Section 3.2, the evaluation criteria are used to assess the performance of each sea defense concept. Figure 5.8 presents the resulting weighting factors per criterion. The criteria are described and ranked by importance below:

1. Minimal required resources: This criterion reflects the total volume of material needed for the construction of Long Island. It is considered the most important, as reducing material use frees up budget and resources for other objectives, such as enhancing recreational and ecological value. It also outweighs adaptability, since reconstructing a sea defense would still require less material than the entire land reclamation.
2. Adaptable sea defense: The ability to adapt to future sea level rise is essential for long-term system performance and cost-efficiency. It ranks above spatial and recreational aspects due to its significant impact on future capital expenditures.
3. Residual flood risk: While the protection level is equal across all concepts, lower platform levels increase the potential for damage and casualties in the event of failure. This makes residual flood risk more important than spatial integration and recreational value.
4. Nature-friendly and recreational coastline: The sea defense should support ecological functions and enable recreational use. Although important, this criterion ranks below residual flood risk, as public safety is prioritized.

5. Integration into the surrounding: A sea defense that integrates well with urban development enhances spatial quality and public access to the waterfront. However, it is considered the least important, as its benefits are more limited in scope compared to the broader ecological and safety-related criteria.

Criteria		a	b	c	d	e	Total	(Total +1)* 2	WF
Minimal required resources	a		1	1	1	1	4	10	10/30= 0.33
Adaptable sea defense	b	0		1	1	1	3	8	8/30= 0.27
Integration into surrounding	c	0	0		0	0	0	2	2/30= 0.07
Nature friendly and recreational coastline	d	0	0	1		0	1	4	4/30= 0.13
Residual flood risk	e	0	0	1	1		2	6	6/30= 0.20
								30	**1.00**

Figure 5.8: Weighting factors (denoted as WF) per evaluation criteria.

5.3.2. Scoring alternatives

To evaluate the performance of each sea defense alternative, scores were assigned per evaluation criterion and weighted according to their relative importance. The weighted scores are presented in Figure 5.9. The highest overall score of 5.8 is achieved by the dike option combined with a platform level of 4.0 m SHD. However, the differences between alternatives are small, with scores ranging from 5.5 to 5.8. The scoring per criterion is summarized below:

1. Minimal required resources: The caisson option scores higher than the dike due to its lower material requirements. As the platform level increases, the score decreases because more material is needed.
2. Adaptable sea defense: The dike scores higher in terms of adaptability. It allows for easier and more flexible modifications over time. In contrast, the caisson has limitations due to its modular concrete units, which restrict the geometry of future adaptations. The platform level does not influence this criterion.
3. Residual flood risk: Lower platform levels score poorly due to higher potential damage and risk in case of failure. Scores improve as the platform level increases. This criterion is not affected by the type of sea defense.
4. Nature-friendly and recreational coastline: Both sea defense types score low, as they result in hard structures. However, the dike performs slightly better due to its sloped profile, which offers more potential for ecological integration. The platform level has no effect on this criterion.
5. Integration into the surrounding: The dike scores higher because of its lower crest height, which improves spatial integration. Lower platform levels score worse, as they require a greater elevation difference to be overcome between the land and the sea defense.

		Dike, -4 m SHD		Dike, -2 m SHD		Dike, 1.5 m SHD		Dike, 4 m SHD		Dike, 4.5 m SHD		Dike, 5.1 m SHD	
Criteria	WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF
Minimal required resources	0.33	7	2.3	7	2.3	4	1.3	3	1.0	2	0.7	2	0.7
Adaptable sea defense	0.27	8	2.1	8	2.1	8	2.1	8	2.1	8	2.1	8	2.1
Integration into surrounding	0.07	4	0.3	5	0.3	5	0.3	8	0.5	8	0.5	8	0.5
Nature friendly and recreational coastline	0.13	4	0.5	4	0.5	4	0.5	4	0.5	4	0.5	4	0.5
Residual flood risk	0.20	2	0.4	2	0.4	6	1.2	8	1.6	9	1.8	9	1.8
Weighted score	1	5.7		5.7		5.5		5.8		5.7		5.7	

		Caisson, -4 m SHD		Caisson, -2 m SHD		Caisson, 1.5 m SHD		Caisson, 4 m SHD		Caisson, 4.5 m SHD		Caisson, 5.1 m SHD	
Criteria	WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF	Score	SC * WF
Minimal required resources	0.33	8	2.7	8	2.7	5	1.7	4	1.3	3	1.0	3	1.0
Adaptable sea defense	0.27	7	1.9	7	1.9	7	1.9	7	1.9	7	1.9	7	1.9
Integration into surrounding	0.07	3	0.2	3	0.2	4	0.3	7	0.5	8	0.5	8	0.5
Nature friendly and recreational coastline	0.13	3	0.4	3	0.4	3	0.4	3	0.4	3	0.4	3	0.4
Residual flood risk	0.20	2	0.4	2	0.4	6	1.2	8	1.6	9	1.8	9	1.8
Weighted score	1	5.5		5.5		5.4		5.7		5.6		5.6	

Figure 5.9: Scoring of the sea defense options in combination with the platform levels. Top: dike option, bottom: caisson option.

5.3.3. Value-cost ratio

The developed scores are divide by the costs of the land reclamation variants to get the value-cost ratio. In Figure 5.10, the value-cost ratio for the initial CAPEX is shown. The highest ratio is found for the platform level with -4 m SHD for both sea defense types, or platform level of -2 m SHD of the dike. But the differences with the other variants is very limited. For the present value calculations, no differences in the value-cost ratios is found across all variants. In Figure 5.11 this is shown for Pathway A, same is found for the other pathways.

	Dike, -4 m SHD	Dike, -2 m SHD	Dike, 1.5 m SHD	Dike, 4 m SHD	Dike, 4.5 m SHD	Dike, 5.1 m SHD
Weighted score	5.7	5.7	5.5	5.8	5.7	5.7
CAPEX concept design [10^9 SGD]	8.0	8.8	9.5	10.6	10.9	11.1
Value-cost ratio	**0.7**	**0.7**	**0.6**	**0.5**	**0.5**	**0.5**

	Caisson, -4 m SHD	Caisson, -2 m SHD	Caisson, 1.5 m SHD	Caisson, 4 m SHD	Caisson, 4.5 m SHD	Caisson, 5.1 m SHD
Weighted score	5.5	5.5	5.4	5.7	5.6	5.6
CAPEX concept design [10^9 SGD]	7.8	8.6	9.5	10.7	10.9	11.2
Value-cost ratio	**0.7**	**0.6**	**0.6**	**0.5**	**0.5**	**0.5**

Figure 5.10: Value-cost ratio based on scoring of the evaluation criteria and the CAPEX of the concept design. Top: dike option, bottom: caisson option.

	Dike, -4 m SHD	Dike, -2 m SHD	Dike, 1.5 m SHD	Dike, 4 m SHD	Dike, 4.5 m SHD	Dike, 5.1 m SHD
Weighted score	5.7	5.7	5.5	5.8	5.7	5.7
PV pathway A [10^9 SGD]	12.8	13.9	13.8	15.4	15.7	16.1
Value-cost ratio	**0.4**	**0.4**	**0.4**	**0.4**	**0.4**	**0.4**

	Caisson, -4 m SHD	Caisson, -2 m SHD	Caisson, 1.5 m SHD	Caisson, 4 m SHD	Caisson, 4.5 m SHD	Caisson, 5.1 m SHD
Weighted score	5.5	5.5	5.4	5.7	5.6	5.6
PV pathway A [10^9 SGD]	12.7	13.8	14.0	15.6	15.9	16.3
Value-cost ratio	**0.4**	**0.4**	**0.4**	**0.4**	**0.4**	**0.3**

Figure 5.11: Value-cost ratio based on scoring of the evaluation criteria and the Present Value of Pathway A. Top: dike option, bottom: caisson option.

5.4. Sensitivity Analysis

To evaluate the robustness of the design outcomes, a sensitivity analysis was performed on key assumptions, including design parameters, Social Discount Rate, and sea level rise scenarios. Variations in design parameters showed that the material costs, the sand key thickness and the water depths have the greatest impact on initial capital expenditures. Changes in SDR significantly affect the Present Value, particularly for lower platform levels due to future pumping costs. Different SLR scenarios had a relatively minor influence on the Present Value, with only slight variations observed across pathway strategies.

5.4.1. Design parameters

Five design parameters were varied to asses the uncertainty in the key design assumptions, with the variation values being detailed in Table 5.1.

- Material costs: Due to unit rates potentially varying over time, material costs are varied using multiplication factors of 1.2 and 0.8.
- Thickness of the sand key at the sea defense: The assumed 10 m thickness is varied by +2 m and -3 m.
- Water depth: Due to uncertainty in the reliability of the publicly available bathymetry map and the simplistic estimations performed, the derived water depths are multiplied by a factor of 1.1 and 0.9.
- Crest height sea defense: To account for potential changes from future physical model testing, the crest height is varied by +1 m and -0.5 m.
- Settlement: Given the unknown subsoil characteristics, the settlement depth is adjusted using factors of 1.1 and 0.9.

Design parameter	Variation type	Maximum	Minimum
Material costs [SGD/m^3]	Multiply	1.2	0.8
Sand key thickness sea defense [m]	Add/Substract	+2 m	-3 m
Water depth [m SHD]	Multiply	1.1	0.9
Crest height sea defense [m SHD]	Add/Substract	+1 m	-0.5 m
Settlement [m]	Multiply	1.1	0.9

Table 5.1: Variations of the design parameters used in the sensitivity analysis.

The combined variations of the design parameters were used to recalculate the initial CAPEX. The resulting upper and lower bounds are shown as error bars in Figure 5.12. Key findings:

- The combination of the variations of the design parameters results in a significant potential decrease or increase of the CAPEX.
    - Dike option: At platform level -4 m SHD, costs range from +30% to -40%. At the highest platform level, the range shifts to +40% and -30%.
    - Caisson option: At platform level -4 m SHD, costs change by +40% or -35%. At the highest platform level, the range is +30% and -30%.
- The range of the error bar increases as the platform level increases.
    - Dike option: The range spans 6.5 billion SGD at the lowest platform level, increasing to 7.9 billion SGD at the highest.
    - Caisson option: The range spans 5.6 billion SGD at the lowest platform level, increasing to 7.2 billion SGD at the highest.
- The cost difference between the lowest and highest platform levels is reduced when considering the lower bound of the sensitivity range.
    - Dike option: The upper limit shows a difference of 4.1 billion SGD between the lowest and highest platform levels, while the lower limit shows a smaller difference of 2.7 billion SGD.
    - Caisson option: The upper limit difference is 4.5 billion SGD, and the lower limit difference is 2.9 billion SGD.

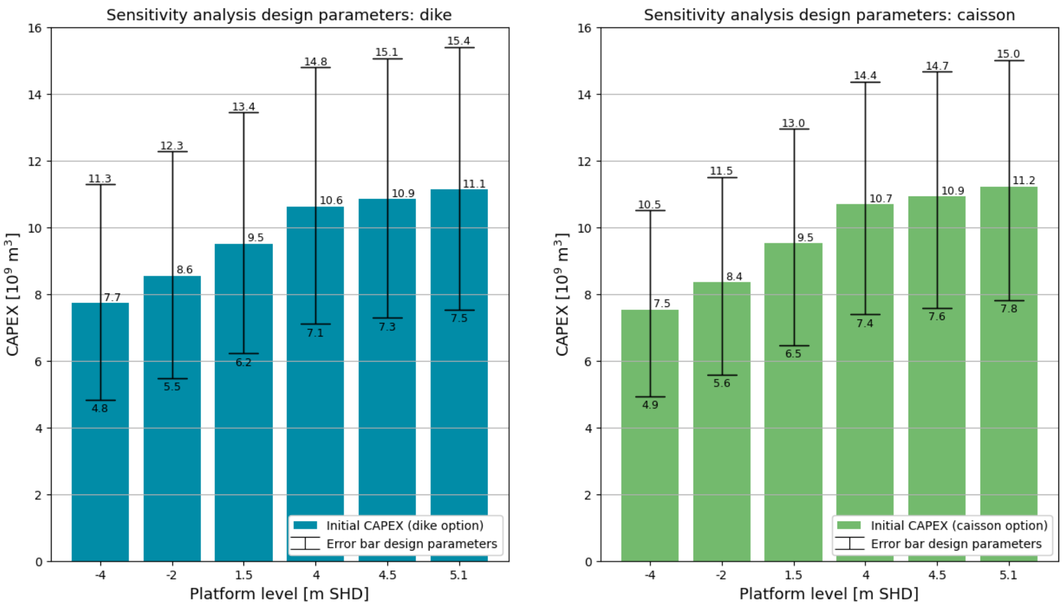


Figure 5.12: Impact of key design parameters on initial capital expenditures for two sea defense types. The bars represent the upper and lower bounds from the sensitivity analysis. Left: dike option, right: caisson option.

To determine which design parameters most significantly affect the initial CAPEX, a one-at-a-time sensitivity analysis is conducted. Only the results for increased parameter values are shown in Table 5.2 for clarity.

Material costs have the greatest influence on CAPEX. This is expected, as they directly affect the costs of the land reclamation, the sea defense, and the reservoir defense. Additionally, the applied variation (+20%) is relatively large. A 20% increase in unit rates results in a proportional 20% increase in total CAPEX, ranging from 1.5 billion SGD for the lowest platform level to 2.3 billion SGD for the highest. Consequently, the cost gap between platform levels widens with increasing material costs. Conversely, if material costs were significantly reduced, the lower platform level would no longer be the most cost-effective, as pumping costs would become more dominant relative to the material costs. But this is unlikely to occur in the context of Long Island, as it would require a drastic reduction in unit rates of more than 60 percent.

The sand key thickness of the sea defense and the water depth also have a notable impact on the CAPEX, each contributing to a 9–10% increase, equivalent to approximately 0.6–0.8 billion SGD across all platform levels. In contrast, variations in the crest height of the sea defense and the settlements of the land reclamation have limited effect, resulting in only a 1–2% increase in CAPEX.

Design parameter	Increase applied	CAPEX increase	CAPEX increase [$\cdot 10^9$ SGD]	
			Dike option	Caisson option
Material costs	+20%	+20%	1.5-2.3	1.5-2.3
Sand key thickness	+2 m	+10%	0.8	0.7
Water depth	+10%	+9%	0.7	0.6
Crest height	+1 m	+2%	0.2	0.2
Settlement	+10%)	+1%	<0.1-0.1	<0.1-0.1

Table 5.2: Results of the one-at-a-time sensitivity analysis for key design parameters, showing the resulting increase in initial CAPEX in billion SGD when each parameter is increased individually. The analysis highlights the relative impact of each parameter on total CAPEX for both the dike and caisson options.

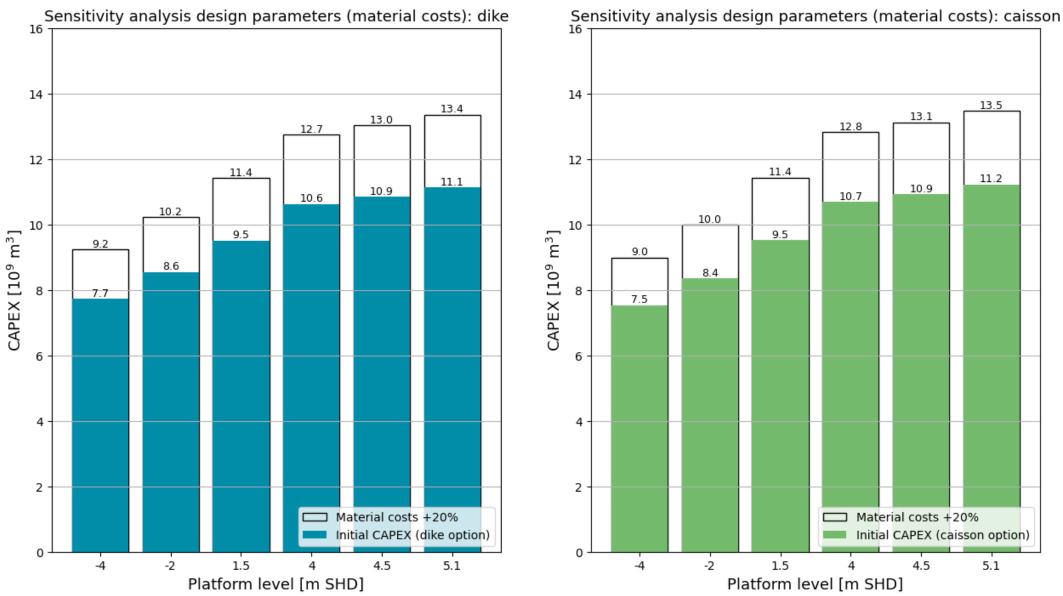


Figure 5.13: Effect of a 20% increase in material costs on initial capital expenditures. The cost difference between the lowest and highest platform levels increases with rising material costs. Left: dike option, right: caisson option.

5.4.2. Social discount rates

A sensitivity analysis was conducted to assess the impact of varying the Social Discount Rate on the Present Value, using values of 0.5% and 4% compared to the 2% base case. Although PVs were calculated for both the dike and caisson options, only the dike results are presented here for clarity (see Figures 5.14 and 5.15). Key findings:

- Reduction of the SDR (0.5%):
    - Future costs associated with the pumping system increase to such an extent that the PV of the -4 m SHD platform level equals that of 1.5 m SHD. It also results in the -2 m SHD platform level becoming more expensive than -4 m SHD.
    - Platform levels from 1.5 m SHD to 5.1 m SHD show a constant increase of the Present Value across all platform levels.
    - Similar trends are observed for the caisson option: -4 m SHD and 1.5 m SHD have equal Present Values, and -2 m SHD is more expensive than -4 m SHD.
- Increase of the SDR (4%):
    - Future pumping costs become less dominant. The PV of the two lowest platform levels decreases by approximately 0.6 billion SGD, compared to a reduction of about 0.1 billion SGD for the higher platform levels.
    - Again, similar trends are observed for the caisson option as for the dike option.

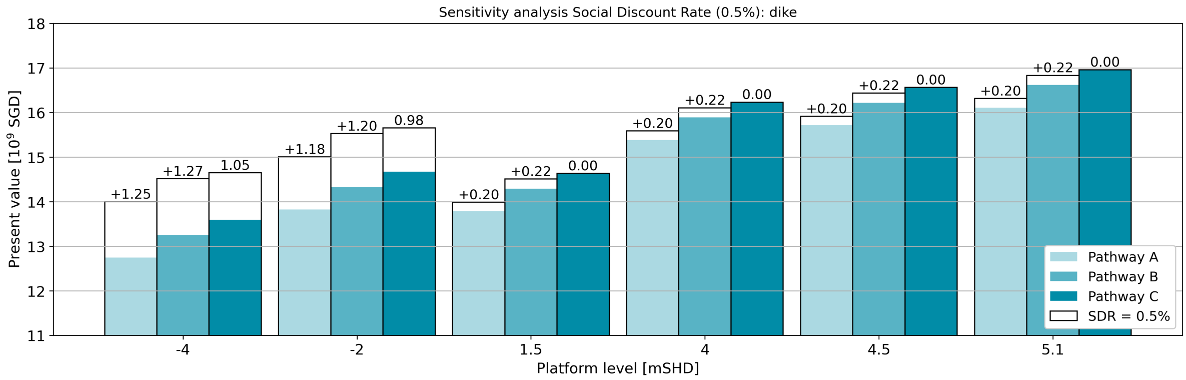


Figure 5.14: Present Value of the dike option under a low Social Discount Rate (0.5%). Lower SDR increases the influence of future costs, making low platform levels (-4 m and -2 m SHD) more expensive due to future costs associated with the pumping system.

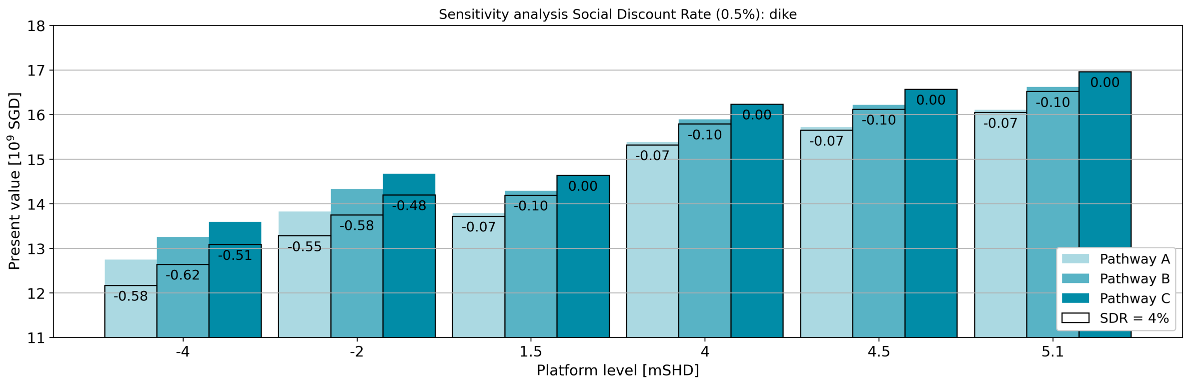


Figure 5.15: Present Value of the dike option under a high Social Discount Rate (4%). Higher SDR reduces the present value of future costs, particularly those related to the pumping system, leading to a more pronounced decrease in PV for the lowest platform levels.

5.4.3. Sea level rise scenarios

The initial Present Value calculations were based on the SSP5-8.5 scenario (83rd percentile). To assess the sensitivity of the calculations to different sea level rise projections, two additional scenarios are considered: a faster SLR scenario (SSP5-8.5, 95th percentile) and a slower SLR scenario (SSP2-4.5, 83rd percentile). Although the Present Value was calculated for both the dike and caisson options, only the results for the dike option are presented here to maintain clarity and focus in the comparison (see Figures 5.16 and 5.17). Key findings:

- The impact of SLR scenarios on the Present Value is limited for both design options.
- In all cases, Pathway C remains unaffected, as the full infrastructure is constructed upfront to accommodate 5 m of sea level rise, thereby excluding the need for any future investments influenced by SLR variations.
- Under the faster SLR scenario:
  - Dike option: The PV increases by approximately 0.03 billion SGD for Pathways A and B.
  - Caisson option: The PV increase ranges from 0.01 to 0.02 billion SGD.
- Under the slower SLR scenario:
  - Dike option: PV decreases by 0.05 and 0.06 billion SGD for Pathways A and B, respectively.
  - Caisson option: PV decreases by approximately 0.02 billion SGD.

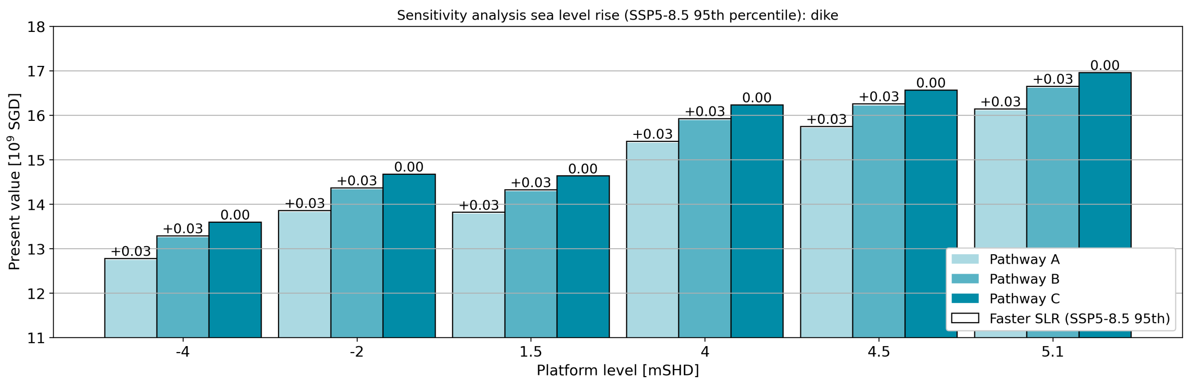


Figure 5.16: Sensitivity analysis for faster sea level rise (SSP5-8.5 95th percentile) for the dike option, showing a relative increase of the Present Value by 0.03 billion SGD.

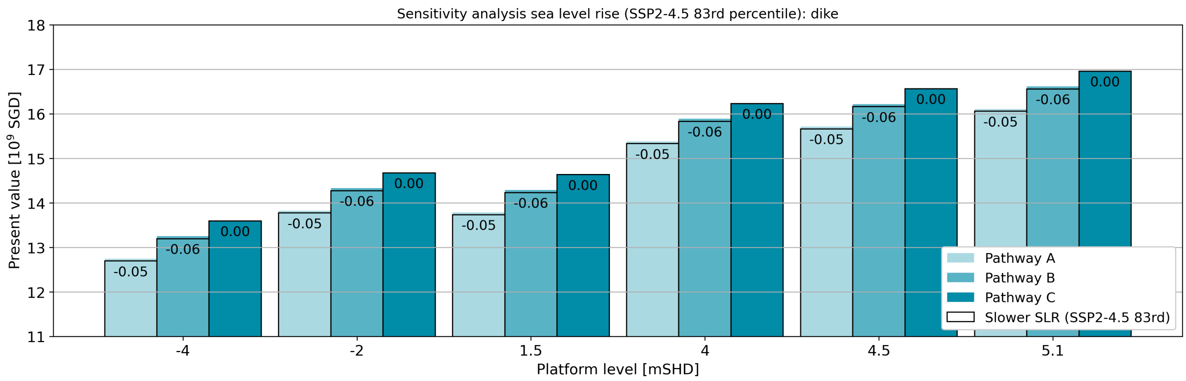


Figure 5.17: Sensitivity analysis for slower sea level rise (SSP2-4.5 83rd percentile) for the dike option, showing a relative decrease of the Present Value by 0.05 billion SGD.

6 Discussion

This chapter reflects on the assumptions, limitations, and implications of the conceptual design and evaluation of Long Island's land reclamation variants. The discussion is structured in three parts. First, the simplifications made in the system modeling are examined in Section 6.1. Second, the engineering aspects and design choices of the conceptual design are discussed, which is found in Section 6.2. Finally, in Section 6.3, the evaluation approach is reviewed, focusing on the sensitivity of the results.

6.1. Simplification system optimization

The conceptual design aimed to be as integral as possible within the limited timeframe of this thesis. The primary processes related to the sea defense and platform level were taken into account to support a comprehensive system-wide perspective. However, some processes, such as reservoir dynamics and seepage behavior, were simplified. Additionally, interactions between components, such as the influence of reservoir water levels on the operation of the pumping system, were not included. In future design phases, further refinement of system processes could help capture these interactions and identify potential risks and trade-offs, particularly under extreme conditions and future climate scenarios.

The dynamics of the reservoir system were simplified by assuming the Mean Sea Level as the target level and estimating the maximum water level based on the elevation of East Coast Park. In the current system optimization, reservoir water levels influence only the design of the reservoir dike and the seepage process. Their impact on drainage capacity in the case of landfill reclamation, or on the operation of the pumping system in the case of polder reclamation, was not included. This simplification was chosen to reduce the complexity of the analysis and is considered acceptable for high-level cost comparisons. However, it is important to recognize that both drainage capacity and pumping system operations play a critical role in managing pluvial flooding. Therefore, for future design detailing, a more comprehensive analysis of reservoir behavior and pluvial flooding processes is essential.

In the current conceptual design, measures against saltwater intrusion were included for the two lowest platform levels by assuming a required penetration depth for the seepage screen. To asses the water quality requirements for the Long Island reservoir, a comprehensive geohydrological assessment should be included in future design phases. This analysis should be carried out for all platform levels, be based on site-specific soil characteristics, and should account for sea level rise scenarios. Its primary objective is to evaluate the risk of saltwater intrusion into the reservoir. Based on the results, appropriate mitigation measures such as impermeable barriers or localized pumping should be developed in detail. The current approach used for the developed conceptual design is considered sufficient when comparing land reclamation variants, as the associated costs are limited compared with the total capital expenditures or the present value. However, to ensure the long-term performance of the reservoir, a more detailed geohydrological assessment remains essential.

6.2. Engineering limitations conceptual design

While the conceptual design provides a valuable foundation for evaluating land reclamation strategies, several engineering limitations were identified. These limitations arise from the multifunctional nature of the Long Island project and the study's focus on comparative design development, sea defense adaptation, and platform level optimization. Specifically, the limitations relate to the qualitative assessment of residual flood risk, simplified rainfall and pumping capacity analysis, preliminary geotechnical design of the sea defense, and the exclusion of nature-based solutions.

Residual flood risk of polders was only qualitatively assessed and not quantitatively evaluated in this study. This limitation results from the defined scope of the research, which prioritized the development and comparison of conceptual land reclamation designs over detailed flood risk modeling. Residual flood risk was therefore included as a qualitative criterion in the Multi-Criteria Analysis (MCA), without probabilistic quantification. Given the promising feasibility and cost-effectiveness of lower platform levels, polder systems appear to be a valid land reclamation option. To further support the decision-making process on platform level selection, further attention to residual flood risk is necessary, including flood risk analysis and stakeholder involvement.

The rainfall and pumping capacity analysis in this study was simplified, potentially underestimating the required capacity and costs. PUB guidelines were not fully applied. Instead, average rainfall intensity with a 25% climate adjustment was used, along with basic storage assumptions. This approach did not account for drainage dynamics or pump operations. Future research should refine these estimates using detailed modeling, which may reveal higher pumping needs, potentially making a platform level of 1.5 m SHD more favorable than -4 or -2 m SHD.

The foundation design of the sea defense focused on first-order dimensioning and assumed a sand key foundation. This foundation type was selected based on its application at Pulau Tekong, a comparable land reclamation project in Singapore. The future loading on the foundation due to the reinforcement of the sea defense to accommodate sea level rise was acknowledged. However, its technical feasibility and cost implications were not quantified. In future design phases, exploring alternative foundation types (e.g., stone columns or deep cement mixing), optimizing the geotechnical design, and incorporating the expandability of the sea defense could lead to more efficient material use while ensuring long-term performance. Finally, the settlement of the sea and reservoir defenses was not considered, which could also impact the associated costs of the flood defenses. While a more detailed geotechnical design could significantly influence the costs of Long Island's sea defense, the relative comparison between platform levels remains valid.

The conceptual design primarily focused on engineered flood protection measures, rather than the integration of nature-based solutions or the exploration of hybrid defenses such as perched beaches. This decision was driven by the emphasis on adapting sea defenses to projected sea level rise. Although the primary focus was on engineered solutions, this research demonstrated that platform level optimization can also create opportunities for nature-based enhancements. By reducing platform levels, the overall reclamation volume decreases. The estimated difference in reclamation volume between the highest and lowest platform levels is approximately 80 million cubic meters, which significantly lowers the demand for sand. This reduction creates the possibility of incorporating more nature-based solutions within the Long Island development. In future research, the integration of nature-based approaches and the application of hybrid defenses should be explored, as these strategies have the potential to enhance ecological value and recreational use. This aligns with the broader multifunctional objectives of the Long Island project.

6.3. Sensitivity evaluation approach

This section evaluates the robustness of the conceptual design through a sensitivity analysis of key cost and performance parameters. It includes an assessment of capital expenditures, present value outcomes, and the influence of economic and physical uncertainties. In addition, although not part of the formal evaluation, relevant long-term considerations such as lock-in risks and structural adaptability are briefly discussed. The section concludes with a reflection on the Multi-Criteria Analysis (MCA) and its implications for design selection.

The evaluation of initial capital expenditures demonstrates a clear cost advantage when adopting lower platform levels. This finding is supported by a sensitivity analysis on key design parameters, including material costs, sand key thickness at the sea defense, water depth, crest height at the sea defense, and settlements at the land reclamation. The analysis confirms that the relative cost-effectiveness of lower platform levels remains consistent despite uncertainties in these parameters. Material costs have the greatest influence on CAPEX, with cost differences between platform levels increasing as material prices rise; a reduction of more than 60% in material costs would be required for higher platform levels to become more economical. Additionally, variations in sand key thickness and water depth significantly affect total CAPEX, with a 2-meter increase in sand key thickness or a 10% increase in water depth leading to an estimated rise of 0.6 to 0.8 billion SGD across all platform levels. These results highlight the importance of research into the local bathymetry and geotechnical conditions to improve cost estimates. In contrast, variations in crest height at the sea defense and settlements at the land reclamation have a comparatively minor impact on overall capital expenditures.

The evaluation based on Present Value calculations revealed that the choice of pathway strategy has a relatively limited influence on the overall PV outcomes. However, the sensitivity analysis demonstrated that the Social Discount Rate plays a critical role in the comparison of platform levels, particularly due to its effect on the weighting of long-term pumping costs. A lower SDR of 0.5% increases the influence of future expenditures, decreasing the differences in PV between the platform levels. This highlights the impact of the SDR in the economic evaluation of Long Island, as it can shift the preferred platform level. In contrast, variations in sea level rise projections had only a minor impact on the PV, indicating that the design outcomes are relatively robust to climate scenario uncertainty within the considered range.

Although not included in the evaluation approach of this thesis, the risk of long-term lock-in is relevant to briefly reflect upon. This risk is particularly associated with sea level rise affecting seepage processes and the bearing capacity of the foundation. One potential lock-in arises from seepage behavior, especially for lower platform levels. Since geohydrological processes were not assessed in detail during this conceptual phase, no firm conclusions can be drawn. Future design stages should therefore include a comprehensive geohydrological analysis to evaluate the long-term impact of sea level rise on seepage processes. Another risk concerns the additional vertical loads on the foundation resulting from future reinforcements of the sea defenses. If the foundation can no longer support these loads, a different structural approach may be required to maintain stability. This applies to both sea defense options and can be further investigated following more advanced geotechnical design development. A final consideration is the technical lifetime of the sea defenses. The caisson option is estimated to last around 200 years, after which reinforcement options such as placing a concrete slab in front, installing a new caisson, or covering the structure with bulk aggregate remain feasible, as confirmed by an expert from Haskoning. The dike option relies on components such as armor rock, geotextiles, and seepage screens, which can be replaced or upgraded over time, allowing for indefinite reinforcement. While future large-scale investments may be required for both sea defenses, their limited impact on present value makes them less critical in the current design comparison.

The Multi-Criteria Analysis provided a structured framework for comparing design alternatives, but it revealed no significant differences between them. The variations in scores were minimal, and the evaluation was based on subjective weightings and qualitative judgments that lacked external validation. Consequently, the MCA does not support definitive conclusions regarding the preferred design. To enhance the robustness and credibility of future assessments, it is essential to involve key stakeholders such as PUB and HDB. Their input can help ensure that the evaluation criteria align with policy objectives and societal values. This is especially important in the context of urban land reclamation projects, where public acceptance and long-term livability are critical to project success.

7 Conclusion

Conclusions are drawn based on the answers to the main research question (Section 7.1) and the related sub-questions (Section 7.2).

7.1. Main question

The research question of this thesis is:

> **What is a resilient conceptual design of Long Island's land reclamation based on system performance and economic effectiveness?**

A resilient conceptual design for Long Island's land reclamation is characterized by a lowered platform level that significantly reduces the sand demand, whilst having long-term functional performance. The reduction in material demand enables the use of materials for creating a high-quality, nature-friendly coastline, which is desired by the Singaporean society. The findings of this thesis provide a technical foundation for detailed design development and guiding policy decisions in the next design stages of the Long Island project.

The most economically effective conceptual design combines a platform level of -4 m SHD with either a dike or caisson sea defense. This polder reclamation is technically feasible and offers substantial cost savings due to the significant reduction in reclamation volumes. Sensitivity analyses confirm its robustness under a wide range of uncertainties, including material costs, sea level rise projections, and discount rates. Both sea defense types are adaptable to future sea level rise, but the optimal choice between the dike and caisson options has yet to be determined, given their comparable construction costs. This decision can be supported by further development of geotechnical design and research into the integration of nature-based solutions.

While the polder system performs well from a technical and economic standpoint, its societal acceptance and integration into Singapore's urban context remain important considerations. In future design phases, the public perception of flood safety, the desirability of polder developments, and the ability to create high-quality, nature-inclusive coastal environments should be further assessed while incorporating stakeholder engagement. Additionally, further research into flood risk analysis, pumping system performance, and the social discount rate is recommended, as these factors directly influence long-term performance and economic outcomes. These recommendations will further inform the decision-making on platform level requirements and support the design development of Long Island.

7.2. Sub-questions

1. **What are the main drivers influencing the optimization of Long Island's land reclamation design?**

The optimization of Long Island's land reclamation is primarily driven by three cost components: reclamation volume, pumping infrastructure for polder systems, and the foundation of the sea defense.

The dominant cost factor is the volume of sand required, which varies significantly with the considered platform levels. Lowering the platform level reduces the material volumes. However, when the platform is situated below mean sea level and the reservoir water level, the reclamation becomes a polder system, necessitating additional seepage control and pumping infrastructure. The pumping infrastructure introduces some complexity, as pumping capacity and storage volume must be balanced. These systems must also account for redundancy and long-term operational and replacement costs, which are critical to the optimization framework. Furthermore, lower platform levels reduce the effective land area due to the inner slopes of the flood defenses, requiring adjustments in the optimization model. While the foundation design of the sea defense does not vary with platform level, it presents a significant opportunity for cost reduction.

2. What are the conceptual land reclamation designs based on the platform levels and sea defense options considered?

The conceptual land reclamation designs integrate six platform levels (-4, -2, 1.5, 4, 4.5, and 5.1 m SHD) with two sea defense options: a dike and a caisson. Additionally, each variant includes the reservoir dike design, effective land area corrections, expected land reclamation settlements, and the pumping system required for polder reclamations.

To maintain a focused analysis on platform-level optimization and climate resilience, nature-based solutions are excluded. The dike and caisson designs are developed in detail to enable accurate cost estimation. These configurations are illustrated in Figures 7.1 and **??**, respectively. The main differences between the sea defense options consist of the slightly lower crest height of the dike, the narrower sand key in the caisson design, and the seepage protection measures applied at the lowest platform levels (−4.0 and −2.0 m SHD), where bentonite walls are used for the dike option and sheet piles for the caisson option.

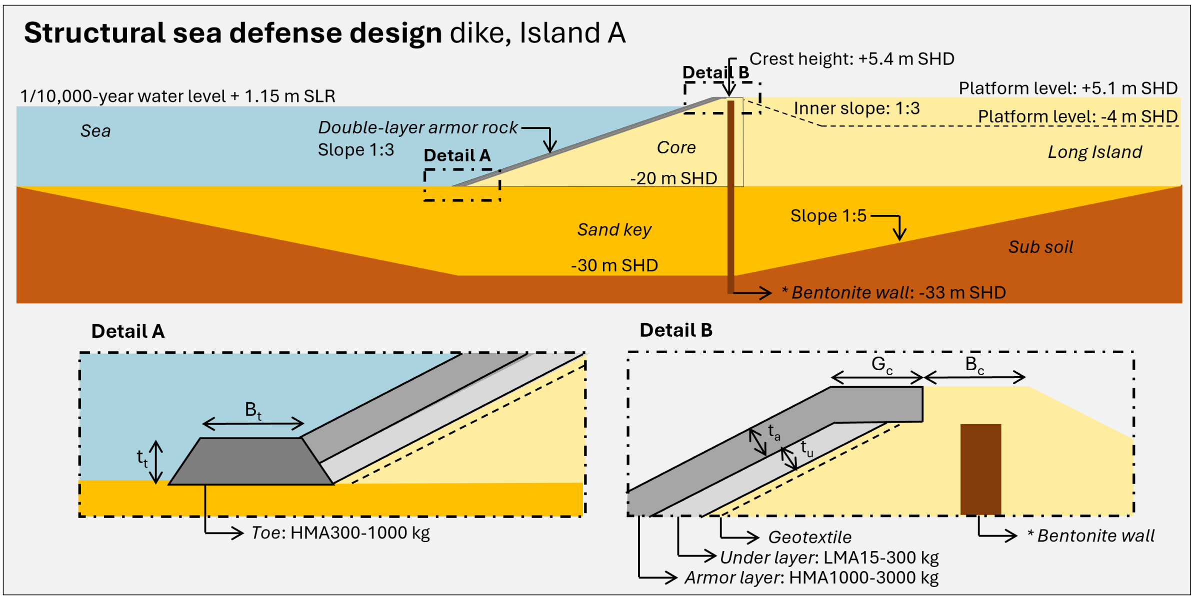


Figure 7.1: Structural dike design Island A. *Bentonite wall only applicable for polder reclamations (platform level -4 m SHD and -2 m SHD).

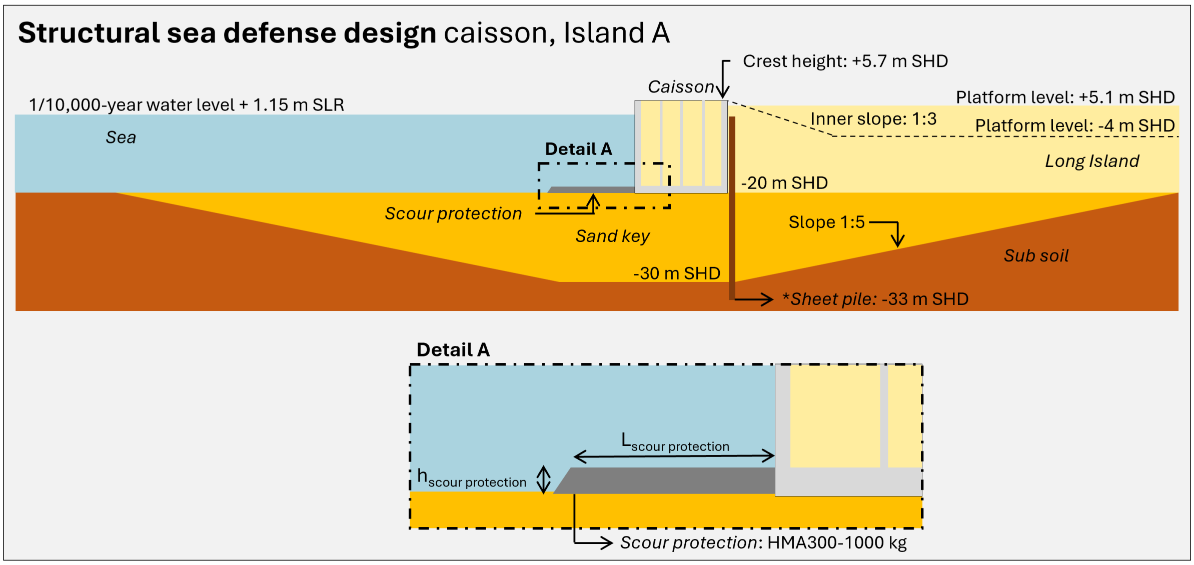


Figure 7.2: Structural caisson design Island A. *Sheet pile only applicable for polder reclamations (platform level -4 m SHD and -2 m SHD).

3. **How can the land reclamation design be adapted to address sea level rise, and what initial robust design choices are required?**

Both the dike and caisson designs can be incrementally adapted to future sea level rise. For the adaptation, until 5 meters of SLR is considered.

In general, when mean sea level exceeds the platform elevation, a seepage screen must be installed to prevent saltwater seepage. In this way, the timing of the installation depends on the platform level. The two lowest platform levels (-4 m SHD and -2 m SHD) already include a seepage screen.

Dike adaptation involves raising the crest height, which requires extending the seaward slope. This extension requires adequate subsoil strength to ensure slope stability, which can be achieved by, for example, extending the foundation during initial construction or providing some type of soil improvement at the time of adaptation. In this design, the soil improvement approach is assumed, but the associated materials and costs are excluded due to their dependency on unknown soil characteristics.

To raise the crest height of the caisson, concrete units are placed on top of the caisson, combined with inner slope extensions to integrate the raised structure into the surroundings. This inner slope extension requires reserving part of Long Island's area, which is accounted for in the effective land area correction. Additionally, the foundation should provide sufficient bearing capacity to accommodate future load increases. In the developed design, it is assumed that the sand key provides sufficient bearing capacity, but this must be verified in future design stages to ensure structural reliability.

4. **Which land reclamation design is most favorable when considering both system performance and long-term cost-effectiveness?**

Based on cost-effectiveness, the land reclamation design with the lowest platform level (-4 m SHD) is the most favorable, offering significant savings and adequate system performance. However, the evaluation of system performance through the Multi-Criteria Analysis does not provide a clear distinction between alternatives, so no definitive conclusion can be drawn on that basis.

Overall, the platform level is the most decisive factor in achieving cost efficiency. The most cost-effective land reclamation design for Long Island is a low platform level at 4 m SHD, combined with an incremental adaptation strategy (Pathway A). This approach minimizes capital expenditures by significantly reducing material use, which is the dominant cost driver for the Long Island development. The resulting capital expenditure is approximately 8 billion SGD, compared to 11 billion SGD for the 5.1 m SHD platform level. In addition, the substantial reduction in reclamation volume, estimated at approximately 80 million cubic meters, creates opportunities to incorporate nature-based solutions that could enhance the ecological and recreational value of Long Island. As both dike and caisson options are similar in cost, no definitive preference is made between them.

While adaptation strategies have some impact on long-term costs, their influence is limited compared to the platform level. Pathway A, which raises defenses in 1-meter steps, is more cost-effective than, for example, building for 5 m SLR in one instance (Pathway C), as it allows for delayed investments leading to a lower present value.

The system performance of the land reclamation variants is evaluated using a Multi-Criteria Analysis. Design criteria were defined, weighted by importance, and used to score each variant, which was then compared against its cost. However, the resulting value-cost ratios are too close to draw clear conclusions. The MCA relies on subjective weightings and qualitative judgments that were not validated with stakeholders, so no conclusive preference can be made based on system performance.

5. What are the key sensitivities influencing the evaluation of the land reclamation designs?

The evaluation outcomes are most sensitive to assumptions about material costs, sand key thickness, and water depth. These factors significantly influence capital expenditures, especially at higher platform levels. In contrast, variations in crest height and settlement assumptions have a relatively minor impact. The social discount rate also plays a critical role in long-term cost evaluations—lower SDRs increase the weight of future pumping costs, making low platform levels less favorable. On the other hand, sea level rise scenarios have limited influence on the present value outcomes, indicating that climate uncertainty, within the range considered in this study, has a limited impact on the design conclusions.

8 Recommendations

This chapter presents recommendations derived from the findings of this thesis, aimed at supporting the decision-making process of the platform level of Long Island and the further development of the Long Island project. The recommendations are categorized into two parts: strategic recommendations (Section 8.1) and design recommendations for the Long Island project (Section 8.2).

8.1. Strategic recommendations

Strategic recommendations from this thesis are directed at decision-makers in Singapore as well as stakeholders involved in land reclamation developments globally.

For Singapore, it is recommended to critically reevaluate the current platform level requirement for the Long Island development. The existing standard of 4.5 m SHD, with a potential increase to 5.1 m SHD, results in excessive material use and higher costs. To illustrate this, the estimated difference in reclamation volumes between -4 and 5.1 m SHD is 80 million cubic meters, and the difference in total initial CAPEX is about 3 billion SGD. This study demonstrates that lower platform levels can significantly reduce material demand and associated expenditures while still ensuring long-term adaptability and performance.

To support this reevaluation, it is recommended to assess the social discount rate applied to the Long Island project, as this has a direct influence on which exact lowered platform level is preferred. Additionally, a quantitative assessment of the residual flood risk associated with polder systems should be conducted. This would provide a more comprehensive understanding of the trade-offs between platform elevation and flood safety in the urban context. Considering the novelty of polder systems in Singapore's urban environment, it is also recommended to explore public perceptions of flood safety and the overall desirability of polder developments. Insights from such research can enhance communication strategies and design decisions which better align with societal expectations.

For other countries considering land reclamation developments, polder-based approaches can offer a viable and cost-effective alternative, particularly in regions where conventional reclamations are challenged by high unit costs or limited access to fill materials. This method can lead to substantial reductions in material volumes and capital investment. However, the long-term success of polder systems depends on the availability of local expertise and resources to operate and maintain the necessary pumping infrastructure and maintain the sea defenses. It is therefore essential for planners and policymakers to ensure that these institutional and technical resources are in place to support these systems throughout their operational lifespan.

8.2. Design recommendations

The following recommendations for Long Island's land reclamation design are based on the key design challenges found in this study.

- **Apply an integral design approach:** Future design phases should adopt an integrated, system-level approach. This will help capture the complex interactions between flood protection, reservoir dynamics, and water management, particularly under extreme conditions and future climate scenarios.
- **Improve reservoir and drainage system modeling:** A detailed analysis of the reservoir dynamics, drainage system and pumping operations is essential for understanding pluvial flooding at Long Island.
- **Improve the pumping system design:** Future research should include a detailed review of PUB guidelines and compare them with improved pumping capacity estimates derived from rainfall, drainage, and polder operations analyses. More accurate modeling may reveal significantly higher required pumping capacities, which would impact both capital and operational expenditures.
- **Perform geohydrological analysis:** A geohydrological analysis should be conducted to better understand seepage processes and their impact on reservoir water quality, taking into account long-term sea level rise. The findings from this analysis should be used to optimize the design of seepage mitigation measures.
- **Improve the geotechnical design**: First, alternative foundation types (e.g., deep cement mixing or stone columns) should be explored to potentially reduce construction costs. Next, the foundation design should be further detailed to accommodate the future expansion of sea defenses in response to long-term sea level rise.
- **Reevaluate long-term adaptability and lock-in risks:** Further research is needed to assess the long-term performance of the land reclamation and the potential lock-in risks. This assessment should consider the long-term impact of sea level rise, including the potential water quality reduction of the Long Island reservoir due to salt intrusion, as well as the geotechnical feasibility of future sea defense expansions.
- **Quantify residual flood risk for polder systems:** A probabilistic flood risk analysis should be conducted to assess the vulnerability of polder systems under various failure scenarios. This analysis should include damage estimates, evacuation planning, and risk communication strategies.
- **Explore alternative reclamation approaches:** Alternative platform configurations and material choices should be explored to enhance the efficiency and sustainability of land reclamation. Designs such as sloped or terraced layouts may offer a balance between flood safety and operational efficiency. For example, a platform that is above mean sea level on the seaward side and just above reservoir level on the landward side could minimize seepage and eliminate the need for pumping. Additionally, the use of alternative fill materials, such as low-quality dredged materials, should be investigated to reduce reliance on imported sand and enhance the sustainability of the dredging process.
- **Develop nature-based and hybrid solutions for sea defense:** The integration of nature-based solutions and hybrid coastal defenses (e.g., perched beaches) should be explored in future design phases. These approaches can enhance ecological value, improve recreational opportunities, and contribute to public acceptance of the project.

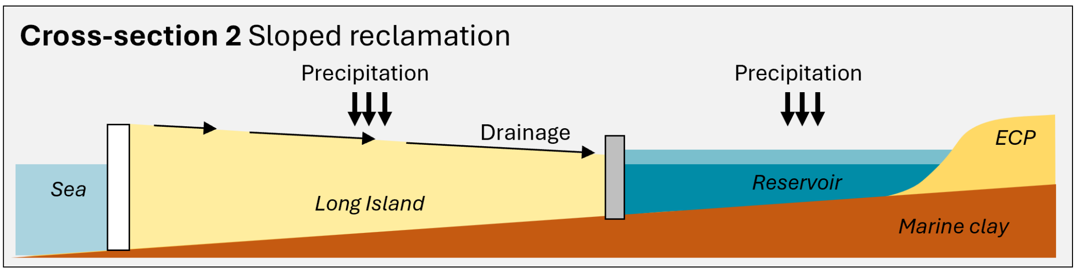


Figure 8.1: Cross-section of Long Island with a sloped platform level, indicating the main water processes.

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A
Rosin Rammler curves

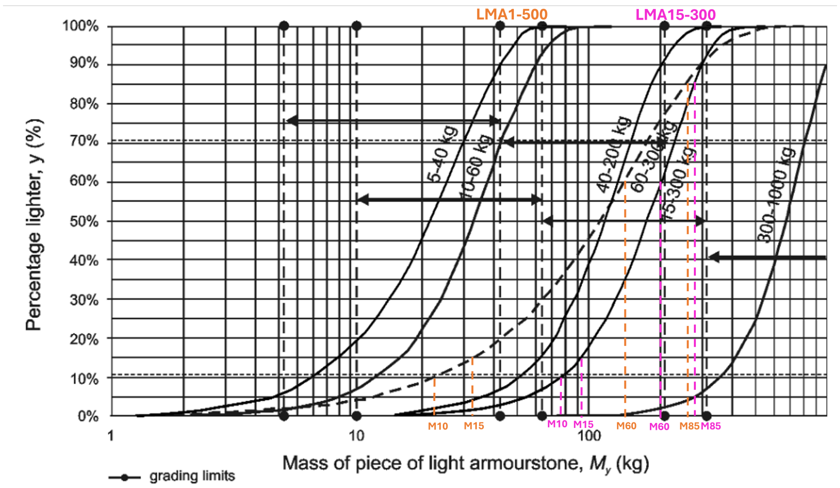

Figure A.1: Idealised Rosin-Rammler curves (CIRIA & CETMEF, 2007) in kg. Orange: LMA1-500 kg, Pink: HMA15-300 kg

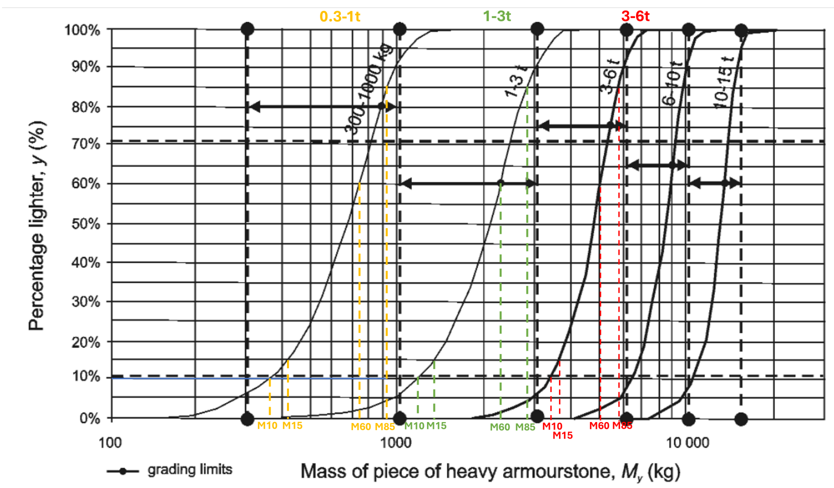

Figure A.2: Idealised Rosin-Rammler curves (CIRIA & CETMEF, 2007) in kg. Yellow: 0.3-1t, green: 1-3t, red: 3-6t.

B

Comparison wave conditions

Based on Bretschneider formulation and the overtopping criteria, two wind orientations are compared to find the critical condition to determine the required freeboard for the sea defense. For the required freeboard calculation, the dike option is considered. The resulting difference in required freeboard is so significant that the critical wave condition for the dike option is also the one for the caisson design.

The wind orientations are shown in Figure B.1. The first wind orientation results in the longest fetch, and thus in the largest waves. The other wind orientation results in perpendicular wave attack. The resulting wave conditions and related required freeboard for the dike are shown in Table B.1. It can be seen that Case 1, with a Southwest orientation and the longest fetch results in the largest required freeboard. Therefore, this is the considered critical wave condition.

Case	φ [°N]	F [m]	H_{m0} [m]	T [s]	R_c [m]
Longest fetch	135-72 = 63	65000	2.44	6.66	3.58
Perpendicular wave attack	72-72 = 0	17000	1.25	4.26	1.83

Table B.1: Required freeboard R_c per wave condition.

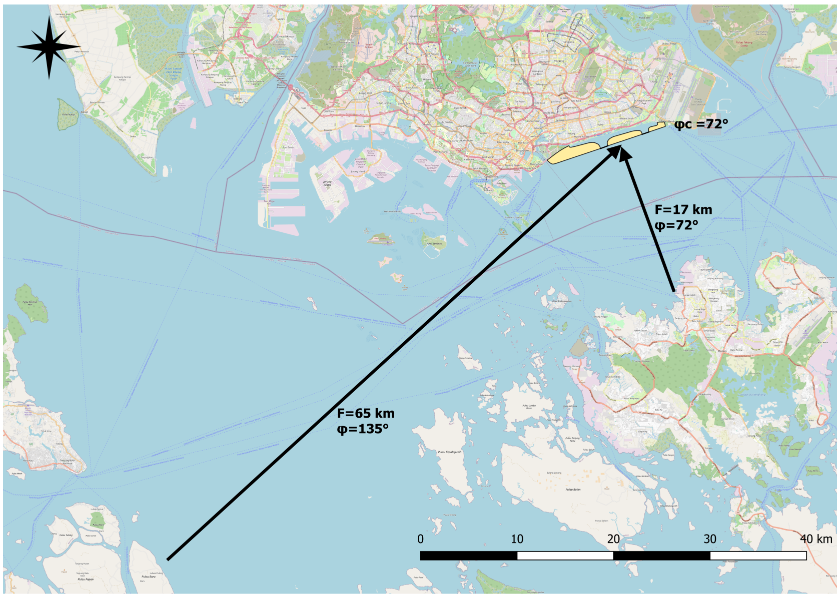

Figure B.1: Top view indicating the different dimensions per fetch case.

C
Stability criteria caisson

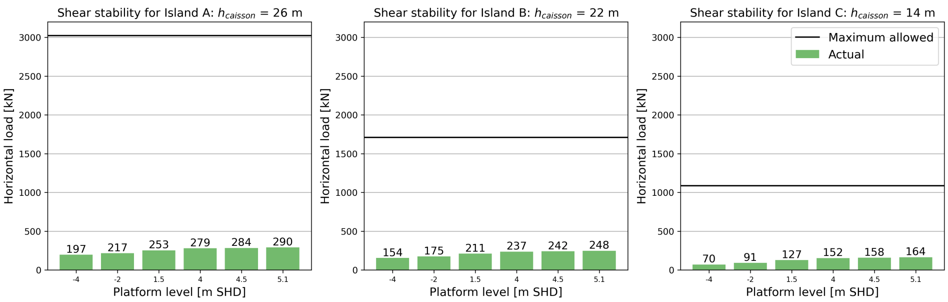

Figure C.1: Shear stability check per island and per platform level. The check is based on the horizontal load in unit kN.

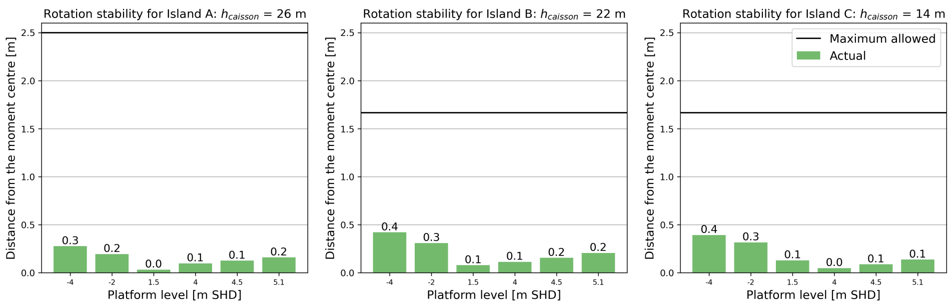

Figure C.2: Rotation stability per island and per platform level. The check is based on the distance from the moment center in meters.

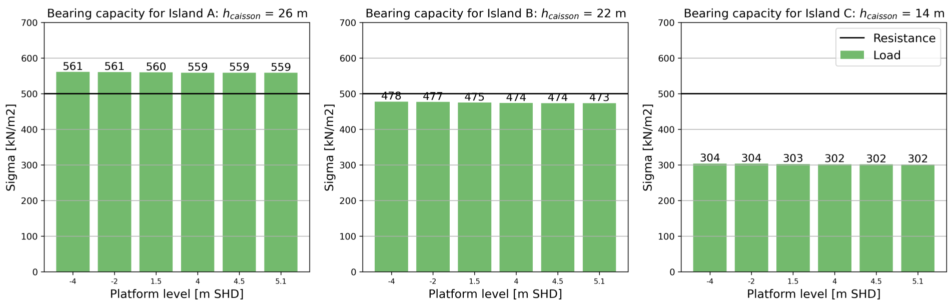

Figure C.3: Bearing capacity per island and per platform level. The check is based on the vertical pressure (sigma) in kN/m^2.

D

Settlement estimation

D.1. Estimation compression coefficient

To estimate the settlements at Long Island, the Changi Airport development is used as a reference due to its close proximity and thus high probability of similar geotechnical characteristics (Chu et al., 2009).

Based on this research, the compression constant of the marine clay can be estimated using Terzaghi's logarithmic formula (Equation D.1) (Verruijt, 2001). The strain is defined in Equation D.2.

$$\varepsilon = -\frac{1}{C} \cdot \ln(\frac{\sigma}{\sigma_0}) \rightarrow C = -\ln(\frac{\sigma}{\sigma_0}) \cdot \frac{1}{\varepsilon} \tag{D.1}$$

Where:

ε	=	strain [-]
C	=	compression constant
σ	=	future vertical stress [kN/m^2]
σ_0	=	initial vertical stress [kN/m^2]

$$\varepsilon = \frac{S}{H} \tag{D.2}$$

Where:

S	=	settlement [m]
H	=	layer thickness [m]

The average ultimate settlement is $S = -1.30\ m$, which occurs over a layer thickness of about $H = 35\ m$, so the strain is $\varepsilon = -0.037$ (Chu et al., 2009). Based on the initial loading σ_0 and future loading σ, the compression index is $C = 25$, which is within the range for clayey soil (Verruijt, 2001).

$$\sigma_0 = H_w \cdot \rho_w \cdot g \tag{D.3}$$

Where:

H_w	=	water depth [m], in this case 11.6 m
ρ_w	=	salt water density [kg/m^3]

$$\sigma = (H_d \cdot \gamma_d + H_s \cdot \gamma_s) \cdot g \tag{D.4}$$

Where:

H_d	=	1.5 [m], layer thickness dry sand
ρ_d	=	1680 [kg/m^3], bulk density dry sand.
H_s	=	14 [m], layer thickness saturated sand
γ_s	=	2000 [kg/m^3], bulk density saturated sand.

D.2. Settlements Long Island

Based on the compression constant C it is possible to estimate the settlements that will occur at Long Island based on the platform level. The results are summarized in Table D.1.

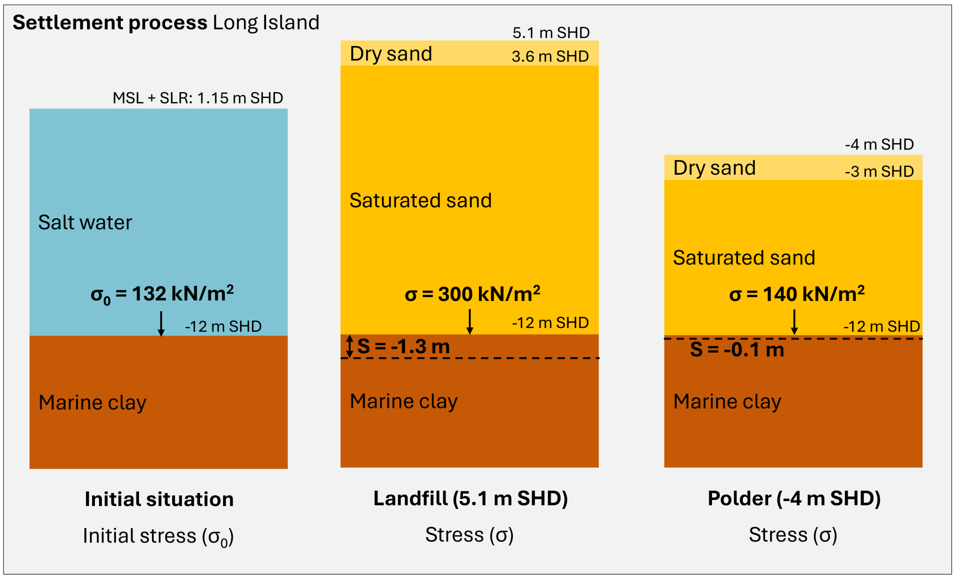

Figure D.1: Settlement process Long Island, for two reclamation cases: middle is 5.1 m SHD, right is -4 m SHD.

First, the initial stress σ_0 is calculated, which is equal to $\sigma_0 = 132\ kN/m^2$. Based on the GWT, it is possible to calculate the loading caused by sand layers, which in term determines the strain ε. By again using a marine clay thickness of $H = 35\ m$, the resulting ultimate settlement can be calculated.

Platform level z_{LI}[m SHD]	Ground water table GWT [m SHD]	Loading σ [kN/m^2]	Average settlement S [m]
-4	-5	154	-0.1
-2	-3	193	-0.4
1.5	0.5	260	-0.9
4	2.5	309	-1.2
4.5	3	319	-1.2
5.1	3.6	331	-1.3

Table D.1: Settlement of the land reclamation of Long Island per platform level.